

Disclaimer

The Technische Adviescommissie voor de Waterkeringen (TAW) has taken the greatest possible care in the preparation of the information included in this publication. This information gives the state of the technology at the time of publication. Nevertheless, the possibility that this publication may contain some errors cannot be excluded and those who make use of it do so entirely at their own risk.

The TAW makes no guarantee that the methods, calculations and data in this report are free from error. The application of the methods and results are at the risk of the user. On behalf of itself and of all who have contributed in any way to this publication the TAW disclaims all responsibility for damages, whether direct, incidental or consequential arising, from such applications or from any other use of this publication.

Foreword

With its publication of the report *Fundamentals Water Retaining Structures* (*Grondslagen voor Waterkeren*) in 1998 the Technical Advice Committee on Water Retaining Structures (TAW) moved towards the creation of a more closely coordinated series of guidelines and technical reports. In the guidelines the accent lies on the process of dimensioning and the choices and criteria for the design, the management and the calculation methods to be used. The calculation methods and techniques for the design and management of water retaining structures are included in technical reports. In practice this will lead to more concise integral guidelines than those previously followed.

To promote the coherence between the technical reports themselves, technical reports have been made on the various main aspects of the structures (for example as revetments) associated technical reports were made. These reports describe the main aspects and all the component aspects that play a role in this field of operation. For various component aspects the reader is referred to specific technical reports. The coordinated technical reports thus form an intermediary between the various guidelines and the different technical reports on component aspects in which the results of the TAW research are included.

The technical report '*Water Retaining Earth Structures*' is one such report and is an aid to the design and management of water retaining structures in relation to geotechnical aspects. The report covers all the geotechnical aspects that play a role and give the necessary references with regard to new developments in research, methods of calculation and standardisation. The report confines itself to the water retaining function with other functions, including residential, traffic and agriculture functions.

The *Guidelines on Sea and Lake Dikes* (*Leidraad Zee- en Meerdijken*) that was published in 1999, was the first title in the new-style. In relation to the transition stage, a basis report was added to this which, amongst other things, included background information about the geotechnical aspects that are important in the dimensioning of the structures. This *Technical report on Water Retaining Earth Structures* replaces the relevant sections of that basis report.

Both the policy on safety and the science and the technology relating to this are subject to change. Therefore from time to time it is necessary to publish new guidelines and technical reports. The need for the publication of new technical reports will derive from the newly developed scientific and technical knowledge. By periodically revising the *Technical Report on Water Retaining Earth Structures* it will be possible to provide the designers and managers with the necessary overview of technical developments as up to date as possible.

1 Introduction

1.1 Objective and scope

Since time immemorial earth structures have been used as water defences in the Netherlands and despite the fact that hydraulic structures and other special structures are being used to an increasing extent, they will undoubtedly continue to play an important role in the future. In this report the term 'earth structures' covers sea, lake and river dikes, dams, drainage canal embankments and polder dikes. Dunes are not considered to be earth structures.

Both the demand for more accurate designing and the assessment of the earth structures are always relevant. For this reason where necessary procedures for monitoring in the context of the five-yearly safety testing are included. This technical report is devoted entirely to the water retaining function of earth structures and is designed to be a compilation of the geotechnical aspects of water retaining earth structures. Other functions such as residential, traffic and agricultural functions are considered only in so far as they interact with the water retaining function.

Developments in the field of research, calculation methods and the setting of norms are continuous and this has led to a string of publications over recent decades. Reference is made to these at the relevant points in this publication. Geo-technical issues are covered in so far as they are required for applying various calculation methods. For further background material, reference may be made to publications that cover the material in greater depth. Issues related to soil structures which are not specifically geo-technical in nature are summarized and, where applicable, references to background publications are included. It should be noted that most of the publications are in Dutch. The names of publications to which reference is made in the report have been translated but the Dutch titles are given in the bibliography.

1.2 Place of the Technical Report within the series of TAW publications

The Technical Advisory Committee on Flood Defences (TAW) provides a coherent series of publications. They can be grouped in three categories:

1. *Fundamentals of Water Retaining Structures:*
Describes the general, legal and social framework for all types of water-retaining structures:
- 2a. Guidelines for all types of water retaining structures
 - *Guidelines on safety monitoring:* Describes the monitoring methods and regulations for all types of water retaining structures
- 2b. Integral guidelines per type of water retaining structure, each providing an integral picture of a specific type of water-retaining structure in which, per type, the general framework and rules, as given in *Fundamentals of Water Retaining Structures* are specified and explained. Examples of such reports include:
 - *Guidelines on hydraulic water retaining structures and special structures.*
 - *Guidelines on sea and lake dikes.*
 - *Guidelines for sandy coasts.*
 - *Guidelines on river dikes* (not yet published, two publications still in use cover the Upper and Lower Rivers).

- 3a Technical reports that cover the various main aspects of water retaining structures. They provide an intermediate stage between the guidelines and the various technical reports that cover specific issues (3b) Examples of such rep[ors include:
- *Technical report on water-retaining earth structures* (this report)
 - *Technical report on revetments* (in preparation)
 - *Technical report on external loading* (not yet published)
- 3b Technical reports covering individual issues:
- *Technical report on sand-carrying springs*
 - *Technical report on the erosion-resistance of grass as a dike revetment*
 - *Technical report on asphalt structures* (not yet published)

In the integral guidelines per type of work, the accent lies on the dimensioning process and the options and criteria for the design and management and the calculation methods to be used. The technical reports cover the calculation methods and techniques for the design and management of works. Both the safety policy and the science and technology are subject to change, so from time to time new publications will be issued. The need to publish new guidelines will primarily lie in the newly developed safety policy. The need for the technical reports will be determined by new developments in science and technology. From a geotechnical point of view the *Technical report on water-retaining earth structures* functions as an intermediary between the various guidelines per types of works and the technical report on specific issues. This relation is shown schematically below.

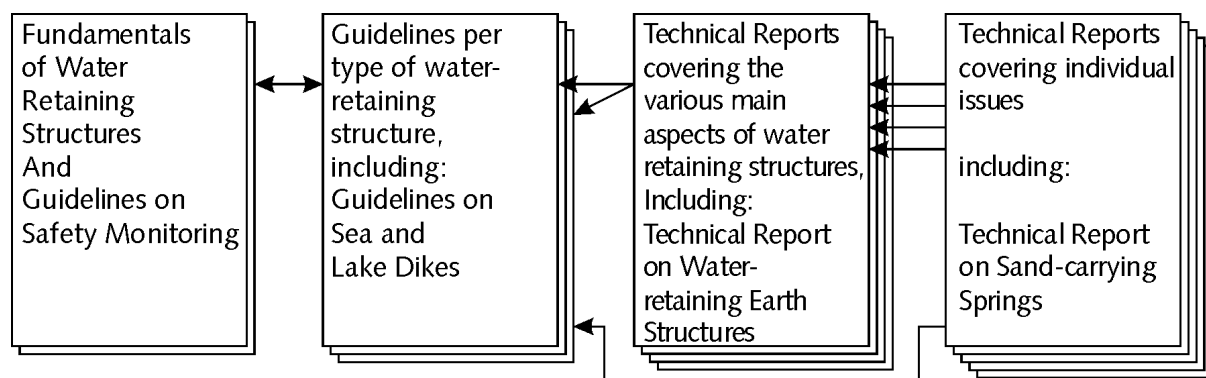


Figure 1.1.1 Relationship between the various TAW publications

This technical report does not replace any well-known publications in the field of earth structures, such as those of the CUR. It provides a comprehensive review of all aspects that are relevant to the design and management of earth structures, special attention being devoted to the latest developments in this area. Relatively new aspects such as the optimization of the transverse profile are covered in detail. Issues such as the characteristic estimation of the shear strength are also covered in detail and are illustrated by means of a practical example. The *Structural Design Manual* will still apply. A summary of various aspects covered in this manual is included in this report, but for further details reference should be made to the manual. If there already is a specific Technical Report for a particular aspect relevant to water-retaining soil structures, a summary of it has been included in this report, but for the full text reference should be made to the relevant technical report.

The *Guidelines Sea and Lake Dikes* is the first of the new integral guidelines in accordance with new policy that is being followed by the report entitled *Fundamentals of Water Retaining Structures*. During the transition phase in order for the TAW to produce a more cohesive series, basic report with technical information has been included with this publication. In the future the information contained in the basic report will be included in various new technical reports. The *Technical Report on Water-retaining Earth Structures* replaces the chapters of *Fundamental Report on Designing Sea and Lake Dikes* that cover

the geotechnical aspects. This means that the Chapters B4 'Optimization of earth structures' and B5 'Soil Mechanical aspects' are omitted from the basic report.

For a complete survey of current guidelines and technical reports and the relationships between these, reference should be made to the subject 'Publications' on the web site of the TAW at:

<http://www.waterland.net.taw>

1.3 Target Group and area of application

The Technical Report is intended for all who are professionally involved with water retaining structures and serves as an aid in following or supervising the of the technical design and as an aid in the monitoring of the structures. It also provides information for management and maintenance.

The Technical Report is intended to cover primary water defences. Although this report can be used for secondary water defences and drainage canal embankments, it should be noted that the safety philosophy and the safety level may differ and that the hydraulic loading for secondary water defences and drainage canal embankments has an entirely different character.

1.4 Readers guide

The various phases and components in the design process are covered consecutively in the way that would generally be followed when drafting a technical design. Chapter 2 covers the various components of a water-retaining earth structure. Chapters 3 and 4 cover loading and soil-mechanical aspects and the geo-hydraulic structures respectively. The dimensions are covered in Chapter 5 Failure Mechanisms. Chapter 6 is devoted to the optimization of the transverse profile, covering, consecutively; the role of soil analysis, the taking into account of actual strength and the possibility of more advanced probabilistic calculations. Chapters 7, 8 and 9 cover issues related to water retaining earth structures. These respectively include revetments needed to protect an earth structure, structures that do not have a water-retaining function and special structures. The latter may serve to replace part of an earth structure if, for example, an earth structure would take up too much room. In Chapter 10 an overview of constructional aspects is given. An example of how this report may be used is provided in Chapter 11. The technical report includes a number of annexes that cover specific issues in greater detail.

1.5 Area of application

□

The Technical Report is intended for primary water defences. This Report can be used for secondary water defences and drainage canal embankments. It should be noted, however, that the safety philosophy and the safety level may differ and that the hydraulic loading for secondary water defences and drainage canal embankments are entirely different in character.

1.6 Formulation of this report

The technical report was commissioned by the DWW under the auspices of the (TAW) by GeoDelft in collaboration with Fugro Ingenieursbureau B.V., Arcadis Heidemij Advies B.V., Grontmij, Ingenieursbureau Amsterdam and Infram.

The project group comprised the following participants:

Ir. L. van Asperen (DWW), TAW-coordinator, from February 2000)
Ir. E.O.F. Calle (GeoDelft, author, from February 2000)
Ir. J.R. Deutekorn (Grontmij, author, up to February 2000)
Ir. E. van Hijurn (Infram, author, up to February 2000)
Dr. ir. G.J.C.M. Hoffmans (DWW, project supervisor)
Ir. R.H.J. Kremer (Ingenieursbureau Amsterdam, editing from February, 2000)
Ir. B.A.N. Koehorst (DWW, author)
Ir. M.T. van der Meer (Fugro, author)
Ir. J. Niemeijer (Arcadis Heidemij Advies, author up to February 2000)
Dr. ir. M.A. Van (GeoDelft, editing, to February 2000)

The project group was supported by members of TAW-B (up to December 1999) and TAW Techniek

2 Analysis of the structural design

2.1 Introduction and objective

Functions and values

The primary function of a dike is to ensure safety, in other words to protect the hinterland from inundation in the event of high water levels. In addition, values relating to landscape, nature and the cultural heritage (LNC values) also play a role, as do the values that have always been associated with dikes such as the provision of sites for housing, carrying traffic, agriculture, water-based activity and recreation. Because so many different functions and values play a role when a water-retaining structure is to be constructed or improved, the choices and decision-making processes are much more complicated than those involved when simply solving a technical problem. This decision process is described in the various guidelines; the most important of which is the Guide 'Fundamentals of Water Defence'. The results presented in this technical guide for the structural design of a dike are true to a greater or lesser degree for other water-retaining structures, such as dams, quays and drainage canal embankments.

Structural Design

This Technical Report, and particularly this Chapter, is specifically concerned with the analysis of the structural design. Besides demands relating to water-retaining capacity, additional demands relating to LNC- values or other uses may also apply. These additional demands are only covered in this report in so far as they relate to the water-retaining function.

The water-retaining capacity (strength) of a dike is characterized by the crown height, and the stability of the dike body:

- Demands with respect to the elevation of the crown relate to the permissible overtopping capacity. The definition of this level depends on the resistance of the dike to erosion and to soil surface shearing on the crown and inner slope as a result of overtopping water. The accessibility of the crown and any berms on the inner slope to any type of traffic under the normative conditions also play a role.
- The stability of earth dikes depends primarily on the inner and outer micro-stability. These can be strongly influenced or determined by the resistance of the slopes to piping and micro-instability.

In addition the stability is influenced by the revetment and the water retaining and other objects on or in the structure

The analysis of the structural design includes:

- The decomposition of the dike into elements, in both longitudinal and in transverse profiles
- The structural function of each element, or in other words, the contribution of each element to the water-retaining capacity of the dike.

The analysis of the structural design lays a basis for optimizing this design later on in the design process (Chapter 6).

2.2 Longitudinal composition of a dike

For a clear description of the relevant components on, in or near a water defence, a dike can be subdivided into various elements, in both its longitudinal and in transverse profiles. Lengthways, the distinctions are first made according to the type of water defence (dike, hydraulic structure, and dune).

When subdividing a dike into sections, the picture given should be as exhaustive as possible with respect to the characteristics and elements present. These include:

- The geometry of the dike or dam profile, including the presence or absence of stability, heave or piping berms, the geometry of the foreland and any channels in this, and the presence of bodies of water in front of or behind the dike (scour holes, berm ditches and similar features)
- The structure of the dike body (including historical dike structure) and the substratum. Here the soil-mechanical conditions and the design criteria that play a role in the construction of the dike are important. It is necessary to make a distinction between the various types of substratum and characteristics such as compressibility, creep, permeability, and sensitivity to settlement flow and shear strength.
- The hydraulic loading, in which the degree of exposure and presence of a foreland play a role, in addition to other loading.
- The type and location of revetments.
- Special structures such as filters, drainage, seepage screens and soil-retaining structures
- Transitional structures linked to structures such as piers or land abutments, shore-parallel embankments or breakwaters and similar structures
- Longitudinal connecting structures (with hydraulic structure, dune or high ground)
- Elements resulting from shared use of the dike, in particular the presence of vegetation and of other elements such as buildings, windmills, cables and pipelines.

The location of the various elements is related to the size of the dike. An attempt is made to introduce dike stretches that are as long as possible, certainly no less than 50m. It may happen that, for the assessment with respect to height, different section sub-division is used from that used in the assessment for stability. Different sub-divisions of sections may also apply for the various aspects in the assessment for various stability aspects.

2.3 The composition of a dike in transverse profile

In addition to a longitudinal description of a dike, the location of the elements should also be indicated with respect to the transverse profile. A representative transverse profile has been assigned to each dike section.

Technical management limits

The limits of the profile should be given on both inner and outer sides of the dike. The management limits depend more or less on the type of dike (river, sea or lake dike). It is generally held that the technical management limits of the profile should be chosen in such a way that changes in external conditions do not result in the direct danger of failure or collapse of the dike, or sections thereof. The limits must therefore be chosen with sufficiently large margins, or be based on calculations.

The elements

Figure 2.1 shows the principle profile for a dike. The subdivision of the transverse profile into elements has been performed on the basis of layers/structures and measurements/slope. The elements indicated are found in both sea and river dikes. The main profile of a lake dike is comparable to that of a sea dike, with the proviso that the normal water level changes very little and that the splash berm transition berm (kreukelberm) is normally at this level. Special water retaining structures are not shown in Figure 2.1.

Elements and dike type

The primary function of an element and its importance in the dike design depend on the type of dike. A river dike is exposed to a relatively long period of high water levels. For this reason, attention is primarily devoted to elements of the structure of the inner side of the dike that play a role in piping, inward macro-stability and micro-stability. For sea dikes, loading is of relatively short duration and is to a high degree characterized by wave attack. Here, extra attention is devoted to the outer slope, crown height and revetment. Although an element

may have various functions, and the primary function is dependent on the type of water-retaining structure, no distinction is made between types of water-retaining structure in this report. This is because in essence the elements can be used for all types of dike and differing only in dimension.

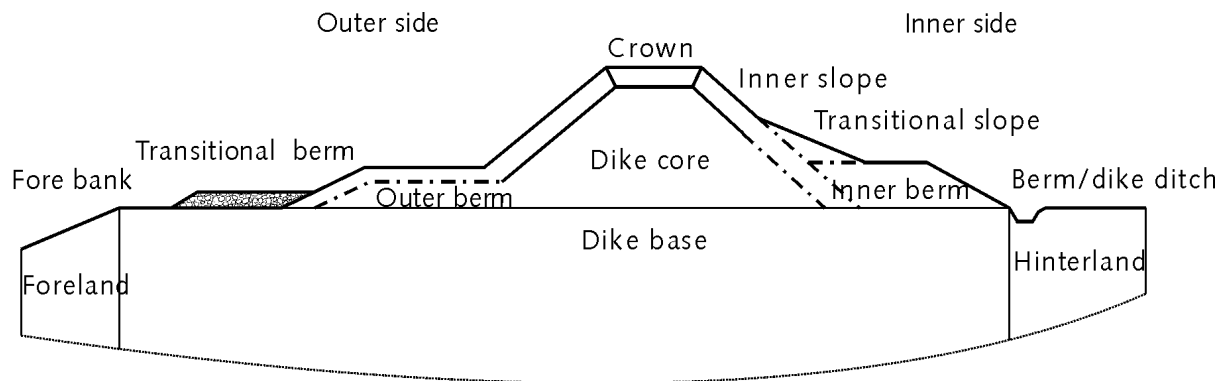


Figure 2.3.1 General profile sea/river dike without special elements

Figure 2.3.1 provides a schematic representation of a dike profile: the figure shows possible combinations of elements in a dike. The limits of the revetment are represented as dashes where elements (such as inner and outer berm) are connected to the dike body. The above figure also gives a geometric division of the substratum into foreland, dike base and hinterland. However, when considering the failure mechanisms (Chapter 5), different zones are used. For example, for the stability a different zoning division assessment is used for the soil properties (for example inner toe, crown and outer toe).

Element groups

The transverse profile of a dike is composed of the following groups and subgroups:

- Elements relating to the substratum
 - Forebank and foreland
 - Dike base
 - Hinterland
- Structural elements
 - Transition berm (kreukelberm sea or lake dike) /plasberm in river area)
 - Outer berm (incl. Revetment)
 - Outer slope (incl. Revetment)
 - Dike core
 - Crown (incl. revetment)
 - Transitional slope (inner dike)
 - Inner berm
 - Berm ditch
- Special elements
 - Seal of water-bearing layer
 - Screen in crown
 - Drainage
 - Other elements

In addition to those mentioned above, water defences contain other elements, such as road foundations. Since these elements do not have a direct water-retaining function, they are not considered. Revetments are covered in detail in Chapter 7. The revetment function has been incorporated in this chapter for the inner and outer slope elements and the crown elements.

2.4 Structural functions of the elements

□

The water-retaining capacity of a dike is primarily determined by the crown. The height of the crown determines the probability of overflowing and the degree of overtopping. The main purpose of the elements in the transverse profile is therefore to ensure that the crown is at the right height and to maintain the crown in this position. At the same time the structure of the earthwork must be so dimensioned that this can resist the design load. This means that under the load the cohesion of the elements may not be broken and that the structural function of the elements themselves may not be disturbed. The role of the elements in bearing these loads and in the prevention of damage is considered below. In Chapter 3 the loads are discussed in greater detail and the relation with the development of damage is shown in Figure 3.1.1. The starting point that the structure must be constructed with earth entails that the specific property of earth, that it consists partly of water, must be taken into account. Moreover certain loading leads to an increase in the groundwater pressure which also runs through the various elements.

Crown

When at the correct elevation, the crown height prevents overflowing and an overtopping capacity that is too high. The crown height and width and the composition of the revetment must be so chosen that the overtopping remains within the design criteria. If the crown fails, damage is not restricted to the erosion of the element itself, but also affects the inner slope, the transitional slope and the inner berm. The combination of erosion and infiltrating water can lead to slip-off on a micro or a macro scale. The reduction in the strength of the soil caused by the increase in the water pressure is a risk factor that must be taken into account in choosing the composition of the soil and of the other elements.

Dike core

The dike core is the bearing element on which the various elements of the transverse profile are laid. The first requirement is that the core itself is stable, under both the loading of the dike elements and external loading. The dimensions of the core and the slopes are calculated with the aid of strength parameters of the type of soil in the core and of the various layers of the substratum. When determining the types of soil and the construction method to be used, it is necessary to take into account the sensitivity to settling flow. The method of calculating this flow is given in Chapter 5.

The weight of the core, together with the shear strength of the substratum, takes care of the strength against lateral displacement of the dike at the Normative High Water level NHW. However the core (together with the other elements) loads the substratum, as a result of which considerable settlement usually occurs. The necessary extra height to compensate for this settling and the settling of the core itself must also be taken into account in the calculation of the height of the core.

In the dimensioning of the core and the choice of materials it must also be taken into account that the relation between the permeability of the core, the substratum and the slopes will affect the water pressure in and the shear strength of the dike. As long as the soil pressure remains the same, the increasing pore pressure reduces the shear strength. This affects both the macro-stability and the micro stability. One must consider slip-off or erosion of the outer slope when water levels fall rapidly when water pressure in the dike is high or shearing and wash out of the inner slope if there is dike seepage when water levels are high.

The width of the foot of the core determines the length of the seepage line for groundwater that moves through water bearing sand layers under the dike through to the polder and thus reduces the resistance of the water retaining structure to erosion by piping. The composition of the dike core will generally be unsuitable to bear the loading from waves and currents. The slopes of the core are therefore protected by revetments. For various constructional reasons berms can also be included in this.

Slopes and berms

Slopes and berms possess the common characteristic that they play an important role in the stability of the dike. In general, the more gentle the slope the wider the berm and the greater the stability. Slopes and berms and their revetments form the protective skin of the dike.

Outer slope

The design of the outer slope is directly related to the loading by waves, which incidentally takes place in association with the position and dimensions of the outer berm. The wave run-up and thus the overtopping capacity can be reduced by using a lower angle of slope and this in turn can then result in a lower crown height. The choice of revetment also influences the wave run-up and must be in compatible with the various hydraulic loads.

When selecting the angle of slope in relation to macro-stability the construction of an outer berm should be taken into consideration because both have influences. Attention must also be paid to the construction phase during which temporary increases in water pressure occur. During this phase the use of a wider outer berm has a greater effect on the stability than does a gentler angle of slope. When choosing the material the permeability must be taken into consideration owing to its influence on the water pressure in the entire earth structure, which again is important to the macro and micro stability of both the inner and outer berms.

Outer berm

In view of its water-retaining function an outer berm may be used to limit wave overtopping of sea and lake dikes. The position of the berm is determined on the basis of hydraulic loading. In the design the effect of a berm should be evaluated together with the effect of various angles of slope. The weight of the outer berm influences the macro-stability of the outer slope to a degree that depends on the place where the measurement takes place. The influence on the effective stress and pore pressure in the dike core also extends to the stability of the inner slope. With regard to the permeability, this is similar to that of the outer slope. Finally the berm also plays a part in the length of the seepage line, which is important for the required resistance to piping.

Low outer berm/ transitional berm (plasberm/kreukelberm)

When a hard revetment is used the edge of this must be protected against damage from waves and currents and for this purpose a plasberm may be constructed just above low water level for sea dikes. It is a transitional structure with a 'half-hard' character, half way between the hard revetment and the soil of the further course of the foreland. For this either a fascine collar with loose stone is used or a revetment of reed or fascine mats with rubble.

Inner slope □

With high water levels, whether or not in combination with wave overtopping, the strength of the inner slope is important in various ways. First, the threat of erosion by the water flows over the slope, which means that the revetment must be able to withstand erosion. Infiltration of overtopping water brings the threat of local instability followed by more extensive infiltration, weakening and loss of stability on macro scale. By making the right choice of the limit of permeability of the revetment and greater permeability and storage capacity of the core of the dike and possibly by drainage at the toe of the slope, this risk can be reduced. In choosing the type of soil attention should be paid to its sensitivity to softening. Finally the angle of slope is normative for the safety in relation to failure of the inner slope.

For very high water levels the lateral stability is also partly determined by the angle of slope of the entire earth structure. In this state of loading the high water pressures in the dike and the substratum reduce the resistance to displacement. Out-flowing seepage water in the inner slope can lead to total instability. When the inner slope cannot adequately take up these mechanisms the stability can be increased by including an inner berm and/or a transitional slope in the design.

Transitional Slope

A transitional slope is used to increase the resistance of the inner slope to lateral displacement and/or to prevent the occurrence of macro-instability caused by the emission of seepage water. The height of the transitional berm is usually determined by the height at which the seepage water would possibly be expelled.

Inner berm

An inner berm can be used to ensure the stability of the inner slope. When dimensioning this berm it is important to assess not only the increase in shear resistance in possible shear surfaces, but also the degree to which efficient use can be made of the contribution to the opposing moment within the slip circle (see § 5.3.3). The second reason for using an inner berm is closely related to the groundwater flow and the water pressure in the dike foundation. Seepage with a steep hydraulic gradient leads to a high risk of internal erosion or piping in an aquifer. The width of the inner berm can be so chosen that the length of the seepage line is long enough to prevent piping. The water pressure can also lead to uplift or heave of the covering layer of the dike foundation or the hinterland. The size of the inner berm can be related to the height of the water pressure and the width of the zone over which the covering layer has insufficient stability.

Berm or dike ditch

A berm or dike ditch has a favourable influence on the groundwater level and the water pressure in the dike and the hinterland. While this works positively on the stability, the thinner covering layers in the bottom of the ditch may have an unfavourable effect because uplift or heave is more likely to occur. The ditch may also be the point of exit for piping flows, so the design must take into account the length of the seepage line.

2.5 Structural functions of the substratum

Choosing a soil structure for a water retaining structure means that the substratum will form part of the structure. This entails that the substratum must do more than merely bear the weight of the dike. The substratum must be able to guarantee that the structure will maintain its position and will be dimensionally stable and at the same time will be able to retain water. Like the soil structure, the substratum is loaded by high water levels, currents and waves. Special attention must be paid to ground water flows and pressures. Piping, heave and uplift can damage the integrity of the inner berm and/or inner slope or of the hinterland (see also § 5.6).

Dike base

The base forms the foundation of the water retaining soil structure. This implies that the various layers of the substratum must be investigated and assessed in relation to their strength and deformation characteristics as a foundation for the dike.

The first consideration is the settling that will be caused by the construction of the dike and the degree to which this settling will proceed evenly in both the longitudinal and transverse directions. This settling is determined by the weak Holocene covering layers. Secondly, the base is of great importance to the macro-stability of the dike core, the slopes and the berms. The properties of the weak layers in particular have a big influence on the stability and possible horizontal deformation.

Usually the strength of the weak layers increases when loading is applied from above, as is the case of the construction of a dike. Unfortunately some time elapses before the substratum has adapted itself to this and obtained this increased strength. During the construction phase the weak layers are indeed heavily loaded but the strength has not yet increased. On the contrary, because the pore water carries the load in the first instance, the water pressure increases and with this the risk of loss of stability.

Improvement of the substratum by the construction of a cutoff trench or vertical drainage may provide a way to prevent loss of stability in the construction phase. This also ensures greater stability in the finished structure and moreover there is a reduction in settling with a cutoff trench and residual settling (with vertical drainage).

Thirdly, the fact that the substratum is part of the structure means that with higher water levels there is a hydraulic gradient over the dike base. If there is a sand aquifer the ground water flow can cause erosion and/or loss of stability by sand carrying springs (piping) and /or heave or uplift of a covering layer in the hinterland. Retrogressive erosion and failure of the covering layer can lead to loss of stability in the overlying soil structure.

In strength calculations also the effects of high water levels and pore pressures must be taken into account as well as the effective stresses and thus the strength of the substratum.

When designing or improving a water retaining soil structure it should be realized that the dike base is part of the structure and that discontinuities in the shape of strong local water carrying channels or extra weak channel sediments can impede the above mentioned structural functions. In other words: the water retaining capacity can be seriously threatened by failure mechanisms deriving from the unevenness of the substratum.

Forebank or foreland

The functions of the foreland are largely similar to those of the outer slope and the outer berm. The foreland has a big influence on the wave run-up and wave overtopping, which must be taken into account when the crown height is being determined. Usually, from the point of view of water retaining structures, one does not go so far as to increase the height of the foreland to reduce wave run-up, but when there are other reasons to consider this it can be included in the optimization (see Chapter 6).

The foreland also fulfils a function in ensuring the outward stability. For the assessment of this the composition of the substratum and the geometry of the underwater are important. A steep forebank with a deep channel or erosion hollow at a short distance from the outer toe demands that extra attention is paid to the shear strength in the soil investigation and later in the stability calculations.

An unusual form of instability that must be taken into consideration is settling flow. Depending on the breadth of the foreland and the channel slope and distance settling flows can occur on thicker sand layers with a certain grain size composition and looser packing. When this is possible it is an aspect to be considered when choosing the line of the structure. If flow settling occurs, the sand flows at a very low gradient so that if a somewhat deeper channel is present the structure can completely collapse.

With higher water levels a hydraulic gradient similar to that described for the dike base forms over the foreland. For the resistance capacity of the water retaining structure the entrance point is important. With piping through a deeper lying sand layer this may lie in a flow channel, for shallow piping through an intermediate sand layer the entrance point and the hydraulic resistance the covering poorly permeable layer must be determined.

Hinterland

For the analysis of the structural contribution of the hinterland it is necessary to have insight into the soil parameters. Often the hinterland will consist of poorly permeable clay and peat layers above highly permeable Pleistocene sand layers. Via the groundwater high outer water levels exert pressure on the separation plane of these layers as a result of which the structure can be substantially affected. Similar problems arise when there are continuous sand layers or substantial sand lenses in the clay and peat series.

The stability of the inner side is greatly influenced by the strength and stability of the covering layers because the water pressure can rise so high that heave or uplift on the separation plane may occur. This results in a considerable loss of stability on the inner side. The method of calculating this stability is given in § 5.3.3. When the covering layer is stable there is a still chance of piping. To determine the reliability of the structure on this point it is of primary concern that the composition of the soil of the covering layer and the aquifer are known. In addition the thickness and elevation of the covering layer play a role, as also does

the exit point of the piping. This can also be determined from new or existing ditches, channels or scour holes, depending on their location and depth.

2.6 Special elements

The use of special elements allows the reduction of the dimensions of the structure. Since many variations are possible, three frequently used elements are illustrated as examples. For a comprehensive description of special structural elements, reference should be made to Chapter 9.

Seal water-bearing intermediate sand layer

Sealing the intermediate sand layer prevents the possibility of piping via this layer. The pore pressure in the intermediate sand layer behind the sealing layer is reduced inside the dike (indirect influence on the stability on the inner dike side). However, the water pressure in the intermediate sand layer in front of the seal is increased. This seal may consist of a screen or a clay coffer.

□

Screen in crown (continuous in the substratum)

The screen increases the length of the seepage line (piping via Pleistocene) and prevents 'shallow' piping. As a result, piping is less normative. In addition, the screen reduces water pressure on the inner side of the dike (depending on the length), but increases water pressure in elements in the outer slope. The screen can also be used as a retaining structure, for example as a foundation wall. A screen influences the effective stress and pore pressure and, through this, the stability of the structure. The magnitude of the influence depends on the position and type of screen.

Drainage

Drainage in the toe reduces the groundwater level and the water pressure and thus increases the inner dike stability. This may also be important to the reliability of the revetment of the inner slope. If drainage is used in a water-retaining earth structure extra attention must be paid to the effective durability of the structure. In practice the design must take into account the deposition or iron oxide from the groundwater or blockage resulting from biological activity must be taken into account.

2.7 Relation between the elements and the failure mechanisms

In Table 2.7.1 the failure mechanisms that affect the various elements of the water-retaining structure are shown. This table indicates that most elements have an effect on a number of failure mechanisms or, in other words, one such mechanism brings the risk of damage to a number of elements.

Table 2.7.1 Influence of elements on failure mechanisms

Failure Mechanism	Settling	Wave overtopping	Slipping outer slope	Slipping inner slope	Settling flows	Micro-stability	Piping
Element							
Crown	x	x	x	x		x	
Dike core	x	x	x	x		x	x
Outer slope		x	x			x	x
Inner slope		x	x			x	x
Low outer berm			x			x	
Inner slope		x		x		x	x

Transitional slope		x	x			x	x
Inner berm				x		x	x
Berm ditch				x			x
Dike base	x		x	x	x	x	x
Foreland		x	x		x		x
Hinterland				x		x	x
Seal of water-bearing intermediate sand layer			x			x	x
Screen in crown			x	x		x	x
Drainage			x	x		x	x

2.8 Connecting Structures

2.8.1 Introduction

Connecting structures are used to connect different types of water defences. Connecting structures include the entire parallel or length profile of a dike, in a differing form at the transition to a dune, high ground or a hydraulic structure. Such connections can therefore be seen as large-scale connecting structures.

The different shape of the connecting section may result in greater hydraulic loading locally that is caused by turbulence and wave diffraction than was initially determined on the basis of indications described in Chapter 3. This may have effects on the morphology at the point of the transition of a dike to a dune, resulting in increased erosion. These increases in loading must be determined by the managers themselves on the basis of their local experience, possibly supported by specialist help.

The following sections cover the design of structures connecting a dike to a dune (§ 2.8.2), of a dike to higher ground (§ 8.3) and of a dike to a hydraulic structure (§ 2.8.4).

2.8.2 Dike to dune connections

The connection between a dike and a dune involves the transition from a soft water defence (dune) to a hard water defence (dike). The term 'hard' is in fact used in a relative sense and can therefore also be seen as 'relatively hard'. With this wider meaning, the connection between an 'undefended dune' and a 'dune foot defence work' is also seen as a connecting structure.

With respect to the morphological effects that occur during dune erosion (see Figure 2.8.2.1) the following can be distinguished:

- I an abrupt termination
- II a gradual transition

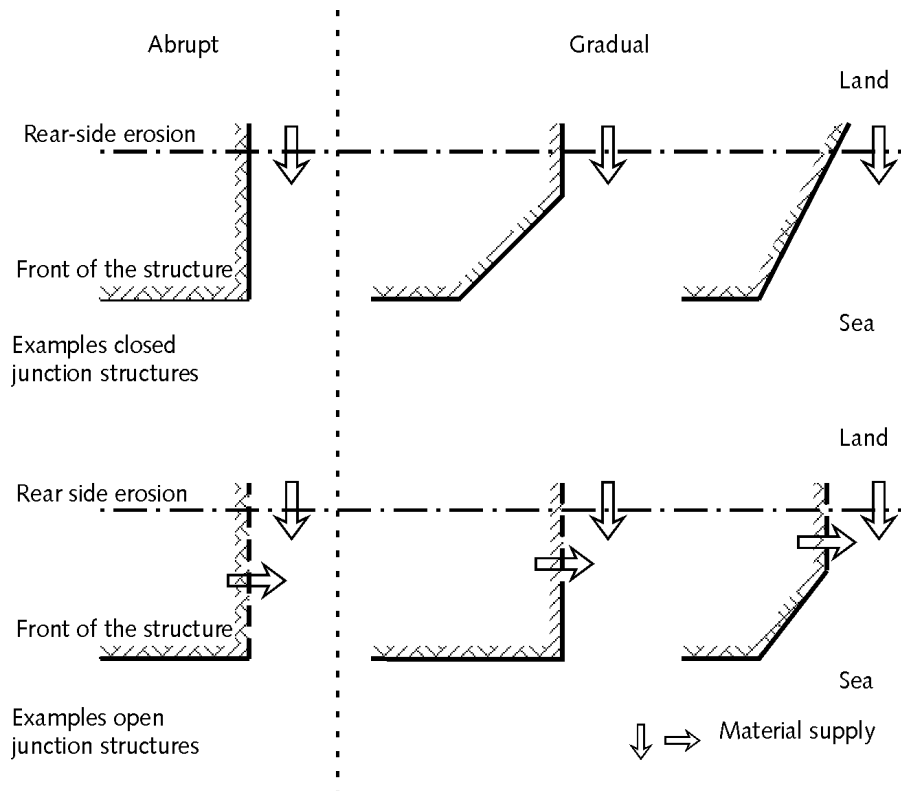
From the water defence perspective, there is an important distinction between:

- A closed termination, in which the structure extends to beyond the erosion zone
- An open termination, in which the water seeps behind the structure under normative erosion conditions

The type of transition to be chosen is partially determined by the dune strength at the point of connection. For marginal dunes, there is insufficient space to construct a closed terminal structure without digging straight through the dune. In a full dike, or dike

reinforcement, this is not a major drawback. The intervention in the entire dike in such cases is so extensive that digging through the dike is of lesser importance from both the technical and the financial perspectives. If it is only the transition that has to be constructed, it is better to avoid digging through the dune, so only the option of an open terminal structure remains, partly because the cost of such a structure is considerably lower.

The transition between a dune and a dike must be carried out in such a way that it cannot fail when the normative storm flood occurs. The transitional structure must have sufficient resistance to waves overtopping and wash out. The *Guidelines for sea and lake dikes* describes a method for preparing a draft design for the connection of a dike to a dune



2.8.1 Types of junction between dikes and dunes

2.8.3 Connection dike to high ground

In contrast to the dunes, in the map of the ring dike enclosure areas in the Flood Defences Act (De wet op de Waterkering), high grounds are indicated as the NAP+1 line for threats of inundation from the IJsselmeer and NAP +2 for threats of inundation from the sea, or if threats from the rivers are higher, as the contour line for the most extreme inundation following from the normative high water level (NHW) on the upstream side of the area enclosed by ring dikes to the lowest crown height of the primary dike on the downstream side of this area.

In principle, the same rules apply for the connection of a dike (in this case lake dikes, river dikes and dike ring separations) to high ground as for the connection of a dike to a dune. A difference may occur in that the composition of the high ground often consists of materials that resist erosion effectively.

If there is no room for an erosion profile, or if an erosion profile is not desired, the water-retaining slopes connected to the dike must be constructed to resist erosion. For main dikes that do not need to retain water under normal circumstances, the dike body usually simple

joins the local sand, the presence of which gives rise to the high ground. Figure 2.8.2 provides an illustration of the main dike along the Neder-Rijn on the Grebbeberg.

In a number of cases 'high ground' as defined here is private property. In order to be able to repair the connection of a dike to high ground, without encountering problems, the test limit must be set sufficiently high.

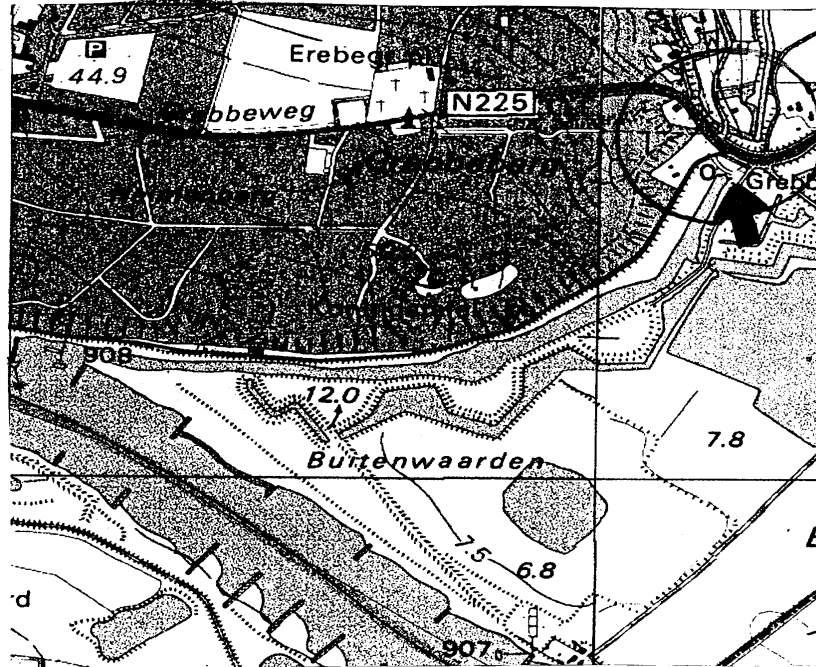


Figure 2.8.2 Connection of dike to high ground

2.8.4 Connection dike to hydraulic structure

Since the walls of hydraulic structures are almost always smooth and rigid, there is almost certainly danger of seepage (frontal and rear) along the hydraulic structure at high water. In order to prevent this, seepage screens will have to be installed below and next to the hydraulic structure. These screens must be sufficiently deep and wide to prevent seepage, heave and/or piping. The top of a seepage screen must be at least at $\text{NHW} + 0.5$ metres to prevent seepage at the rear must lie at a minimum level of $\text{NHW} + 5.0$. Design rules for seepage screens are given in the *Guidelines on hydraulic structures and special structures* and the Technical report on sand-bearing springs.

As a result of the presence of vertical walls, strong wave attack occurs at certain points. This demands special attention when calculating the dimensions of transitional structures to connect the slope revetments to the hydraulic structure.

3 Loading

3.1 Introduction

The loads on water-retaining earth constructions may be either permanent or variable in nature. In this report, they have been categorized according to cause and duration as follows:

- Permanent (see § 3.2)
 - Eigen weight and ground pressure (for special structures)
 - Extraction of material (water, oil, salt, natural gas) from the substratum
- Variable
 - Hydraulic loading (see § 3.3)
 - Other loading (see § 3.4)

Figure 3.1 provides a general representation of the relationship between loading and damage development.

3.2 Permanent loading

□

3.2.1 Eigen (own) weight and soil pressure

The weight of the dike body itself must be seen as a load. The driving moment in the analysis of the macro-stability (see § 5.3) is largely determined by this eigen weight. The eigen weight depends on the mass by unit volume of the material present in the dike (sand, clay, and road surfacing material), as well as the degree of saturation of these materials. It should be noted that the eigen weight also contributes to the strength of the dike; the shear resistance along the slip plane through and under the actual dike body increases as a result of this.

3.2.2 Extraction from the substratum

The extraction of substances such as water, oil, salt, and natural gas at greater or lesser depths can be considered as loading. Shallow water extraction will be associated with settlement in the substratum as a result of a decrease in pore pressure, and therefore as an increase in effective stress (see § 5.2.3). The extraction of oil, salt or natural gas will lead to a lowering of the ground level as a result of compaction in the extraction mass. If the extraction includes settlement or a lowering of the ground level, this affects the height of the dike body, and particularly of the crown. A supplementary over height allowance will be required to compensate for the lower crown height.

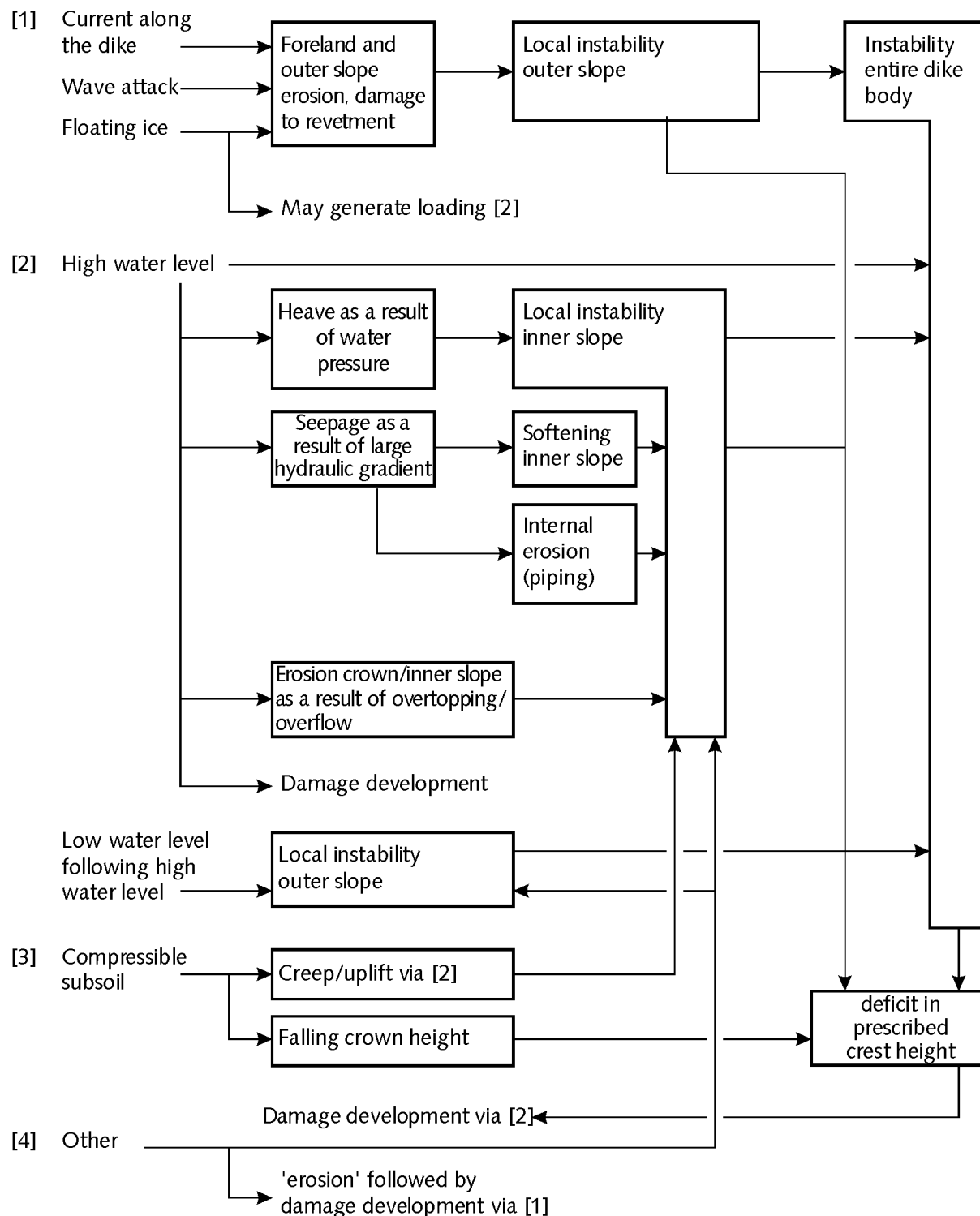


Figure 3.1.1 Relationship between loading and damage development

3.3 Hydraulic loading

3.3.1 Introduction

Hydraulic loading is related to the legal safety norm in relation to flooding as indicated in the *Flood Defences Act* (ref. *Flood Defences Act*). In this law the safety norm for the main water-retaining structures is given as the average exceedence frequency of high water level per

year. This stipulates that separate calculations must be made of every water-retaining structure, taking into account other water-retaining capacity normative factors such as low water levels, waves, currents, and fluctuations in water levels, the duration of storms, the strength properties of the structure and the substratum, subsidence of the ground and rising sea levels.

The following sub-sections form the link with the publication (ref. *Hydraulic Boundary Conditions for Primary Water Defences*) and, where necessary, provide additional information about the hydraulic boundary conditions. In calculating the dimensions of a dike, and for the execution of works on a dike, the following hydraulic parameters are of importance:

- Normative high water level (NHW0 (§ 3.3.2)
- Development of changes in water level under normative loading (§ 3.3.3)
- High water rise (§ 3.3.4)
- Local water level increases (§ 3.3.5),
- Wave overtopping height (§ 3.3.6)
- Shipping-induced hydraulic loading (§ 3.3.7)
- Wave boundary conditions for water levels lower than NHW (§ 3.3.8)
- Currents (§ 5.7.2)
- Excess pressure under a revetment (§ 5.4 and Chapter 7)
- Extreme precipitation (§ 4.6.3)

3.3.2 Normative high water level (NHW)

In *Fundamentals of Water-retaining Structures* (Chapter 3) an overview of water-retaining structure and related systems is given. A distinction is made between categories (in terms of position and the protection of the areas within ring dikes) and the types of structure

Primary water retaining structures

The monitoring level given in the boundary conditions publication and gives information only for primary water-retaining structures in Category 1 (i.e. structures that are part of the system that directly encloses the area within the ring dike and keeps out water from outside the dike).

The addition of the year to the term NHW indicates the reference year for which the NHW has been determined (e.g. NHW 2005). The boundary conditions are based on the safety norm from the Water-retaining Structures Act and are updated and published every five years. For the first Boundary Conditions publication the stipulated level is NHW 2000 = Monitoring level 2000.

When the water level norms have been expressed as safety norms these will be extended to the other categories; until this time reference should be made to *Fundamentals of Water-retaining Structures* and the interpretation of this in the management plan of the managers of the water-retaining structures.

Secondary Water-retaining structures

In a number of areas in the Netherlands in addition to the primary water-retaining structures there is a system of secondary water-retaining structures or 'sleeping dikes'. The determination of the safety level for the regional water-retaining structures is the responsibility of the provincial authorities. Determinations relating to these (not yet indicated) water-retaining structures are given in the 'Provincial Regulations for Water-retaining Structures'.

To determine the boundary conditions for polder dikes, which also considered part of the regional system of water-retaining structures, reference should be made to the *Technical Report for the monitoring of polder dikes* and the *IPO Guidelines for determining the safety of polder dikes*.

3.3.3 Water level development during normative loading

Water level development during normative loading used in designing or testing (monitoring) are covered sequentially for the following:

- Rivers
- Sea dikes
- The lake area
- The delta
- The estuaries

River dikes

For the Upper and Lower Rivers: The development of the high water wave can be supplied by HIS-R (High Water Information Service – Regional), via his@dww.rws.minvenw.nl.

Sea dikes

The following assumptions apply for dikes along the seacoast:

- During normative loading, the type of water level development (see Figure 3.3.2) is the same along the entire coast, including the Wadden area.
- Storm duration, t_s in the Wadden area is 45 hours. Along the smooth coast, it is 35 hours
- The peak of the surge is at NHW – ATA, in which ATA is the average tidal amplitude for that location. The base is at the average water level (AWL). In this figure, the average tidal curve of the location in question (known by the manager) is superimposed in such a way that the time of a peak of the tidal curve coincides with the peak of the surge. The maximum water level is equal to NHW.□

□

Lake dikes

For lake dikes, normative loading consists of a combination of a high lake level and strong winds. In determining the crown height, the influence of wind is of particular importance. For the soil-mechanical aspects, the lake level is dominant. An increased lake level can last for a period of several weeks (Ref. *Analysis of the duration of the duration of extreme high water levels of the Markermeer*). When no other rules follow from the projected calculation models such as HYDRA_M, the following assumptions suffice:

- Storm duration t_s is 35 hours
- The surge occurs as shown by the bold line in Figure 3.3.2. ¹

Dikes in the deltas

The same approach is used in the deltas as that used for dikes along the seacoast, with a storm duration of 35 hours.

Dikes in the estuaries

In the Oosterschelde and the Westerschelde, the same approach is used as with sea dikes. Here water level development during normative loading in both the Westerschelde and on the outer side of the Oosterschelde defence is the same as that which applies for the entire coast. Storm duration is 35 hours.

The presence of the storm surge barrier is a complicating factor for the resulting water levels in the Oosterschelde. The most negative development of water level with respect to the residual strength occurs when the water level fluctuates very little over for a long period of time. This occurs when the barrier is closed. Assuming water level development on the outer side of the barrier according to the previous assumptions, the barrier will be closed a number of times in succession. For this, it can be assumed that during the first water peak, the storm surge barrier is closed at a basin level of NAP + 1.00 m (centre of the basin) and that the water level in the basin will be approximately NAP + 2.00 m for the two subsequent tides. This is caused by the closure strategy defined for the storm surge barrier by which an attempt is made to set the alternating basin level at NAP + 1.00 m and NAP + .00 m. In the event of greatly increased low tide during a storm period, it will not always be possible to return from

¹ The maximum water level (top) is at $TL + (NHW) - TL$, in which TL the target level of the lake (known to the manager) and NHW is the normative crown elevation. This is based on the TL

NAP + 2.00 m to NAP + 1.00 m. For this reason, a water level of NAP + 2.00 m should be assumed for a period of 20 hours.

The top of the surge development is at NHW – ATA. Surge development is perpendicular (see Figure 3.3.1). Superposition of this surge and the average tidal curve provides the water level development under normative conditions.

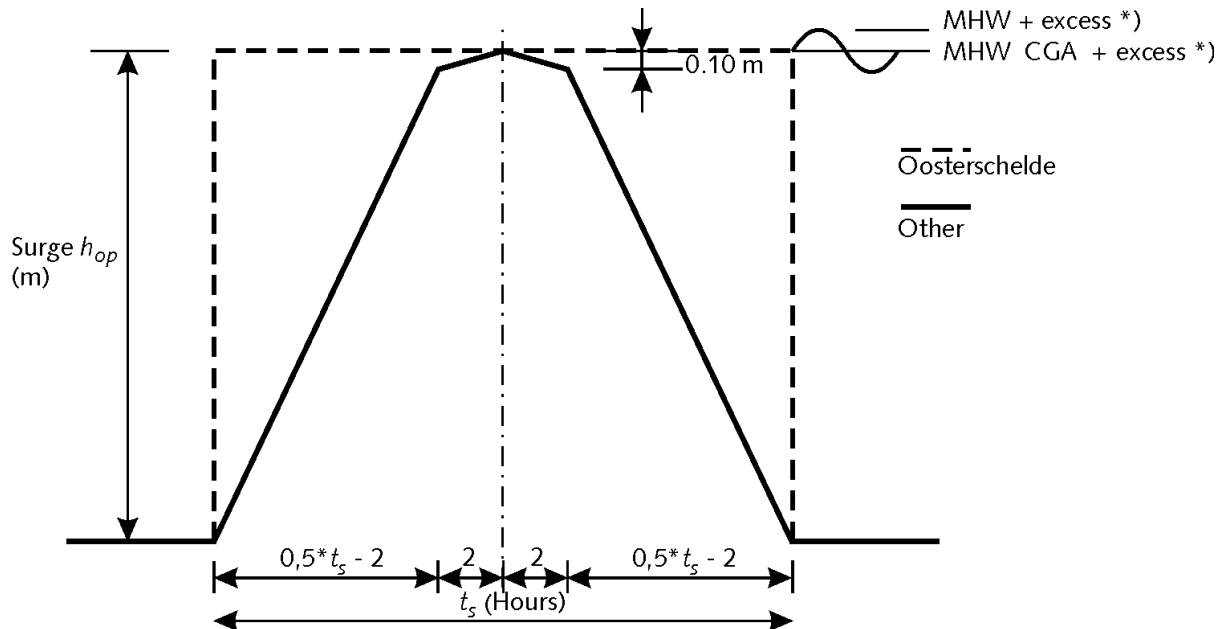


Figure 3.3.2 Development of the storm surge

3.3.4 Increase in high water level

The increase in high water level is due to the world-wide increase in sea level, the falling sea level (NAP) and possible local changes in tidal amplitude; the latter is the result of human interference in local situations and the associated morphological developments.

3.3.5 Local increases in water level

In virtually closed basins, oscillations may occur when wind loading suddenly falls away. This can result in a rise in the water level caused by wind loading changing to an equal fall in level or a to reduction in level caused by wind loading to change into an equally high rise in level.

In long, extended basins, like ports and harbours, that are closed on one side and connected to the water outside the dike, oscillations, or seiches, may occur. These entail a major increase of low frequency variations of the outer water level. Significant effects of seiches can only be expected in very extended and large basins. Examples include the basins in the Europoort area (wave periods between 10 and 180 minutes), the ports of Vlissingen (wave period less than 10 minutes), and IJmuiden (period about 10 minutes).

If no solution is provided by the (ref. *Hydraulic Preconditions for Primary Water Defences*), the magnitude of a seiche can be calculated in two ways:

- On the basis of records. In those areas in which significant seiches occur, records are usually available.
- On the basis of numeric calculations provided by the services that specialize in this field.

In addition to seiches, transverse oscillations and shorter parallel oscillations may also occur. The details of these will be found in the existing records. To calculate increases in water level caused by local winds, storm gusts, and wind oscillations, unless otherwise advised in the boundary condition guide, § 5.5 of the *Guidelines on designing river dikes; Part 2 – Lower Rivers* and Part 4 of the report of the Delta committee can be used for the coast and the deltas.

3.3.6 Wave overtopping height

The wave overtopping height is related to a design wave overtopping capacity. In the past the overtopping was expressed as a percentage of the waves that passed over the crown. In the design of dike reinforcements in the framework of the Delta Act, until recently, this was very roughly dimensioned on the wave overtopping height that is exceeded by 2% of the approaching waves.

Taking an overtopping capacity makes possible a more refined approach. Depending on the erosion resistance and stability of the inner slope more or less overtopping can be tolerated and with this a lower or higher crown height. It is necessary to take into account the desired accessibility of the structure in extreme conditions. If the crown is inaccessible for dike observers on foot it may be still be accessible to an off-road vehicle or freight truck. It is also necessary to take into account the fact that in areas with buildings directly behind the dike the feeling of security decreases rapidly with increased wave overtopping.

The relation between wave overtopping is discussed in (Ref.) *Technical report on wave run up and wave overtopping in dikes* and on (Ref.) *Technical report on wave overtopping and forces on vertical water retaining structures*.

Wind waves

If the boundary conditions for waves are not given in the boundary conditions publication, wave growth and wave formation up to the defence can be roughly calculated manually by using the method described in § 7.2 and the Annexes 9 and 12 of the *Guidelines on designing river dikes; Part 1 – Upper Rivers* (§ 5.4, § 6.3) and Annexes 7.2 and 10 in *Guidelines on designing river dikes; Part 2 – Lower Rivers*. A more accurate result can be achieved by using methods of calculation such as HISWA or SWAN. If the latter are used, it is recommended that the assistance of specialists be sought.

If a location of a dike in an outer port is exposed to such a degree that boundary conditions relating to waves are important in the design, wave penetration calculations, including refraction and diffraction will have to be made, for example by using PHAROS or DIFFRAC models. Possible wave transmission must also be taken into account.

3.3.7 Vessel-induced hydraulic loading

Shipping-induced loading is strongly location-dependent. Significant deviations from the annual average are not to be expected. The effects of this type of loading on the structure are known to the managers.

Current speeds will generally be less than 2 m/s⁻¹. Primary and secondary waves are generally less than 0.5 m, with an occasional rare exception of 1.00 m with a period of 2.00 to 5.00 m/s. Therefore shipping-induced loading does not usually play a role in the design. Should managers have any doubts regarding this matter they are advised to contact the HelpDesk Waterkeringen of the Dienst Weg en Waterbouw.

3.3.8 Wave boundary conditions under lower than normative water levels NHW

The publication (Ref.) *Hydraulic Boundary conditions for primary water defences* provides monitoring levels along the coast and the associated wave data. This combination of water levels and significant wave height is used to determine the required crown height. For other damage mechanisms, other combinations, especially those for lower water levels, may be

normative. These combinations will have to be determined by further investigation. As long as they are not known, the wave height given in the hydraulic boundary condition book may also be used for low water levels, with a maximum of 0.7 h, in which h [m] is the depth of the water in the foreland. The same argument is true for the waves to be taken into account in the safety considerations relating to the implementation of work in the summer months. If the outer water level plays a role in the stability analysis, safe assumptions must be used for this value. Calculations must be made using extreme water levels, based on high water statistics for the summer period, with an exceedence frequency that is equal to the NHW-exceedence frequency applicable for that area (Ref.) *Summer Boundary conditions*.□□

□

□

3.4 Other loading

3.4.1 Introduction

Types of loading other than hydraulic or soil-mechanical loading can be described as influences or processes that threaten the stability of the defence. These include the following:

- Wind (§ 3.4.2)
- Ice (§ 3.4.3)
- Traffic (§ 3.4.4)
- Earthquakes (§ 3.4.5)
- Soil analysis and explosions (§ 3.4.6)
- Collisions and floating objects (§ 3.4.7)
- Biological damage (§ 3.4.8)
- Chemical damage (§ 3.4.19)
- Damage resulting from climate (§ 3.4.10), en
- Vandalism (§ 3.4.11)

3.4.2 Wind

For data on wind reference should be made to (Ref.) *Hydraulic Boundary Conditions for Primary Water Defences* and also (Ref.) *NEN 6702*. The indirect influence of wind on the water defences through water levels and waves is covered in § 3.3.

Influence of wind

The direct influence of wind is present in the following cases:

- If there is a beach located in front of the outer toe of the dike, this may cause the dispersal and choking of the grass revetment. Open bank structures may also be sanded in. This aspect must be included when setting the dimensions.
- Vegetation (trees and shrubs), may transmit wind loading to the water defence. These forces may have a negative impact on the stability of the water defence, particularly under normative conditions. The magnitude of these forces can be investigated on a case-by-case basis, as also can their impact on the dike, assuming that the anchoring of the trees is such that it is impossible that they can be blown over. In the Lower River area and along the coast, the consequences of uprooting (collapse) of the foundations must be included; this implies that measures must be taken to ensure that uprooting will not lead to damage to the water-retaining component of the dike profile (see § 8.2 and the (Ref.) *Structural design manual*).
- Earth structures are pre-eminently located in places that with respect to wind supply are highly suitable for solitary wind turbines, or more especially, for wind turbine parks. However, is so far as possible the location of wind turbines within the water-retaining zone must be avoided. Only in exceptional cases can their location on or within the stability zones of the water defence be considered. This is because rotating turbines cause vibrations with possible consequences for settlement behaviour and the stability of the defence.

- The direct influence of wind on wave overtopping plays a role in both slopes (spray) and vertical structures. This impact has been included when formulating the calculation rules in the Technical Reports.

3.4.3 Ice

The Netherlands does not have fixed rules and regulations about how ice loading should be taken into account when designing and assessing structures. The temperature and the salinity of the coastal areas of the North Sea do not give rise to any significant degree of ice formation. However some cases of significant ice formation in coastal inlets and more specifically in lakes have been recorded; the closure of the coastal inlets in Zeeland has influenced degree of ice formation along the shores ice formation along the banks in Zeeland. Ice formation may also influence the water level on rivers.

In spite of the damage that ice can cause, in the Dutch situation, the probability of inundation in the hinterland as a result is relatively low. For serious damage to occur in this situation it would have to coincide with high water levels and heavy wave attack, such that the dike body under the damaged revetment could erode further and collapse. This coincidence of events is unlikely, since the wave height is greatly limited by the ice on the water.

3.4.4 Traffic

In accordance with the (Ref.) *Structural design manual* a load of 400 kN per m¹ an equivalent of 13.3 kn/m² over a width of 2.5 metres must be taken into account in the design calculations. This load must be taken into consideration even if there is no roadway on the dike, since in potentially calamitous situations it may be necessary to transport heavy materials or equipment on the dike. Annex 2 of the *Structural design manual* indicates how allowances for the loading must be taken into account in the calculations. The traffic bearing capacity must be related to the calculated overtopping capacity.

3.4.5 Earthquakes

In the Netherlands the probability of damage to a defence is very small ($< 10^{-4}$ per year). Since high water levels and earthquakes are independent phenomena, their contribution to the chance of the failure of a defence is totally negligible.

3.4.6 Geotechnical surveys and explosions

The (Ref.) *Guidelines for geotechnical Surveys in or close to water-retaining structures* covers how to handle soil drilling, penetration tests, explorative drilling, seismic soil surveys and test pile driving.

3.4.7 Collisions, floating objects**

Under normal conditions, collisions between coastal shipping and a dike will very seldom occur. However, it is just during storms that vessels are most likely to lose steerage and run up against a dike and just at such moments the consequences of impact are likely to be most serious. Firstly the impact of the vessel on the dike is at its greatest, and secondly the water level is high and there are high waves, so that initial damage can quickly spread. Ships beaching under such conditions can therefore represent a serious threat of a dike breach. The stranding of the 'Limbourg' on the Hondsbossche Zeewering in 1955 provides an example of this.

Along rivers the effects of storms are less serious. However if a vessel loses steerage, there is a chance that it will strike a dike. The amount of damage is determined by the

velocity of the vessel and by the angle at which it vessel strikes the dike. However, in the event of dangerously high water all shipping movements will have been stopped.

The managers themselves will have to ascertain whether the potential effect of the combination of a shipping collision with particular hydraulic conditions should be one of the design criteria, in which case specialist assistance will be required. A risk analysis of this type was performed for the design of the storm surge barrier in the Nieuwe Waterweg. Information on this is available from the Construction Service of the Directorate-General for Public Works and Water Management, Head Offices Hydraulic Construction in Utrecht.

Floating debris or driftwood is generally too small to cause any serious damage to the revetment, although it may damage grass cover.

3.4.8 Biological damage

Water plants and animals can attach themselves onto and between the constituents of the revetment, but usually cause little damage. However the physical characteristics may change; the permeability of open revetments may decrease for example. Plant growth on a smooth revetment may increase roughness, as a result of which loading by waves and currents may increase. Bituminous revetments are slightly sensitive to damage by sheep and cattle faeces. Animals that dig holes, such as rats, muskrats and moles may undermine a slope or damage a covering clay revetment by digging through it. The latter may result in a higher phreatic line in the embankment. It is also possible that sand may flow out from the core of the structure. Susceptibility to the undermining activities of the muskrat is particularly prevalent in waterways located along the dikes, such as scour holes and watercourses, and in main dikes. Ineffectively planned pasturing may cause damage to the grass slopes.

3.4.9 Chemical damage

Construction materials may be damaged by oxidation, salt water and materials found in contaminated water. Bituminous revetments are sensitive to chemical damage, while grass and geotextiles are sensitive to damage by chemicals. Attention must be paid to combinations of loading. For example, a concrete revetment may be damaged by erosion, as a result of which any reinforcement is exposed to oxygen, and water and the contaminants they contain, resulting in corrosion. This is also true for cables in block mats.

3.4.10 Damage resulting from climate

Enclosed water will expand when frozen and can thereby damage construction material. Long-term drought can dry out a clay layer that is too thin to such an extent that the grass covering the slope is seriously damaged. Shrinkage tears occur when clay dries out, reducing its resistance to erosion. The formation of these structures results in a difference between the erosion resistance of clay layers on north and south sides (even when they are covered by stone) due to variations in temperature and moisture content.

Geo-textiles, artificial screens and other man made materials must be protected from ultraviolet radiation.

3.4.11 Vandalism and shared use

A revetment must be as resistant as possible to recreational use. Anglers occasionally forget to replace stones that they have removed from a revetment. Asphalt revetments have only poor resistance to fires. In the vicinity of residential areas and also elsewhere, loose stones are often removed by children or by the owners of garden. For easily accessible locations, it is therefore recommended that when determining the weight of a revetment the use stones that are too heavy for people to carry should be one of the design criteria. Near places where there are steps in the slope and at other locations where many people climb up and down the dike, bare patches develop in the grass cover. Bicycle wheel ramps or channels, soil filled cellular cement blocks or local revetments can be used to counter this.

4. Soil-mechanical and geo-hydrological aspects

4.1 Introduction

The substratum under a dike section is an important parameter in dike design. In order to gain a useful and reliable image of the substratum, both geological and geo-technical knowledge is required. Pore pressure in the dike body and substratum are important loading factors on the water defence so insight into the groundwater flow and consolidation conditions is required. To obtain an effective picture of this it is essential to have insight into the bedding of the substratum. Figure 4.1 describes the various steps that must be followed in dike design, from collecting data to assessing the relevant failure mechanisms (with reference to the relevant sections).



Figure 4.1.1 Data and calculation models soil mechanics / geo-hydrology

The eventual order and the level of detail followed in taking the steps in Figure 4.1 depends on the project phasing and the optimization to be employed (see Chapter 6). Usually each step will first be worked out roughly (but conservatively), and subsequently completed at the level of detail desired.

4.2 Geological description of the area

4.2.1 Introduction

A geological description of the area comprises a global investigation of the bedding and the properties of the substratum. Initially, the geological description is based on existing data, including geological maps, groundwater maps and archive research. Several publications

cover the geological history of the Netherlands, including the composition the various geological series and their geo-technical properties. Moreover, it is assumed that those reading this report will already have some knowledge of geology. This section provides only the geological information that may be of importance in the analysis of the soil layers and the interpretation of the soil analyses. For further information, reference should be made to the literature and to geological, geo-morphological and geo-hydrological maps.

4.2.2 Objective of the geological description of the area

The objective of the geological description of the area is to estimate:

- The geo-hydrological conditions, in particular the pore pressure expected in water-carrying strata during high water levels
- The failure mechanisms that are important for the dike design
- The geo-hydrological and geo-technical zone of influence of the water defence (width and depth)
- The required pore pressure measurements and soil analysis

On the basis of geological knowledge and experience in the area, an assessment can be made of which geological formations may still contain channels, layer interruptions and similar features the presence of which cannot be excluded with sufficient reliability on the basis of the available data. If this could be important to the dike design, in many cases it may be verified by means of geo-physical investigation at an early stage. Missing a water-carrying channel in the substratum of an old dune (Pleistocene dune) would constitute a serious error that might have serious consequences.

The categorization of the substratum into various series occurs on the basis of lithostratigraphy. The categories are based on the composition of the layers and order of deposition, and on the basis of geo-technical properties such as cone resistance (see § 4.3.3) and mass by unit volume (see § 4.3.2). This categorization gives series that are so constant in their geo-technical properties that they can directly serve as geometric models, for example, in settlement calculations and stability investigations.

4.2.3 Geology Upper River area

In the Upper River area, the substratum is composed of Pleistocene and Holocene sediments. The composition of the substratum in the river area may vary considerably, even over short distances.

Pleistocene

The upper strata of the Pleistocene consist of either fluvial sediments or, outside the former river valleys, of windblown cover sands deposited during in the last ice age. A general impression of the location of the river valleys can be found in Figure 4.2. The cover sand areas are located on both sides of the river valleys. They are generally overlain by the Holocene settlements, except in the easterly and south-easterly provinces, where they surface locally.

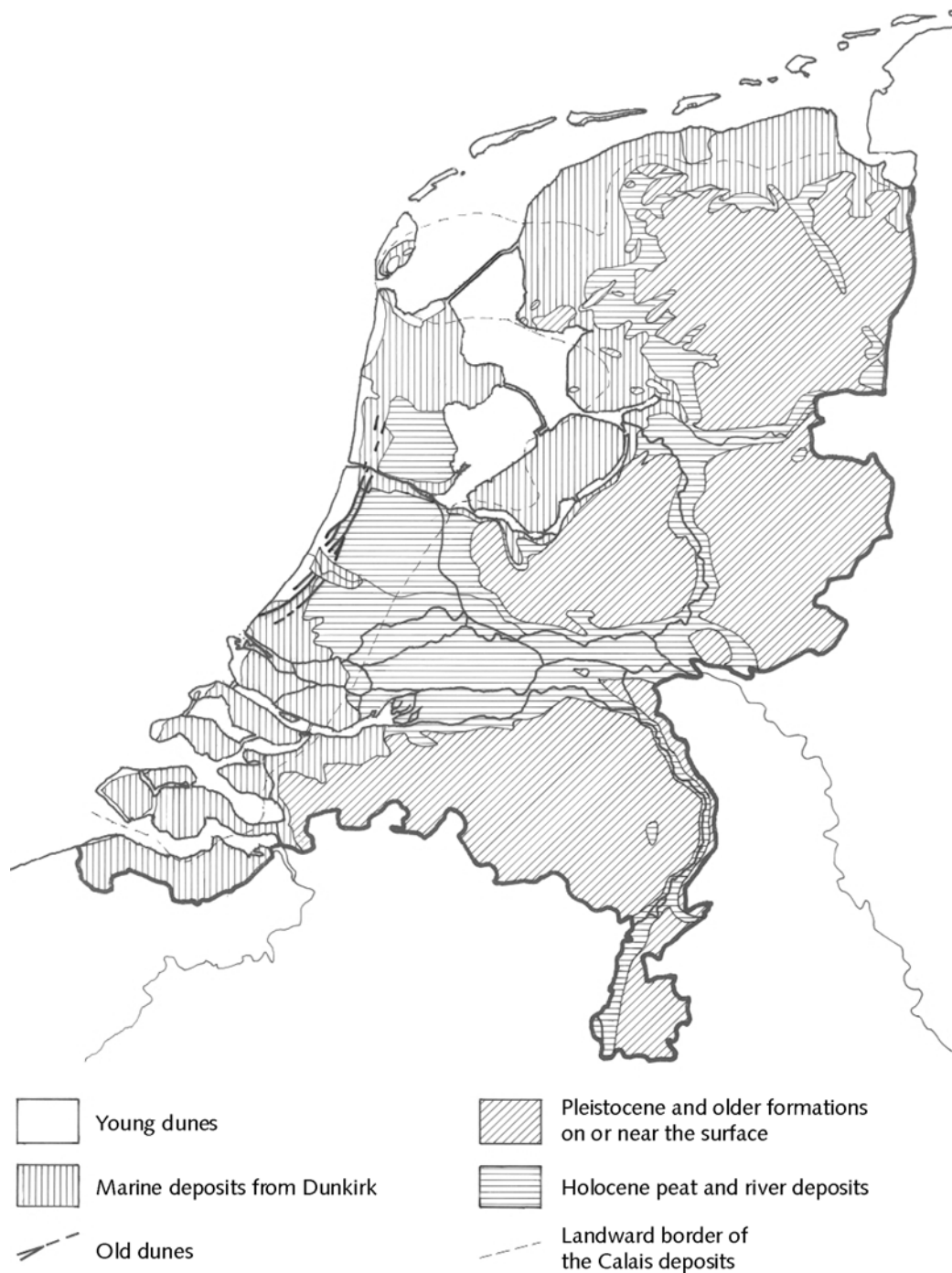


Figure 4.2.1 Geological map of the Netherlands

□

Holocene

During the Holocene, the major rivers deposited a wide variety of sediments. These are classified according to their origin as belt, levee/bank and basin settlements. The first of these were deposited in the areas where the rivers actively flowed: on the area flood plain where the meandering courses of the rivers constantly changed as bends were cut off and new channels were created. The bank deposits border the flow belt deposits; they were deposited in the area where when the river burst its banks the water still had sufficient force to carry the sand along, and subsequently deposit it under falling flow speeds. They represent the transition between the flow belt and basin deposits that were formed where the water

ceased to flow so that the silt sank. The basin deposits consist primarily of clay. Locally, peat or peaty clay can be found at the base of the basin clay.

The geological picture sketched above indicates that the Holocene deposits in the river beds are frequently intersected or interrupted by Holocene river channels that were choked by sand or silt, as a result of which the composition of the geological structure and bedding may vary considerably from place to place.

4.2.4 Geology of the Lower River area

In the Lower River area, there are two geological formations the Pleistocene: Kreftenheye formation and the Holocene: Westland formation. Large variations in soil structure may occur over short distances.

Pleistocene: Kreftenheye formation

The Kreftenheye formation, formed in the late Pleistocene, consists of:

1. *Fluvial deposits*
The fluvial deposits, which represent the main element in this Pleistocene formation, were deposited by rivers and occur throughout the entire area. The top of these Pleistocene deposits, which consist primarily of medium to coarse-grained sands, mixed locally with gravel, lies between –10 m and –23 m below sea level (NAP). These river deposits are increasingly fine at the top, and are usually sealed by several decimetres of a thick, silty and fine-grained sand or loam layer.
2. *Eolian deposits*
Eolian sands, which were deposited by the wind, formed the Pleistocene dunes, known as *donken*. The sand is fine to medium-grained and originates from the dry riverbeds of that period. These Pleistocene dunes were deposited at the same time as, or just after the fluvial deposits. They are irregularly spread across the entire area and vary greatly in thickness from a few decimetres to 5 m to 10 m and may be exposed locally.

Holocene: Westland formation

Some 10,000 years ago, temperatures began to rise, and the last ice age came to an end. The warm period that followed is called the Holocene. As a result of the increase in temperature, the ice caps to the north melted and the sea level rose. All the sediments deposited as a result of this sea level rise are part of the Westland formation. The Westland formation comprises four types of deposit, each with its own lithology. These include:

1. *Old and young dune and beach deposits*
The dune and beach deposits consist of fine to medium-grained sands. Dune sand is beach sand that was blown onshore and was deposited close to the sea. These deposits occur only in the west near to the coast.
2. *Marine deposits*
Marine deposits were formed under the direct influence of the sea. The older deposits are called the Calais series. The lower parts of these consist primarily of sand, while the upper strata are mainly clays. The younger strata are known as Dunkirk deposits. In the channels, the Dunkirk deposits consist mainly of sand or sandy clay, but outside the channel systems clay predominates.
3. *Perimarine deposits*
These are deposits fluvial deposits laid down under the influence of rises in sea level. The older sediments are called the Gorkum deposits, and the younger sediments are the Tiel deposits. Both the Gorkum and the Tiel deposits consist of clay, peaty clay and, particularly in bigger channels, layers of sand.
4. *Organic deposits*
Two different layers of peat are distinguished in the Westland formation: *basal peat* and *Dutch peat*. The *basal peat* only includes the layer of peat that occurs up to the base of the Westland formation. The other layers of peat are termed *Dutch peat*. In

the maritime area, Dutch peat represents the division between the Calais deposits and the Dunkirk deposits. Both the basal and the Dutch peat have nutrient-rich and nutrient-poor varieties.

4.2.5 Coastal area geology

The coastal area of the Netherlands can be divided into three physical geographic areas: the south-western sea clay area, the dune area and the Friesland-Groningen sea clay area. There are two important geological units in this area: the Pleistocene Twente formation and the Holocene Westland formation.

Pleistocene: Twente formation

The Pleistocene period is characterized by alternations between cold (glacial) and warm (interglacial) climates. The last cold period preceding the Holocene is called the Weichselien. The Twente formation was deposited in this period. There was no land ice in the Netherlands at that time, although it was cold enough for the ground to remain frozen for long periods each year. The Netherlands was covered by sparse tundra vegetation, consisting of low, bushes and grasses. This vegetation gives only little protection against the strong winds, which carry away sand and deposit it elsewhere. With the exception only of the river area, a thin cover of eolian sand was deposited across the whole of the Netherlands giving rise to the name 'dekzand' (cover sand), which is also applied to the Twente formation.

The eolian sands are fine to medium-grained, and include small local layers of loam and lines of gravel. Phenomena resulting from frost impact frequently occur. The cover sands blanket the entire area and vary greatly in thickness, from one metre up to around ten metres.

Holocene: Westland formation in the south-western sea clay area

The erosion and sedimentation patterns of south-western sea clay area are very different from those of the Friesland-Groningen sea clay area and the dune area. One important reason for this is the presence of large river estuaries while, in addition, the tidal range in the south-west of the Netherlands is greater as a result back up effects caused by the Coriolis force and of the flows in the Calais narrows. In the south-western sea clay area, the Westland formation has four types of deposit, each with its own lithology. These include:

1. *Young dune and beach deposits.*

For a description, reference may be made to the description of the Old and Young dune and beach sediments associated with the Westland formation for the Lower Rivers. In the south-westerly sea-clay area, these deposits only occur near the coast.

2. *Marine deposits*

For a description of these, reference may be made to the description of marine deposits associated with the Westland formation for the Lower Rivers. In the south-western sea clay area, the Dunkirk deposits have developed locally as shoal deposits.

3. *Perimarine deposits*

Perimarine deposits are river deposits set down under the influence of a rise in sea level. Only the older deposits can be found in the south-western sea clay area. These deposits are called the Gorkum deposits, and consist of clay with thin layers of peat. The thickness of these layers varies between 1 and 2 metres.

4. *Organic deposits*

For a description of these, reference may be made to the description of the organic deposits associated with the Westland formation for the Lower Rivers.

Holocene: Westland formation in the dune area

The dune area comprises the coastal strip to the west of the line running through Groet-Alkmaar-Velsen-Haarlem-Leiden-Voorburg-Monster. The dune area can be divided into two morphological units, namely the old dunes and the young dunes. Beach banks were formed

at the beginning of the Holocene period. Locally, river estuaries interrupted the beach bank complex. The old dunes were formed on the beach banks while the young dunes were formed in the early Middle Ages. Marine sediments were laid down between the beach banks, allowing peat to develop. The Westland formation includes three types of deposit in the dune area, each with its own lithology. These include:

1. *Old dune, Young dune and beach deposits*
For a description of these reference may be made to the description of the Old and Young dune and beach deposits, associated with the Westland formation for the Lower Rivers.
2. *Marine deposits*
For a description of these, reference may be made to the description of marine deposits, associated with the Westland formation for the Lower Rivers. In the dune area, the Dunkirk deposits consist primarily of a clay deposit of one to several metres in thickness. Sands of the Dunkirk deposits are only found in those locations where the beach bank complex is interrupted by river estuaries.
3. *Organic deposits*
For a description of these reference may be made to the description of organic deposits, associated with the Westland formation for the Lower Rivers. In the dune area of the Westland formation the basal peat is no more than 1 metre thick.
□

Holocene: Westland Formation in the Friesland-Groningen sea clay area

In the Friesland-Groningen sea clay area the Pleistocene is relatively shallow, and dips away towards the north. During the Holocene marine deposits were laid down on the Pleistocene substratum, and coastal peats were formed. In the Friesland-Groningen sea clay area the Westland formation includes two types of deposit, each with its own lithology. These are:

- 1 *Marine deposits*
For a description of these, reference may be made to the description of marine deposits, associated with the Westland formation for the Lower Rivers.
- 2 *Organic deposits*
For a description of these, reference may be made to the description of organic deposits, associated with the Westland formation for the Lower Rivers.

4.3 Soil behaviour and soil parameters

4.3.1 Introduction

The most prevalent types of soil in the Netherlands are sand, clay and peat, and a wide range of mixtures of these. Soil consists of three components: solid particles (grains), water and air. The water and air fill the pores between the particles. Besides the properties of the solid components, the proportion of these components influences the behaviour of the soil. The soil properties may vary within a wide range. There are major differences in:

- The nature of the grains, or soil particles
- The size and shape of the grains, and the grain distribution
- The manner of deposit and their origin
- The history, especially in relation to previous loading

Sand is characterized by a loose, non-cohesive grain structure, good permeability and a relatively large mass per unit volume. Clay is a cohesive type of soil, which is not usually very permeable and is resistant to erosion. If structural formation has occurred through ripening, however, the clay may achieve a high degree of permeability. The weight by unit volume (kN/m³), or the mass by unit volume (kg/m³), which is also often used, depends strongly on the organic component content and on the degree of consolidation. Peat is very cohesive, and weakly to moderately permeable type of soil, which has a mass by unit volume that is

lower than that of all other soils. Young peat deposits may possess very poor friction properties and be strongly compressible. If the peat has a well-developed fibre structure, the friction properties may be higher, although there is a difference between friction the direction of the fibres and at right angles to them (anisotropy).

This subsection describes the behaviour of the soil and the soil parameters used to describe this behaviour. Water-retaining soil structures can be placed into categories on the basis of the most important elements of soil behaviour are considered. These are:

- Mass by unit volume (§ 4.3.2)
- Shearing behaviour (§ 4.3.3 up to and including § 4.3.7)
- Deformation and settlement (§ 4.3.8)
- Permeability (§ 4.3.9)

Shearing behaviour can be described under drained or undrained conditions. When modelling, a distinction can also be made between isotropic and anisotropic material behaviour. In addition to a description of the rigid plastic behaviour of soil, it is also possible to include the deformation in the description of the soil behaviour. For both one-dimensional and the multi-dimensional approaches, the deformation and settlement behaviour of soil can also be investigated by using a finite elements method.

The structure of the substratum and properties of the material can be determined by using laboratory and in situ tests (§ 4.4). In § 4.5 and § 4.6 attention is paid to pore pressure and hydraulic heads in the groundwater. The role of this data in stability and deformation calculations is covered in § 5.2 and § 5.4.

4.3.2 Mass by unit volume

The natural weight by unit volume of soil is an important parameter:

- In a stability analysis, the weight by unit volume of the soil on the active side is an activating force, while on the passive side this is counteractive force;
- The weight by unit volume of soil in part determines the effective stress on the slip plane and is therefore also a determinant when assessing the magnitude of the shear resistance to be mobilized;
- The weight by unit volume of the compressible soil layers on the inside of the dike determines whether or not uplift or heave occurs. This determines the type of stability analysis that must be used;
- The weight by unit volume of the soil is co-determinant in whether heave occurs, and therefore whether piping can play a role (see § 5.6).
- Applying soil does leads not only to an increase in soil pressure in the substratum, but also to settlement and horizontal deformation
- A weight by unit volume of a type of soil is an indication of consistency. Peat with a weight by unit volume of 10 kN/m³ is softer and more easily compressed than peat with a weight by unit volume of 12 kN/m³.

In soil analysis, attention must therefore always be paid to determining the weight by unit volume at sufficient points. For sand above groundwater level, it is clear that a different weight by unit volume must be used (γ_{wet}) than for sand that is under water. For clay, this difference is less clear. This is because there may be a great deal of water in the clay above the ground watertable as a result of capillary pressure.

Each time the substratum is drilled into the weight by unit volume is often determined by using the Pycnometer (Ref. NEN 5111:1992). More information on the definitions of the above-mentioned parameters can be found in the CUR 162 handbook (Ref. *Constructing with soil*).

4.3.3 Description of soil during shearing

There are three ways to analyze soil behaviour in shearing (rigid plastic). These include:

1. Drained pressure analysis

2. Undrained total pressure analysis
3. Undrained effective pressure analysis on the basis of measured or predicted pore pressure

□

Drained pressure analysis

In current Dutch design practice, a drained analysis using effective strength parameters c' (effective cohesion) and ϕ' (effective angle of internal friction) are frequently used to assess the final stability of a soil structure. The values of c' and ϕ' are determined by making triaxial tests on core soil samples taken. In addition to triaxial tests, cell tests are also used to determine the parameters mentioned above and great deal of practical experience of their use has been gained under Dutch conditions. The results of the cell tests are often kept in a regional test collection and remain valid for the triaxial test (Ref.) *Statistical processing of cell and triaxial tests*). Subsection 5.3.3 explains how to deal with the differences between the results of cell tests and triaxial tests. If they are available, the arithmetic average values for c' and ϕ' in the test collection are used as shear strength parameters. If there is no test collection, the values collected during local testing are used. The characteristic shear strength is then determined by using another method (see § 4.3.5).

This Technical Report proposes that the effective strength parameters for c' and ϕ' should no longer be determined via a cell test, but instead by using a CU triaxial test according to NEN 5117. Reasons to change to use of the triaxial test include:

1. The values determined for c' and ϕ' using the cell test are measured at a lower level of deformation and deformation speed than in the triaxial test, in which loading up to collapse occurs. The cell test will therefore usually give lower values for c' and ϕ' (Ref.) *Statistical processing of cell and triaxial tests*).
2. Assuming a situation that is frequently encountered in practice, in which the structure is loaded over a relatively short period of time (undrained application of loading), the pressure line followed in the cell test differs from that in the soil structure. In an undrained triaxial test, the pressure line agrees more closely with practical conditions (active zone of slip plane).
3. Agreement with international methods is desirable.
4. The triaxial test is more reproducible.

One consequence of the use of c' and ϕ' from the triaxial test is that a new safety approach will have to be set, with associated safety factor SF. In the year 2000, this changeover was still the subject of ongoing discussions.

Undrained total pressure analysis

Undrained analysis is used in situations with poorly permeable soil, where water overpressure is generated as a result of changes in loading. Total pressure analysis implies that the distribution of effective stress and water overpressure is not taken into account. Subsections 4.4.5 and 4.4.12 describe how to determine the undrained shear strength.

□

Undrained effective pressure analysis

In an undrained effective pressure analysis, the of drained strength properties c' and ϕ' are used on the basis of predicted or measured water overpressures. The water overpressures can be introduced 1:1 or by means of adaptation percentages that indicate the degree of consolidation (see § 4.3.8). Effective pressure, or safety factor SF, can be deduced from the total pressure and the water overpressure.

The use of this method has the drawback that it is difficult to determine the correct pressure or shear stress when an increased load is applied. However, in practice the method is often successfully used by consulting engineers during the implementation phase.

4.3.4 Behaviour and strength in shearing

A typical characteristic of soil is that in the event of increasing shear stress, soil can deform to such a degree that collapse occurs. In this context, Coulomb defined what he termed 'critical shear stress' (Ref. *Construction in soil*).

[EQUATION] (4.3.1)

$$\tau'_{kr} = c' + \sigma'_n \tan \phi' \quad (4.3.1)$$

In which:

- c' cohesion
- ϕ' angle of internal friction
- σ'_n effective stress perpendicular to the surface in question
- τ'_{cr} critical shear stress

As long as the shear stress on a particular surface is smaller than the critical value, the deformations are limited. However, if the shear stress reaches a critical value on that surface, deformation is unlimited and collapse will occur. A point on Mohr's circle represents normal stress and the shear stress on a particular surface. On certain surfaces, shear stress can be so large that Coulomb's collapse conditions are met (see Figure 4.3.1). In two shear surfaces AC and AD, the shear stress is critical at points C and D; on all other surfaces, shear stress is lower than the critical value. This is known as the Mohr-Coulomb collapse criterion.

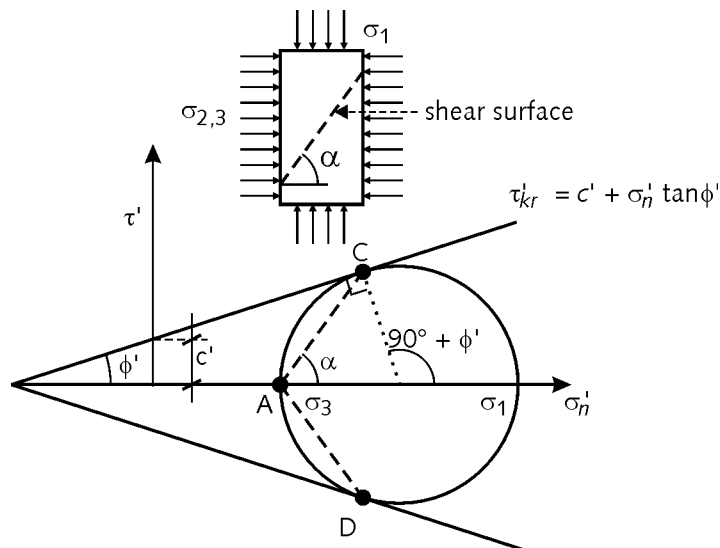


Figure 4.3.1 The Mohr-Coulomb collapse criterion

It should be noted that the strength of the soil is dependent on effective stress. The linear connection found by Coulomb between normal stress and critical shear stress is effective for loose-grained materials, such as sand and for normal consolidated clay. However, in clay with very poor bearing capacity and extremely high water content and in peat, such great deviations from the linear model may occur that for certain normal stress directions, combinations of c' and ϕ' will have to be determined.

In more solid soil layers, the maximum shear strength is reached with far less deformation than in relatively weak layers such as clay and peat. Amongst other factors, the shear strength is dependent on deformation. The acting shear stress with respect to critical shear stress of the material in question is shown in Figure 4.3.2 against angular displacement (horizontal deformation).

In practice, the soil is generally wholly or partially saturated with water. A distinction is usually made between drained shear strength and undrained shear strength. Drained shear strength affects the effective stress in the soil and, in principle, can be used in calculating the stability for any pore pressure situation.

[EQUATION] (4.3.2)

$$\tau'_m = c' + \sigma' \tan \phi' \quad \text{met} \quad \sigma' = \sigma - u \quad (4.3.2)$$

In which:

c' drained or effective cohesion
 u water pressure
 ϕ' drained or effective angle of internal friction
 σ undrained effective stress
 σ' drained effective stress
 τ'_m drained shear capacity

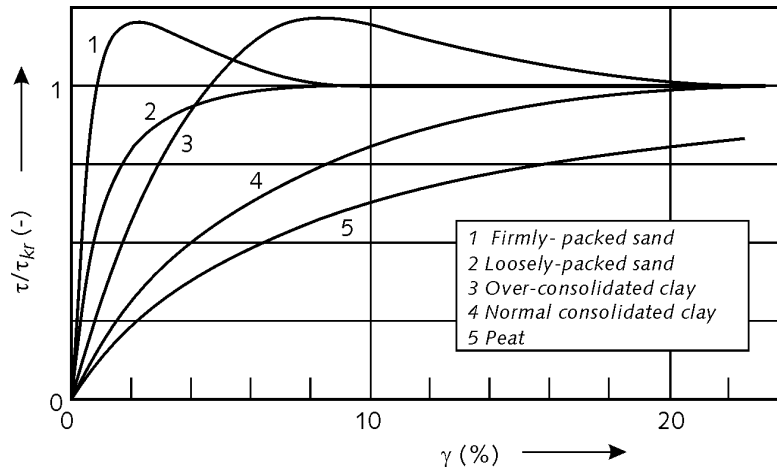


Figure 4.3.2 Pressure deformation behaviour of various soil types

4.3.5 Shear strength parameters

The friction properties of soil, expressed by the drained shear parameters c' and ϕ' or the undrained shear strength f_{undr} can be determined in several ways. In practice, the term C_u is also used to indicate undrained shear strength. The tests used to determine this can basically be split into two groups:

- Test with directly applied shear deformation on samples, such as the vane test in the laboratory, the hand vane test (§ 4.4.5) and the direct shear test and the DSS-test (see § 4.4.13 and Annexes 7 and 8);
- Pressure tests on cylindrical samples, such as the triaxial test and the outdated cell test.

It is necessary to take a number of samples from each soil layer. If the number is sufficiently large, a test collection can be formed. The larger the number, the more accurate will be the normative parameter in the stability calculation (the characteristic value of the local average shear strength), because the standard deviation decreases. In practice, it turns out that little gain can be achieved by using more than 8 to 10 samples. This must be taken into account when setting the scope of the tests.

Determining shear strength from laboratory tests

These tests often produce clearly different friction properties for the same soil. To a large extent, the differences are the result of the setting-up of the testing apparatus and the test and the way in which the test is carried out. Triaxial tests are the most common one used by engineering consultants. The values of c' and ϕ' determined with a cell test are measured at lower deformation rate than in the triaxial test, where loading is applied until collapse. As a result, the cell test usually gives lower values for the strength parameters (see also § 4.3.8).

In the Netherlands, stability is usually calculated on the basis of effective grain stress and drained strength properties. When implementing the tests it is necessary to know the consolidation behaviour of the soil (see § 4.3.8). In situations where stability has to be assessed directly after a change in loading, it may be useful to use the total soil pressures and undrained strength properties of the soil to make an f_{undr} analysis.

The undrained shear strength f_{undr} is the strength that applies in undrained loading situations and is dependent on the effective pressure before the application of the loading. In such cases it is advisable to determine the f_{undr} value on the basis of ϕ' and c' with the aid of the following formulation $f_{undr} = c' \cos \phi' + 0.5(\sigma_1 + \sigma_3) \sin \phi'$. This corresponds to the radius of Mohr's circle. (Figure 4.3.1). The normal pressures σ_1 and σ_3 are, respectively, the horizontal and vertical primary pressures. The f_{undr} value can also be determined from field measurements, such as vane tests or penetration tests. In the same way, the undrained strength parameter can be deduced from empirical relationships. A relationship that is often applied internationally is the relation between the quotient of the effective stress (often the vertical grain stress) and the undrained shear strength and the plasticity index (Ref. *Construction in Soil, Annex A2*).

Characteristic estimates of shear strength

In the Dutch norms and guidelines, which are in accordance with the Eurocodes, a semi-probabilistic set-up of the arithmetic analyses is advocated. As a result, first the 'characteristic values' for loading and strength parameters must be set. If loading is involved, these are multiplied by partial safety factors (the loading factors). From this the design or arithmetic values to be used in the analysis are found. For more information on the philosophy underlying this method is given in (Ref.) *Fundamentals of water retaining structures*.

A geo-technical analysis therefore assumes the 'characteristic values' of the soil parameters, or formulated a little more accurately, the characteristic estimates of the average values of the soil properties within the soil layers at the location where the analysis is performed. A characteristic estimate is one that is unlikely to be reached (for strength parameters) or exceeded (for loading) by the actual value at the site. In Dutch regulations and guidelines such as Geo-technical Standard (Ref.) NEN 6740) or the TAW Guidelines, a cumulative probability less than 5% (deficiency) or exceedence probability is generally used. The term 'actual value of soil properties present' demands some additional explanation. Every soil characteristic varies from place to place. An example is given in Figure 4.3.3.

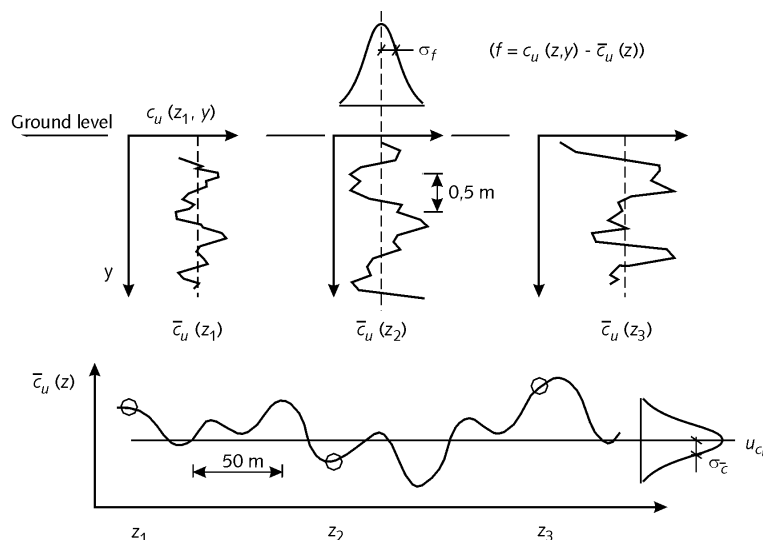


Figure 4.3.3

Possible variation pattern for undrained shear strength in a weak clay layer (note: in this figure, the undrained shear strength is represented by the international figure c_u instead of the national symbol f_{undr}).

When measuring a soil characteristic, the value that is determined is representative for the volume of the soil that is tested in measurement. For example, in a triaxial test, this relates to a soil sample of no more than several hundred cm³. For in situ measurements, such as penetration tests, the value relates to areas of influence in the soil no larger than some decimetres. In geo-technical analyses, the value relates to slip planes (in balance analyses) or particular soil volumes (in deformation analyses) measuring up to tens of metres. The measurements of the soil volume in question in laboratory or in situ tests are therefore small, in comparison to the measurements of slip planes or soil volumes in deformation tests. The measured values of soil properties are therefore indicated as 'point values'. Within slip planes, and similar features, point values of soil properties may vary. In equilibrium and deformation analyses, the averages of the soil properties within the slip planes or soil volumes in question are important. In the subsection above, 'actual value present' means the average value of the soil characteristic within the slip plane or volume that is present. Variations in point values are therefore more or less averaged out.

In Dutch geo-technology, this idea has led to the use of characteristic estimates of the average from the test series (within a soil layer) when determining soil properties on the basis of laboratory or in situ tests. In fact, these are characteristic estimates of the average of the soil layer within the area of investigation (the geographical area from which the samples are taken or in which the in situ tests are performed). It is necessary to consider to what degree averages within such an area of investigation are representative of a slip plane or a particular volume of soil in deformation analyses. As long as the area of investigation more or less corresponds with the area in which the slip plane or the volume of soil in question is situated, this can be considered reasonable. This is a case of local soil investigation. However, if the area of investigation is much larger, as in regional test collections, it is necessary to take into account fact that the averages over slip planes or soil volumes may vary considerably from the overall average within the entire area of investigation. The average of the test collection is not representative for the average along a slip plane. When setting characteristic values for geo-technical analyses on the basis of regional test collections, it is necessary to take into account the distribution of averages along a slip plane within the area of investigation. The method therefore deviates from the method used to determine characteristic values from local soil analysis.

Annex 1 describes the method used to determine characteristic values from a test sample. To ensure compatibility with the Annex, the text of this last part of § 4.3.5 is almost entirely repeated as an introduction to Annex 1. The Annex focuses on shear strength parameters, although the methodology can be used for other soil mechanical parameters.

The distribution of test results for a series of soil samples is generally ascribed to spatial variation in soil properties (within a specific soil layer). Although the possibility that the inability to reproduce soil mechanical tests is partly responsible for the distribution in test results should not be excluded, in the following subsection it is assumed that the contribution made by spatial variation is dominant.

For full understanding of the methodology, a picture of the underlying (generic) conceptual model for spatial distribution is desirable. This model is therefore covered in Annex 1 directly after the introduction. Following this techniques for determining characteristic estimates and the use of regional test collections are covered.

4.3.6 Anisotropy

The soil behaviour described in § 4.3.5 assumes that the strength of the soil is the same in the active, neutral and passive zones (see Annex 2). This is not entirely correct: in compression (increase average pressure) the strength of the soil differs from that in extension (decrease in average pressure). Shear strength may also vary in the direction of the slip plane. This behaviour of material is known as anisotropy. In other countries, where anisotropy is being taken into account this is proving advantageous, but in the Netherlands, in the year 2000, the value of this approach could not be precisely quantified.

Clay displays anisotropic properties. In practice, for the time being this anisotropic behaviour can probably be ignored. However in peat this behaviour is dominant (Ref.

Construction in Soil, Annex E4). The anisotropy of peat results from its fibrous structure. Under loading, the as the fibres are pressed anisotropic behaviour increases even further in the preferred direction.

The important question lies in how far the value of these refinements is eliminated by spatial variation. The anisotropic behaviour of peat probably means that strengths c' and ϕ' are direction-dependent. When the Bishop method was used in stability calculations for earth structures on peat an unexpectedly low equilibrium factor was found in a number of cases. This low equilibrium factor is probably caused by the use of shear strength parameters derived only from triaxial tests that are consequently seen as representative of the active section of the slip plane, regardless of the direction of local loading. In the neutral zone, and possibly also in the passive zone of the slip plane, strengths that can be measured by using simple shear or direct shear tests, or triaxial extension tests, are assumed.

One of the internationally recognized methods for taking anisotropy into account is the ADP method (Active, Direct, Passive). In order to be able to use the anisotropy method, active, horizontal and passive collapse zones are distinguished. The ADP method is an undrained stability analysis to determine the strength of soil structures. The method is used frequently in Scandinavia and in English-speaking countries. In these countries, stability is frequently an issue in excavations, and the method functions effectively in this situation. The ADP method is described in detail as Annex 2.

4.3.7 Elasto-plastic behaviour of the Soil

□

For clay and peat, the friction properties and deformation rate are interdependent. When deformation rates are rapid the soil generally has greater friction properties than it does when deformation rates are low. In long-term loading, a deformation criterion may be normative in determining which shear stress should be seen as the maximum permissible for a particular effective stress. The collapse analysis can be described by using the Finite Elements Method (FEM) and is recommended even in complex situations. With this method, a direct link can be laid between stability and deformation, without the need to designate a collapse mechanism. In deformation calculations of major deformations, the *Soft Soil Creep* model in the Plaxis model can be applied.

□

Elasto-plastic model according to Mohr-Coulomb

If it is necessary to investigate only the collapse of a soil structure and deformation is of little or no importance, an FEM analysis and the use of the Mohr-Coulomb model will suffice. Various elastic constants are used in calculations. In practice the elasticity modulus (Young's modulus) is often applied. This modulus represents the link between corresponding pressure and deformation using the transverse contraction coefficient (Poisson variable) and the slip modulus. Elastic deformation occurs immediately following the application of the load and is reversed immediately after removal of the load. In cases of relatively high loading, the grains in the grain skeleton will shift with respect to each other. When this phenomenon occurs, the deformation may not be reversed when the load is removed. The process is irreversible and is termed *plastic deformation*. In these cases, the Mohr-Coulomb model does not provide an effective solution.

Elasto-plastic model according to Soft Soil Creep Method

If deformation is an important criterion, the Mohr-Coulomb model is inadequate and the Soft Soil Creep model in Plaxis offers good prospects. If one-dimensional deformation is under consideration simpler methods can be used. Further details can be found in § 4.3.8.

□

4.3.8 Deformations and settlement

When building a dike, settlement as a result of soil compressibility should be taken into account. In this context, a supplementary height allowance should added so that the dike will come to rest at the planned height after settlement. In addition, it is essential to know the anticipated settlement when calculating the amount of soil to be moved.

Primary settlement

In compression tests in which sideways deformation is prevented, Terzaghi found a linear relationship between compression and the logarithm of the load.

[EQUATION] (4.3.3)

$$\frac{\Delta h_p}{h} = \frac{C_c}{1 + e_0} \log \frac{\sigma'}{\sigma'_0} \quad \text{met} \quad \sigma' = \sigma'_0 + \Delta \sigma' \quad (4.3.3)$$

In which:

C_c	primary compression index [-]
e_0	pore number at initial effective stress [-]
h	thickness of the soil layer [m]
Δh_p	primary compression of the soil layer [m]
σ'	grain pressure (or effective stress) [gPa]
σ'_0	initial effective stress [gPa]
$\Delta \sigma'$	increase in effective stress [gPa]

Equation (4.3.3) gives the primary compression that occurs when applying an increase in loading to soil. In dry ground, this settlement occurs almost immediately. In soil that is saturated by water, this settlement is only reached once the grain skeleton has assumed the entire load. This situation is reached when, as a result of the expulsion of all the pore water under simultaneous compression of the soil, there is no longer any pore pressure. This phenomenon is called consolidation.

Soil loaded at an earlier stage generally acts in a stiffer manner. Compressibility of soil in the pressure zone, up to the pre-consolidation pressure (the maximum pressure by which the soil was loaded previously), can be represented by swell constant C_s . Equation 4.3.3 is valid for relatively small deformations (no more than 20%). In the event of larger deformations, equation 4.3.3 may be used if the primary compression index C_c that will occur in practice has been determined for the loading area. Another possible solution is to continuously adapt layer thickness and pore number after a particular deformation (Figure 4.3.4) (see also CUR 162, (Ref.) *Construction in Soil; Annex A3*).

Secondary settlement

Once the consolidation phase or the hydrodynamic period has ended, the relationship between compression and the logarithm of time is linear. Bjerrum presented the entire concept of the Anglo-Saxon method. This is as follows (Ref. *Construction in Soil*):

[EQUATION] (4.3.4)

$$\frac{\Delta h_s}{h} = C_{\alpha} \log \frac{\Delta t}{\Delta t_d} \quad (4.3.4)$$

In which:

Δh_s	secondary compression of the soil layer [m]
C_{α}	secondary compression index [-]
Δt	duration of the loading [day]
Δt_d	1-day duration

In highly compressible soils such as clay and peat, in which changes in volume can be very large, Butterfield and Den Haan proved that the Anglo-Saxon method can also be used for these deformations. In this case, a double logarithmic scale is used. (Ref. *Constructing in Soil, Annex E3*). The advantage of this expansion is that finite settlement is calculated and not an excessive, unrealistic settlement.

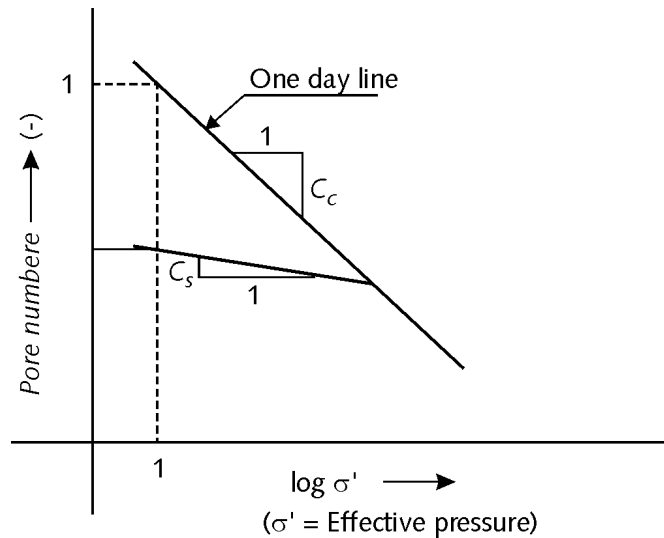


Figure 4.3.4 Relationship between porosity and pressure

In Dutch engineering consultancy practice a number of calculation programs are available for calculating the settlement behaviour of earth structures. The *Msettle* program can be used to calculate one-dimensional settlement behaviour. The intention of the program is to use both the method described in NEN 6744 and the Anglo-Saxon method. In *Msettle*, the Anglo-Saxon method is implemented in such a way that, in practice, primary compression constants C_c and C_s and the secondary compression constant C_{α} as determined in a compression test according to NEN 5118 can be used.

Final settlement

The final settlement, or total settlement (see Figure 4.3.5), is calculated by summation of primary and secondary settlement. In the Netherlands, it is assumed that the compression is completed after an arbitrarily chosen period of 10^4 days (about 30 years).

t	time (day)
Z_i	total settlement (m)
$Z_p(\sigma')$	primary settlement according to NEN 6744
$Z_s(t)$	secondary settlement according to NEN 6744
$U(t)$	degree of consolidation

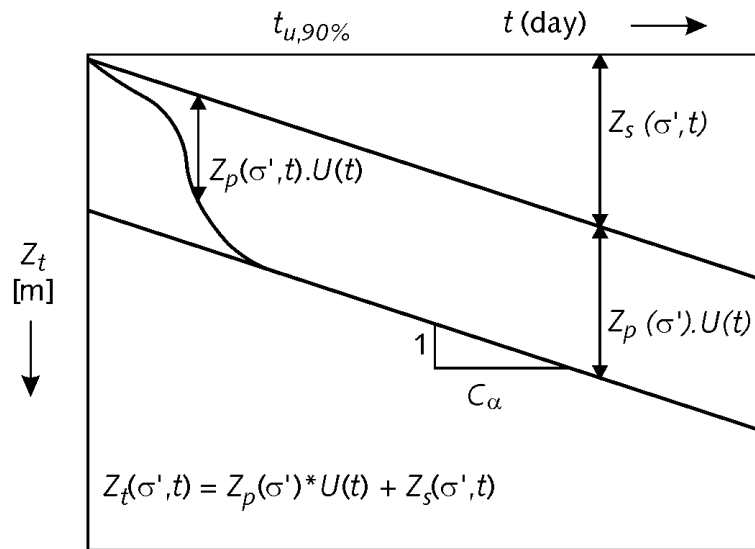


Figure 4.3.5 Primary and secondary compression

Consolidation

Terzaghi's consolidation theory provides an explanation of the effect of time on the compression of soil that is entirely saturated by water. It maintains the following assumptions and limitations:

- The soil is homogeneous and entirely saturated by water
- The grain material and the water are incompressible
- There is a linear relationship between compression and increase in effective stress
- The consolidation coefficient is constant for the duration of the consolidation process

The compression is minor in relation to the thickness of the soil layer in question. A characteristic parameter for the consolidation process is the consolidation coefficient.

[EQUATION] (4.3.5)

$$c_v = \frac{k_v}{m_v \gamma_w} \quad (4.3.5)$$

In which:

- C_v the consolidation coefficient (m^2/s)
 K_v vertical permeability coefficient (m/s) (§ 4.3.8)
 m_v vertical volume-compression coefficient. In general, for sand $m_v = 10^{-5}$ to 10^{-4} m^2/kN ,
for clay $m_v = 10^{-4}$ to 10^{-3} m^2/kN
 γ_w mass of water by unit volume (kN/m^3)

The relationship between compression at time t and the final compression at time $t = \alpha$ is defined as the vertical consolidation level U . The consolidation level is therefore a function of time. In order to represent the relationship between consolidation level and time, a time factor T is defined:

[EQUATION] (4.3.6)

$$T = \frac{c_v \Delta t}{(\alpha h)^2} \quad (4.3.6)$$

In this equation, α is a run-off constant. In equation (4.3.7), a distinction can be made between one-sided run-off and two-sided run-off. In one-sided run-off ($\alpha = 1$), the water runs via the top, or via the bottom of the layer in question. Two-sided run-off ($\alpha = 0.5$) occurs via the top and the bottom of the layer. The relationship between the consolidation level and the time factor is shown in Figure 4.3.6.

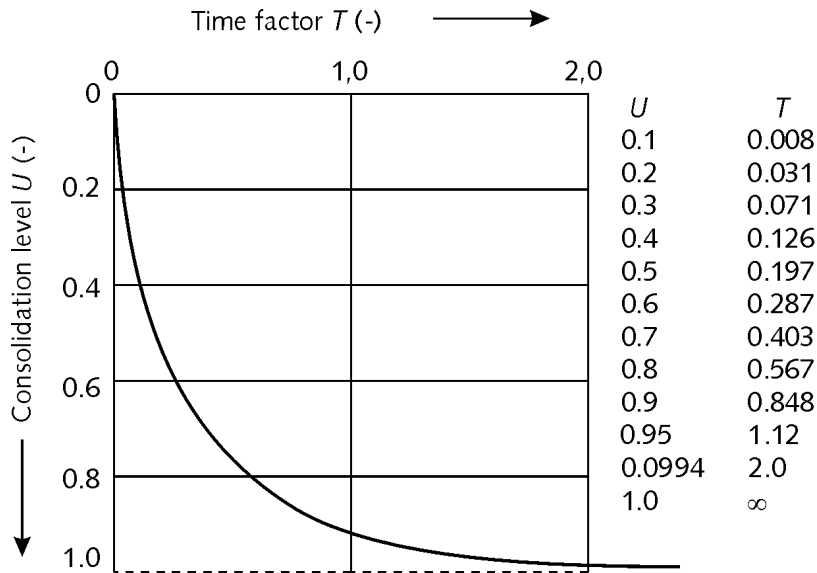


Figure 4.3.6 Relationship between consolidation level U and time factor T

The consolidation process ends when the pore pressure has entirely disappeared. Theoretically this is not achieved until after an infinite amount of time. Therefore, in practice, a consolidation level of $U = 0.994$ is often used, in which $T = 2.0$. Using equation 4.3.7, the consolidation period, (duration Δt_e), can be calculated up to what, for all practical purposes, is the end of the consolidation process.

[EQUATION] (4.3.7)

$$\Delta t_e = \frac{2(\alpha h)^2}{c_v} \quad (4.3.7)$$

Indications of the duration of the consolidation process for soil structures as in West-Nederland are:

- $\Delta t_e = (\alpha h)^2/5$ (year) in the event of primarily peat
- $\Delta t_e = (\alpha h)^2/2.5$ (year) in the event of mixed clay and peat ground
- $\Delta t_e = (\alpha h)^2/1.25$ (year) in the event of primarily clay

In this, Δt_e and h are expressed in years and metres respectively. A far higher Δt_e is usually valid for highly compressible clay with vegetative remains ($\gamma_{sat} = 13 \text{ kN/m}^3$).

FEM method for dimensional behaviour

When calculating multi-dimensional deformation and settlement in complex layer structures and/or complicated loading configurations, the methods described above are unsuitable. In such cases, a Finite Element Model (such as PLAXIS) provides a realistic means to determine the relationship between pressure and deformation.

Within PLAXIS, a number of models of materials have been used to describe the behaviour of soil. Table 4.3.1 provides an overview of the available models and the material

parameters they require. Generally, the Soft Soil Creep method is recommended for settlement calculations. For a description of this model, refer to the Plaxis manual (Ref. *Plaxis manual*). Den Haan's model is comparable to the 1-dimensional representation of the Soft Soil Creep model as implemented in the finite elements program Plaxis.

Table 4.3.1 Available material models and required parameters in Plaxis

Material model / parameters	Strength			Stiffness						
	ϕ' [°]	C^1 [kNm ²]	ψ [°]	v [-]	E_{ref50} [kNm ²]	E_{oed}^{50} [kNm ²]	m [-]	λ^* [-]	K^* [-]	μ^* [-]
Mohr Coulomb	x	x	x	x						
Hardening Soil Model	x	x	x	x	X	x	x			
Soft Soil Creep Model	x	x	x	x				x	x	x
Soft Soil Model (Cam clay)	x	x	x	x				x	x	

ϕ'	angle of internal friction (from a CU triaxial test, see Annex Triaxial test)□
C^1	cohesion (from a triaxial test)
ψ	dilatency angle
v	transverse contraction coefficient
E	elasticity modulus (from triaxial and other tests)
E_{ref50}	elasticity modulus (e.g. from triaxial test) at 50% of the peak shear strength in a triaxial test associated with a particular reference pressure
E_{oed}^{50}	elasticity modulus determined with a compression test
m	exponent in the Hard Soil model sets stiffness as dependent on pressure level (for sand $m = 0,3$)
λ^*	primary compression constant (from compression test)
K^*	swell index (from compression test)
μ^*	secondary compression constant (from compression test)

N.B. there is a direct relationship between compression constant C_c , C_s and C_α (NEN 5118) and the parameters λ^* , K^* , and μ^* . These relationships are:

EQUATION without number (4.3.8) □

$$\lambda^* = \frac{C_c}{2,3(1+e_0)} \quad K^* = \frac{C_s}{2,3(1+e_0)} \quad \mu^* = \frac{C_\alpha}{2,3(1+e_0)}$$

□

4.3.9 Permeability

The permeability of soil is characterized by the parameter κ . The permeability of the clay revetment a dike directly influences its quality in its function as water defence. The permeability determines the height of the phreatic line under normative circumstances, and thus influence the macro-stability (§ 5.3).

The permissibly of clay and peat in the substratum contributes to the compression behaviour (see § 4.3.8). The permeability of these layers directly influences:

- The consolidation period, and therefore the settlement development
- The stability during construction with respect to the adaptation of the layers (degree of consolidation) to the load applied
- The rate of implementation

The permeability of sand in a water-bearing layer (aquifer) is important when investigating piping and the potential development in the sand, thus the associated stability. The permeability can be determined by means of permeability tests. For this purpose the constant head test for highly permeable soil types such as sand) or the falling head test (for soil types of poor permeability such as clay) can be used (see § 4.4.10).

Practice shows that it is difficult to determine the permeability of soil types such as clay and peat the laboratory and generates a wide distribution in test results. For this reason, field tests such as the pump test (§ 4.4.9), the infiltration test (§ 4.4.10) or the Hooghoudt test may be preferable. The pump test provides information on the permeability of the water-bearing sand beds (aquifers) and the hydraulic resistance of the overlying layers with poor permeability. The infiltration test is suitable for the upper layers directly under ground level. The Hooghoudt test is suitable for layers below the ground watertable extending down to some 4 metres below ground level.

The TAW creep investigation (*Ref. Construction in soil, CUR 178 and CUR 191 Backgrounds in numeric modelling of geo-technical structures, Parts 1 and 2*), has shown that with regard to the behaviour of soil, the results of the calculation from the Msettle and the Soft Soil Creep models in Plaxis are highly sensitive to the degree of permeability.

4.4 Soil Analysis

4.4.1 Introduction

There are many soil analysis techniques. In 4.4 those most often used in the Netherlands are considered. These can be divided into two categories.

Field tests:

- geo-physical tests (§ 4.4.2) □
- penetration tests (§ 4.4.3) □
- drilling (§ 4.4.4) □
- vane tests (§ 4.4.5) □
- pressiometer, dilatometer and cone pressure meter tests (§ 4.4.6) □
- pore pressure measurements and hydraulic head measurement (§ 4.4.7) □
- electrical density tests (§ 4.4.8) □
- permeability tests (§ 4.4.9, § 4.4.10) □
- Radioactive density measurements (Ref. Standard RAW-regulations, Test 4.6) □
- Press ring method (Ref. Standard RAW-regulations, Test 4.4) □

□

□

laboratory tests

- determination of the weight by unit volume (*Ref. Construction in Soil*) □
- permeability tests (§ 4.11) □
- compressibility tests (§ 4.4.12) □
- direct shear tests (§ 4.4.13) □
- DSS test (§ 4.4.13) □
- triaxial test (§ 4.4.13) □
- sieve test (Ref. Standard RAW-regulations, Test 6.0) □
- determination of plasticity (Ref. Standard RAW-regulations, Test 15) □

Tables 4.4. and 4.4.2 indicate which tests are appropriate for the determination of the various soil properties. Geophysical techniques are not considered. For a detailed description reference should be made to CUR 162 (*Ref. Constructing with soil*).

Table 4.4.1 Flow methods of investigation

	Flow methods of investigation							
	Penetration test	Drilling	Determination of unit volume	Sieve tests	Determination of plasticity	Measurement of pore pressure	Compression tests	Triaxial test
Section or Reference	4.4.3	4.4.4	CUR 162	¹ 6.0	¹ 15	4.4.7	4.4.12	NEN 5117
Soil properties								
Compressibility							x	
Horizontal deformation								
Bedding	x	x						
Thickness of Bed	x	x						
Permeability				2x				
Shear strength (undrained)								x
Angle of friction (effective)								x
Cohesion (effective)								x
Pore pressure and hydraulic head						x		
Density grade								
Hydraulic resistance								
Grain distribution				x				
Classification	x	x						
Mass by unit Volume			x					
Index of Plasticity					x			

¹ Refers to the test number in question in the Standard RAW-regulations

² Indicative with regard to VO calculations

Table 4.4.2 Specialist methods investigated

	Specialist methods investigated									
	Nuclear/ radio active	Press ring method	Electric density measure	Vane test	Pressio, dilation cone	Pump/w ell tests	Infiltratio n tests	Permeab ility tests (laborat	Direct shear test	DSS test
Section reference or	4.6	4.4	4.4.8	4.4.5	4.4.6	4.4.9	4.4.10	4.4.11	4.4.13	4.4.13
Soil properties										
Compressibility										
Horizontal deformation					x					
Bedding										
Thickness of bed										
Permeability					x	x	x	x		
Shear strength (undrained)				x	x				x	x
Angle of friction (effective)									x	x
Cohesion (effective)										
Pore pressure and hydraulic head										
Density grade	x	x	x							
Hydraulic resistance						x	x	x		
Grain distribution										
Classification										
Mass by unit volume	x									
Index of plasticity										

¹ Refers to the test number in question in the Standard RAW-regulations

If no soil analysis has taken place and a first exploratory calculation has to be made, the parameters recorded in Table 4.4.3 can be used. These are relatively conservative values (on the safe side) and are only intended to be informative. More detailed indicative soil properties are provided in NEN 6740 and CUR 162 (*ref. Construction in Soil*). Depending on the quality of the measurements and the number of measurements, values can be calculated by using factors relating to the material for these parameters that are less conservative than the values in Table 4.4.3

Table 4.4.3 General soil properties

Type of soil	Mass by unit volume	Angle of internal friction	Cohesion	Primary compression constant	Swell constant	Secondary compression constant	Permeability
	ρ	ϕ'	C'	C_c	C_s	C_α	κ

	[kNm ³]	[°]	[kNm ³]	[-]	[-]	[-]	[m/s]
Sand	19-21	30-38	0	0.002-0.02	0.001-0.007	0	>10 ⁻⁵
Clay	16-19	20-30	1-10	0.126-1.36	0.04-0.45	0.004-0.012	10 ⁻⁸ -10 ⁻⁶
Peat	10-11	15-25	1-15	0.9-7.6	0.3-2.5	0.016-0.023	10 ⁻⁸ -10 ⁻⁵

4.4.2 Geophysical investigation

Introduction

This section contains general information about various geophysical techniques for use in the field to determine the structure of the soil. In addition to providing non-destructive means to determine the soil structure, geophysical measurements can also be used to detect cables, pipelines and other obstacles. The data for each technique are summarized in Table 4.4.4 (Ref. Inventory of techniques for and RWS-experience of the construction of a digital field model in a river area). For the sake of completeness some irrelevant techniques have been included in the table and indicated as 'not relevant' in the column relating to suitability for use in investigations for water retaining structures

General

The purpose of geophysical measurements is to support the interpolation between drilling and sounding measurements and thus to reduce the number of such measurements that is required to establish a geological model. The results obtained by using geophysical techniques consist of profiles and/or contour maps that are obtained by interpreting of the data. For a reliable interpretation of the result of geophysical measurements it is always necessary to carry out calibration drilling. Geophysical measurements cannot provide accurate depth measurements of the bedding planes to within centimetres.

The results of geophysical measurements are to a large extent determined by the nature of the substratum. This sets limits on the suitability of the methods. When using geo-electric and electro-magnetic methods it is necessary to take equivalence into account. This means that different geological models can give the same measured result.

The vertical accuracy of the results of the measurements is dependent not only on the accuracy of the position fixing system, but also on the accuracy with which certain parameters have been estimated or determined. Parameters that are used in the interpretation are:

- the velocity of sound in the sediment (seismic)
- the electrical resistance of the beds (geo-electrical measurements)
- the velocity of electromagnetic waves (soil radar)

All seismic and electromagnetic methods used in the field are laborious and thus expensive. For these methods cables must be laid out and geophones or electrodes inserted into the ground. To date seismic refraction can only be used in deeper layers. Moreover seismic refraction cannot detect layers with a lower sonic velocity than the layers lying above upper (hidden layer). A detailed description of the instruments and their use in practice can be found in CUR 162 (Ref. *Construction in soil*, 4.3.1).

Table 4.4.4 Geophysical techniques on land

Method	Parameter	Resolution	Depth	Result	Throughput time	Limitation	Status	Suitability for investigation of water retaining structure
High resolution seismic profiling	Low stiffness limits	1 m	50-1000	Profile/ contour map (interpreted)	1 day rec. 3 days proc. rec.750 m/day	Not less deep than 30 m	In development for shallow measurements	Not relevant owing to depth
Seismic refraction		0.3m	0-200	Profile/ contour map (interpreted)	1 day rec. 3 days proc. rec.750 m/day	Velocity inversion 'hidden layer' local noise	Product	0
Seismic tomography		0.5 m	Depending on the depth of the drill hole	Profile (interpreted)	1 day rec. 3 days proc.	Only in existing boreholes laborious and equivalence	New product development in progress	0
Geo-electric penetration test	Resistance	Ca. 10-15% of the depth	10 – 1000	Profile/ contour map (interpreted)	1 day rec. 3 days proc. rec.750 m/day		Product execution and processing in development	+
Geo-electric profiling	Resistance	Ca. 10-15% of the depth	0-300	Profile/ contour map (interpreted)	1 day rec. 3 days proc. rec.1.5 km/day	Equivalence	Product execution and processing in development	+
Electromagnetic Profiling (TEM)	Conductivity	Resolution decreases rapidly with depth	0-100	Profile/ contour map (interpreted)	1 day rec. 3 days proc. Rec.750 m/day	Equivalence not in urban areas, not in salt/fresh ground water	Product	++

Electro-magnetic Profiling (FEM)	Conductivity	Resolution decreases rapidly with depth	0-100	Profile/ contour map (interpreted)	1 day rec. 3 days proc. Rec.7.5 km/day	Equivalence not in urban areas, not in salt/fresh ground water	Product	++
Soil-radar (low frequency)	Geometry and indication of porosity	0.2 m to several metres (depending on frequency)	0-20	Profile (interpreted)	1 day rec. 3 days proc. Rec.10 km/day	Not in clay not in salt/fresh ground water	Product processing development	0
Soil-radar (high frequency)	Geometry of obstacles	Several centimetres to a decimetre	0-3	Profile p (interpreted)	1 day rec. 2 days proc. rec., 10 km/day	Not in clay not in salt/fresh ground water	Product processing Development	-
Soil radar (tomography)	Indication of porosity	0.5m	Depends on depth of drill hole	Profile (interpreted)	1 day rec. 2 days proc.	Very laborious Only in existing bore hole	New product development in progress	0
Gravimetry	Density variations	Qualitative	10-10000	Contour map (interpreted)		Small objects at great depth and small density variations	Product	Not relevant
Gamma logging	Clay content heavy minerals	Qualitative	Depends on depth of drill hole	Profile (interpreted)		Very laborious Only in existing bore hole	Product marketing and processing in development	-
Neutron		Quali	Depen	Profile		Very	Product	Not

logging		tative	ds on depth of drill hole	(interpr eted		laborio us only in existing bore hole	marketi ng and processi ng in develo pment	relevant
---------	--	--------	------------------------------------	------------------	--	--	---	----------

+) not suitable 0 moderately suitable + suitable ++ suitable and often used

rec. recording proc. processing

Explanation of Table 4.4.4

- 'Method' in this column indicates the technique under consideration
- 'Parameter' in this column indicates what is being measured or observed
- 'Depth reached' in this column indicates the depth reached in relation to the ground surface
- 'Result' in this column indicates the type of end product. For all geophysical measuring methods this is an interpreted contour map of an interpreted vertical profile
- 'Throughput time' in this column indicates the time needed for the interpretation of the data of one measuring day. The relation between the measuring time and the processing time improves increasingly as the duration of the measuring period increases, because a number of actions remain the same however long the measuring period.
- 'Limitation' in this column indicates the limitations imposed by the method in question:
 - Nature of the substratum
 - Equivalence
 - External factors such as high tension cable and sheet piles
- 'Status' in this column indicates which phase the development of the method has reached
- 'Suitability to investigate water-retaining structures' in this column indicates the degree to which the method in question is applicable in determining a geological model for the design of a water retaining structure.

Application in the field

For water defences, geo-physical measurements are generally performed longitudinally. If they relate to a new dike, longitudinal measurements should be taken along two lines; one along the outer toe and one along the inner toe. It is desirable that measurements are also taken along a line 15 to 20 metres from the inner toe.

In an existing dike, which is to be included as part of the new dike, a geo-physical investigation should be made along a line as close as possible to the inner toe. No useful purpose is served by taking measurements along the crown of a dike, as the results are seriously distorted by the presence of pipelines and cables in the soil. Moreover, the dike is already higher than surrounding area, as a result of which the two-dimensional measurement is not representative. Further insight into the soil structure can be gained by taking transverse measurements on the dike, although such measurements are often omitted.

The practical application of geo-physical methods depends on the following aspects:

- In an area with brackish or salt groundwater, the absolute value and the difference in electric resistance are smaller; in such cases the interpretation of the measurement results demands close attention and considerable professional skill. The specific resistance decreases with an increase in the salinity of the groundwater. It should be noted that the use of these methods is not always possible under brackish or saline conditions.
- In a many-layered structure or if the most important, strongly compressible and weak layers are far below ground level, geophysical measurements are less suitable.

- Geo-electric measurements require a certain amount of physical space. For this reason, it is usually impossible to take measurements on the crown of a dike, and measurements are influenced by ditches and buildings.

Penetration tests and drilling are also required for a good interpretation of geo-physical measurements. For a further description of the possible uses and restrictions on geo-physical measurement techniques, reference may be made to the literature, including CUR Publication 182 (*Ref. Geo-physical techniques in Soil Analysis*) and (*Ref.*) *Applied Geophysics, code of practice*.

Figure 4.4.1 provides an example of a geo-physical trench measurement, using three different electrode distances per measurement location. This illustrates how soil structure is expressed as resistance profile.

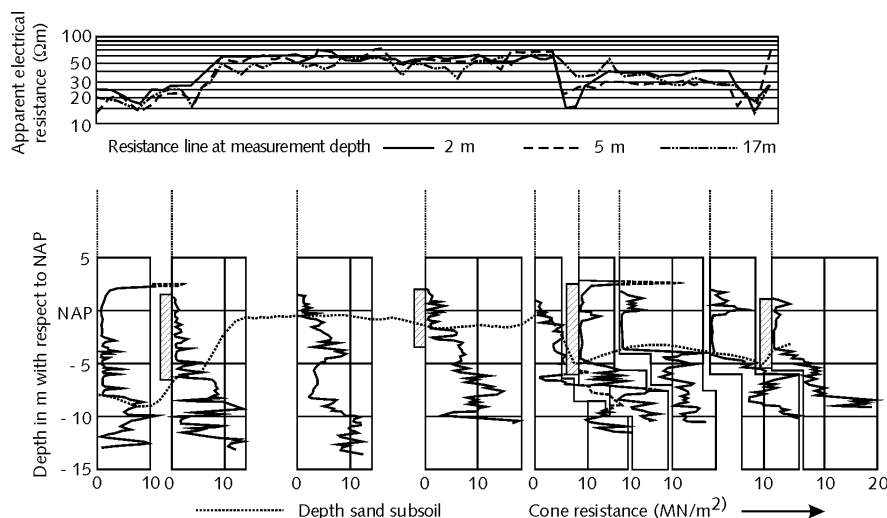


Figure 4.4.1 Example of geo-electric profiling

4.4.3 Penetration tests

In determining soil structure, the standard electrical pressure penetration test (Cone Penetration Test), which measures skin friction ('stickiness'), or mantle friction, is an important tool. In principle, penetration tests measure the resistance undergone by a cone-shaped body (see Figure 4.4.2) under pressure in the soil. The method has been standardized both nationally (*Ref. NEN 3680*) and internationally. By using the friction measurement, the bedding planes between such soil types as clay and peat layers with the same cone resistance can be made visible, providing a considerably clearer picture than is given by cone measurement alone. The electrical penetration test with the friction measurement can be used in almost all types of soil in the Netherlands and is sufficiently accurate. Moreover penetration measurements are also relatively cheap.

Penetration tests according to NEN 3680 can be implemented by using penetration cones of various shapes. Since the shape of the cone affects the resistance of the soil, it is necessary to indicate the type of cone suited each type of soil according to NEN 3680. More information relating to the sensitivity of the cone with respect to soil structure can be found in CUR 162 *Ref. Construction in Soil; § 4.3.2*.

In NEN 3680, the penetration tests are divided into three quality classes. The mechanical cone falls into the lowest quality, class 1 and is now seldom used. Mechanical cones are still in used soil containing a great deal of rubble, which would damage a less robust electrical cone. Current practice makes almost exclusive use of electrical penetration testing. The development of soil rigidity with depth, as shown by cone resistance and local friction, provides a good picture of layer structure and composition of the soil layers. Figure 4.4.2 shows a penetration cone, while Figure 4.4.3 provides an example of a penetration test with a cylindrical electrical friction mantle cone.

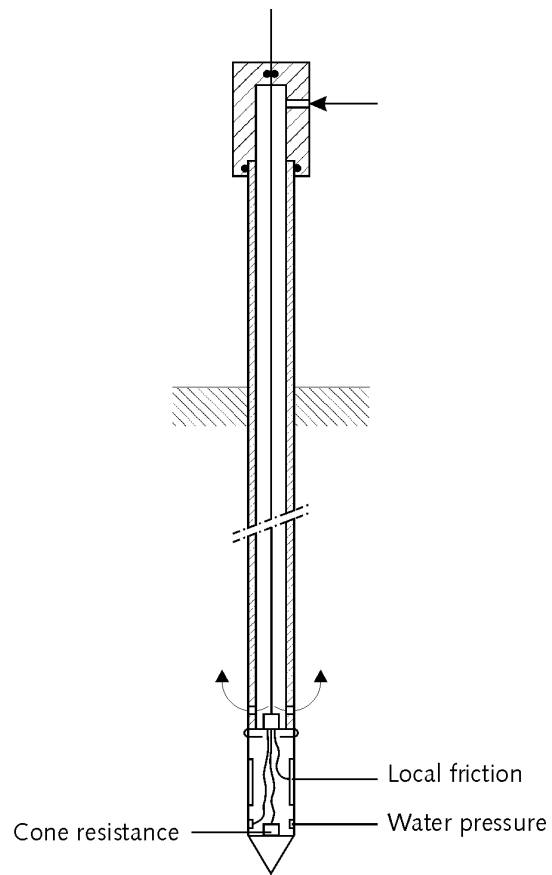


Figure 4.4.2 Penetration test cone

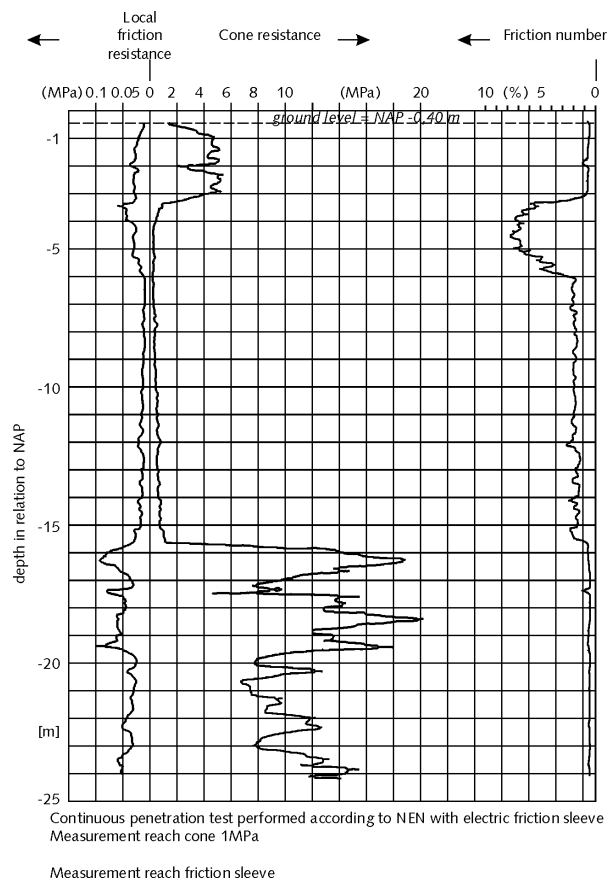


Figure 4.4.3 Example of a penetration test graph

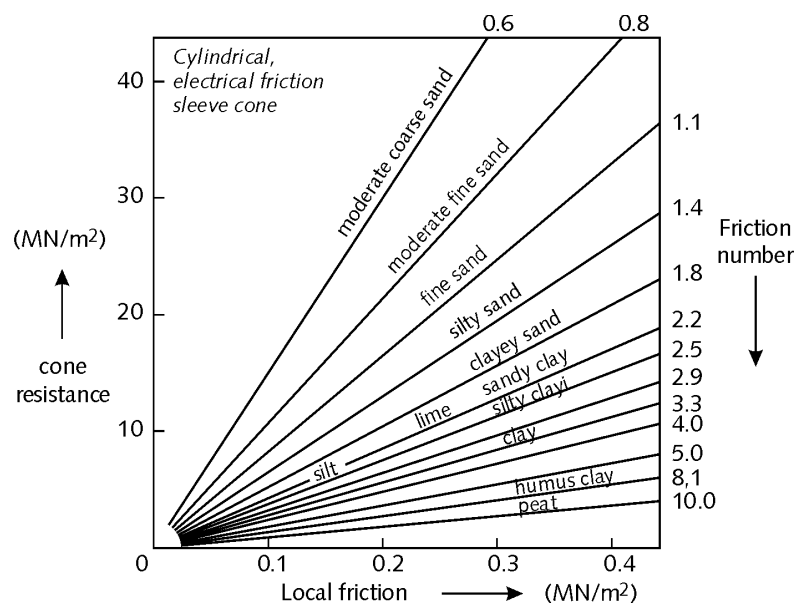


Figure 4.4.4 Relationship between probe resistance, local friction and soil type

For soil below the level of the ground watertable the friction value gives a good indication of the type of soil. Figure 4.4.4 shows the relation between cone resistance and local friction in various types of soil for penetration tests with the cylindrical electric friction-mantle cone.

When complemented by the results of a number of manual drilling samples and geo-electrical testing, penetration tests provide a reliable picture of the location and thickness of the various soil layers. There are innumerable correlations with penetration resistance and compression parameters. The most commonly used correlations are given in the CUR-publication 163, Annex A (Ref. *Construction with soil*)

4.4.4 Drilling

In comparison to penetration tests, drilling is an expensive alternative and they are generally used for the classification and identification of the soil. Drilling is an effective, yet laborious method to accurately determine a soil profile. Classification of the soil layers is usually done by visual description of soil raised by the drill according to (Ref.) *NEN 5104*. It is desirable that the visual assessment should be verified with the aid of laboratory classification tests on samples of the soil in question. Examples of these tests weight by volume determination, grain distribution tests, and determination of the flow limits, roll-out limit and shrinkage limit of clay. Drilling is often performed to acquire undisturbed samples from various soil layers. These samples are used to determine weight by volume and mechanical properties such as strength and deformation parameters of the soil. They form the basis of the soil-mechanical analysis required for the assessment of the stability of a dike (§ 5.4). It is therefore of the greatest importance that the samples are truly undisturbed when they are removed from the soil and arrive in the laboratory. A short description of the drilling methods used in the Netherlands is given in Annex 3.

4.4.5 Vane tests

Vane tests are performed both in the laboratory and in the field to obtain direct measurements of the undrained shear strength of cohesive soil. In principle, the method depends on determining the torsion moment required to make the vertical axis rotate in the soil. By alternately drilling more deeply and carrying out vane tests it is possible to obtain a soil profile in a simple way that gives a picture of the undrained shear strength as a function of the depth (Figure 4.4.5). Annex 4 gives a detailed description of the vane test and the apparatus that is required for this test.

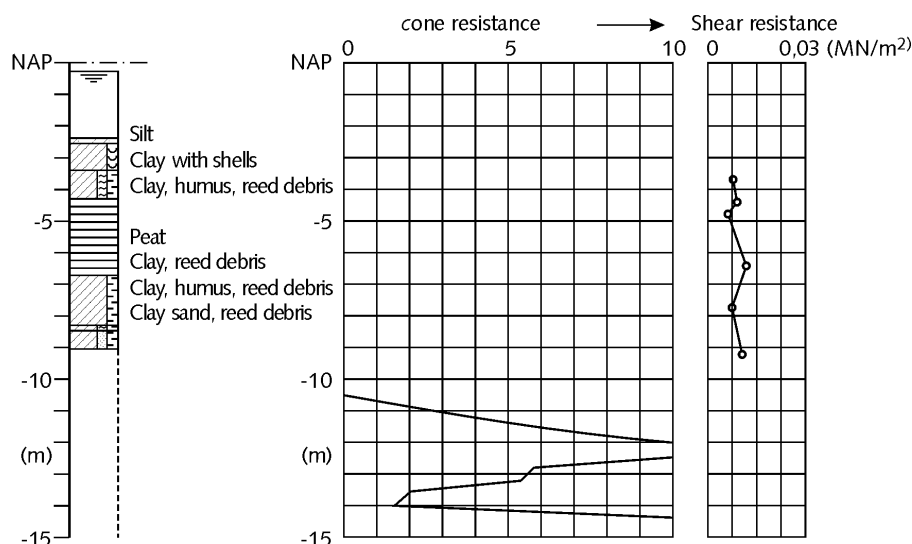


Figure 4.4.5 Development of undrained strength with depth

4.4.6 Field investigation to determine deformation parameters

In situ measurement of deformation parameters is not straightforward as the distribution of loads applied and the development of water overpressure have to be simulated. For this reason, there are hardly any in situ methods that offer deformation parameters directly. Interpretation is always required.

Two field tests that can be used to determine deformation parameters are the pressiometer and dilatometer tests. A cone pressure meter test is a method used to calculate both strength and stiffness properties of the substratum. Within the framework of the TAW investigation 'determination of parameters' a user-friendly model (PIANO) has been developed. This model can directly determine the slip modulus and the undrained shear strength from the measurement results of the cone pressure meter and use these in an FEM calculation. In Annex 5 the pressiometer, the dilatometer, the cone pressure meter and the investigations for which these instruments are used are described.

4.4.7 Measuring pore pressure and hydraulic head

The location of the groundwater levels influences the various geo-technical and geo-hydrological calculations. The actual and, where possible, both the highest and the lowest hydraulic head of the groundwater must be determined. In addition, it is necessary to ascertain which factors affect changes in the water level. The effects on water level that may occur as a result of infiltration, springs and water extraction, (e.g. drinking water extraction) are important as also are, for example, the presence of a river with varying water levels. The groundwater pressure can be measured by means of piezometric penetration, although this relates to only a moment in time. For short and long-term measurements, it is preferable to use other measurement techniques. In the Netherlands, wide variety of water pressure meters is available. The most commonly used include:

- The open piezometric gauge
- The Bourdon water pressure meter
- Electric water pressure meters

Table 4.4.5. gives an overview of the use, accuracy, reliability and cost of the water pressure measurement devices.

Table 4.4.5 Evaluation of pore pressure relating to applicability on soil type

Measurement device	Soil type	Accuracy	Reliability	Costs
Piezometric gauge	Sand peat ¹⁾	++	++	++
Bourdon water pressure meter	Sand, peat, clay ¹⁾	+	-	O
Electric water pressure meter	Sand, peat clay	++	+	--

¹⁾ Not applicable for cyclic or dynamic water pressure measurements

Explanation:

++ = good or inexpensive o = moderate

+ = reasonable

- = poor

-- = very poor or very expensive

For more information on the applicability of the various measurement methods, reference may be made to CUR 162 (*Ref. Construction in Soil*). During penetration tests, pore pressure at the cone point is measured continuously (see Figure 4.4.6).

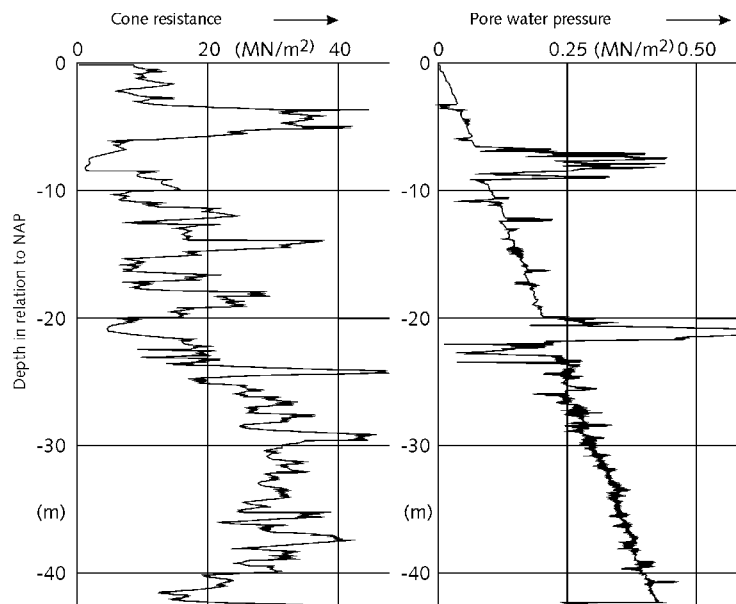


Figure 4.4.6 Example of penetration test with pore pressure measurement

4.4.8 Electrical density measurement

Chapter 2 mentions settlement flow as a failure mechanism. To determine the sensitivity of the foreland to this mechanism it is necessary to determine the in situ density of the sand layers. The previously described methods of investigation such as penetration tests and drilling are not suitable for this but GeoDelft has developed apparatus which can be used for this purpose. It is an expensive specialist investigation but it must be remembered that serious damage results from the occurrence of settling.

GeoDelft has developed instruments to determine pore content of water-saturated sand on site as a function of depth. These instruments can be introduced into the ground from land or from the water bodies using standard penetration equipment. The method relies on the fact that sand grains do not conduct electricity, while the water in the pores does conduct electric charges. The measurement itself consists of two phases. In the first phase, the specific electrical resistance of the entire soil mass, including both grains and pore water, is measured using the soil probe. This soil probe consists of a special penetration tube, containing the electrically insulated measurement elements. The measurement elements consist of two combinations of four electrodes. One combination has a zone of influence extending for a few decimetres, while the other has a zone of influence of about one meter. The probe is also provided with a cone and a friction mantle, allowing simultaneous recording of cone resistance and local mantle friction. The probe is inserted into the soil, and the specific electric resistance is measured every 0.20 metres.

In the second phase of the measurement, the specific electric resistance of the pore water is determined by means of a 'water probe'. The water probe consists of a measurement cell that can be filled by sucking up a small volume of pore water via a filter, after which the specific electric resistance of this water is measured. This measurement is performed at various depths.

The pore content can be deduced from the quotient of both specific electric resistances measured. It is not possible to make a theoretical deduction of the relationship as a sand mass consists of grains of various shapes and sizes, irregularly ordered. The desired relationship is therefore determined in the laboratory, as a calibration line, by performing tests on samples from the sand beds in question. For this a water saturated sand model with a known pore content is built up in an insulated cylinder. This allows measurement of the specific electric resistance of both the saturated sand and the pore water. This test is

repeated for various pore contents of a sample. After calibration of a number of samples from the area under investigation an average calibration line is determined.

4.4.9 Pump and pit tests

Pump and pit tests are used to determine geo-hydrological parameters such as the permeability and absorption coefficient of a water-bearing bed, and the resistance to vertical flows through a layer with poor permeability.

In a pump test, water is extracted by means of one or more pits from one or more water-bearing beds. During the pumping, and for a certain period of time afterwards, the changes in hydraulic head are measured at as many points as possible at various distances from the pump pit. By using the phreatic lines that are registered in the level filters it is possible to calculate the various geo-hydrological properties of the substratum.

When during pumping only the hydraulic head is recorded in the pump pit or in the gravel box, of the pit this is called a pit test. This is a simplified version of the pump test that is frequently used to prevent the need to drill pits and install expensive level filters. However the disadvantage is that this test can only be used to determine the permeability of the water-bearing beds. The soil parameter calculated in this way also applies only for the immediate surrounds of the pit. Much research into geo-hydrological parameters using pit and pump tests is performed by various institutes in the field of drinking water provision. The results are collected by Groundwater Investigation Service of TNO (Dienst Grondwaterverkenning).

4.4.10 Infiltration tests

Infiltration tests are used to determine the vertical permeability of soil in situ. In these tests, a ring of usually about 0.15 metres in diameter is placed at ground level. To prevent peripheral effects, this is surrounded by a larger ring of 1 metre diameter. Both rings are filled with water. The permeability can be deduced from the additional flow and the measured pore pressure or hydraulic head in the ground under the rings. A disadvantage is that, despite the presence of a second ring, a certain peripheral effect cannot be avoided as a result of lateral flows, which makes it impossible to measure pure vertical permeability. The advantage of these tests over permeability tests in the laboratory is that the sample tested is more representative with regard to the structure in the soil such as splits and worm holes. Moreover, the sample is undisturbed. The test must be continued until a continuous downward flow is reached. This can take some time (several hours).

4.4.11 Permeability tests in the laboratory

Depending on the type of soil, two types of tests are used for testing permeability in the laboratory. These are the 'constant head' test and the 'falling head' test, described respectively in (Ref.) NEN 5123 and NEN 5124.

Constant head test

The constant head test is used for soil that has a good degree of permeability and is weakly compressible, such as sand. In this test, water is allowed to flow through the sample at a constant rate. The volume of water that flows through the sample per unit of time is recorded. On the basis of flow thus measured it is possible to determine the permeability of the soil by using Darcy's law. The sample is often disturbed. The test is performed on the sample several times under various packing densities. The 'constant head' test can also be performed in a triaxial instrument.

Falling head test

The falling head test is used for weakly permeable soil that has a high degree of compressibility, such as peat and clay. In these tests, the pore content, and therefore the permeability, is highly dependent on the degree of compression. Permeability tests on clay

and peat are therefore performed by applying various loads on the sample. The compression equipment is often used to do this.

The permeability with a particular load applied from above is calculated on the basis of the settlement of a water column in a very narrow vertical tube that is connected to the soil sample. This method is necessary since the influence of evaporation on the very small flow from the sample must be minimized.

A similar type of test is used to determine horizontal permeability. The limit for applying the 'constant head test' and the 'falling head test' is approximately 15 weight percent of the parts < 16 µm.

Great uncertainty is associated with use of small samples to determine the permeability of clay and peat in the laboratory. The actual permeability of these types of soil is often determined by considering the splits and wormholes that are present. Therefore it is better to determine the permeability in situ by using pump tests or infiltration tests.

Recent experience with two advanced creep models has shown that the permeability exerts great influence on creep behaviour. The number of permeability tests can normally be limited since:

- By using compressibility tests for clay and fractional analysis for sand, it is possible to estimate permeability
- Local permeability does not always play a large role in permeability of a larger mass.

4.4.12 Compression test (odometer test)

The compression test, or odometer test, is used to determine compression constants and consolidation characteristics of the soil in the laboratory. Annex 6 gives a detailed description of the compression test the equipment required for this.

4.4.13 Measurement of shear stress parameters in the laboratory

The most frequently used method to determine the shear strength parameters is the triaxial test. The triaxial test is described in CUR 162 (Ref. *Constructing with soil*) and (Ref.) *NEN 5117*. The old direct shear test had a few disadvantages, but new developments have led to the 'Direct Simple Shear Test' becoming available.

Direct shear test

The direct shear test is one of the oldest testing methods used to determine friction properties, in particular the undrained shear strength. In Annex 7 there is a description of the direct shear test and the equipment required for this. The test is primarily used to determine the friction properties of sand and the undrained shear strength of clay. A drained shear test is seldom carried out on a clay sample because of the long duration of the test. An exception is made when measuring the angle of friction for very large cumulative movements of clay samples (residual strength). In a triaxial test this cannot be done. In addition the test is used to test geotextile samples and membranes. The direct shear test has a number of disadvantages; the most important of these is that the test leads to overestimation of the friction characteristics. It is better to use the Direct Simple Shear test that is described below.

Direct Simple Shear Test

In the direct simple shear test (DSS-test) the soil sample is loaded with a constant vertical force. After the consolidation of the sample (one dimensional), the sample is pushed away, such that shearing deformation is possible in one direction. The DSS test gives a state of deformation that is more homogenous than that of the direct shear test, (Ref. *Comparison of truly undrained and constant volume direct simple shear test*. Annex 8 contains a description of the DSS-test and the equipment needed to carry out this test.

Critical shear plane

In the Direct Simple Shear Test the critical shear plane never coincides with the horizontal shear plane. However, in large deformations the angle between the critical shear plane and the horizontal shear plane is smaller. (Ref. *Lateral stress measurements during static and cyclic direct simple shear testing*).

The orientation of the critical shear planes depends on the entire stress state of the soil monster. In the standard configuration of most shear test equipment (as in the direct shear test, see Annex 7 the lateral shear stress is not measured. Only the shear stress ($T/A = \tau_h$) and the vertical normal stress ($N/A = \sigma_{vc}'$) are measured. This means that the entire stress field is not known. Accordingly it is assumed that the horizontal (or vertical) shear plane is the critical shear plane. From this follow the shear strength parameters (c and ϕ).

Recent developments

One of the recent developments in the DSS-test is the measurement of lateral stress in the sample (Ref. *Lateral stress measurements during static and cyclic direct simple shear testing*) with the apparatus of the NGL. The execution of these measurements during the shear test ensures that the entire stress state is known. In this case the results of the test agree with those of the triaxial test.

With most shear test apparatus a standard undrained test in which the pore pressure is measured cannot be entirely completed (Ref. *Comparison of truly undrained and constant volume direct simple shear test*). Instead of this shear tests are carried out in which the volume of the sample is kept constant. Because the equipment cannot test lateral deformation of the sample, it is only necessary to ensure that the height of the sample is kept constant during the test.

Anisotropy

In a shear test the load is not applied by means of extension or compression by a pushing movement. This difference in loading is significant when the strength (ϕ' and c') of the type of soil is strongly directionally dependent (Ref. *Constructing with soil*). In a number of cases the use of shear strength parameters from triaxial compression tests in slip plane calculations leads to a safety factor that is too low. With peat in particular anisotropic behaviour is very dominant

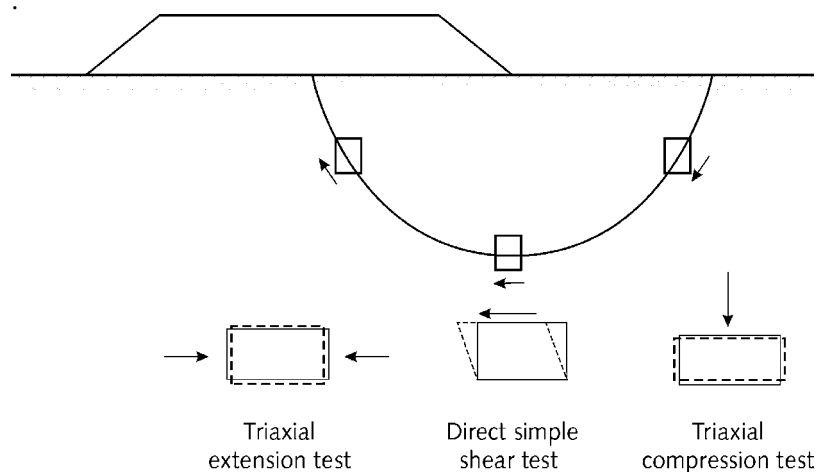


Figure 4.4.7 Slip plane and soil investigation

The dependence of the direction of loading can be obtained by using various tests (see Figure 4.4.7) and also the ADP method in Annex 2. In the neutral and possibly also in the passive zone of the slip plane shear strengths are assumed that can be measured with simple shear or direct shear tests or triaxial-extension tests.

□

4.5 Geo-hydrological aspects

4.5.1 Introduction

□

The stability of the dike body depends primarily on the pore pressure and hydraulic head in and under the dike during design loading. The design load of a dike is generally a normative high water level but might also occur, for example, in the event of heavy precipitation (see § 4.5). The acting pore pressure is dependent on the acting groundwater flow, which in turn is dependent on the geological bedding, permeability, retention capacity and consolidation process.

4.5.2 Groundwater flows in the Holland profile

The geology of the Netherlands is largely comprised of relatively impermeable, weak Holocene, upper layers, overlying relatively permeable and incompressible Pleistocene sand layers. This is the Holland profile (see Figures 4.5.1 and 4.5.2). Occasionally, deeper water-bearing layers also influence the flow pattern.

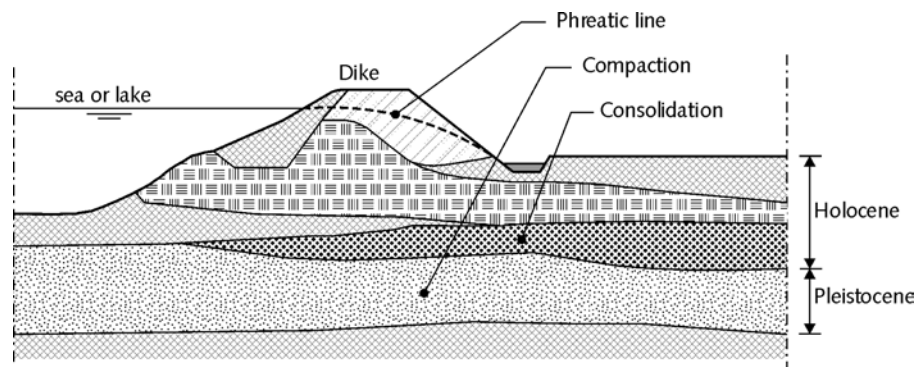


Figure 4.5.1 Holland profile and retention capacity

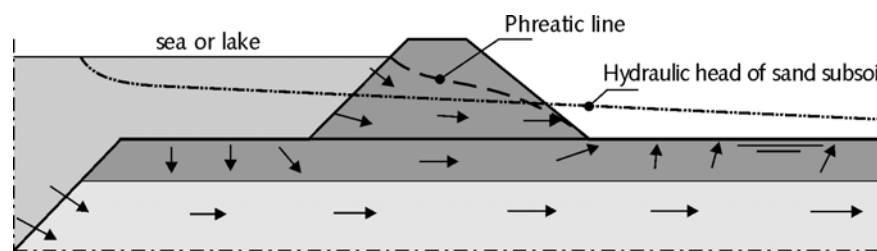


Figure 4.5.2 Groundwater flows in dike profile and substratum

The entry point of the groundwater is generally in the unprotected foreland. Water usually exits through the covering layer of the hinterland.

In these cases, the groundwater flow can generally be divided as follows (see Figure 4.5.2):

- A two-dimensional flow in a vertical plane through the dike body and the substratum
- The flow through the covering layer near the exit point that is virtually vertical
- The flow of the pressure flow in the sand substratum is virtually horizontal

The following elements are used in describing the hydraulic head in the dike and substratum

- The hydraulic head in the Pleistocene / water-bearing strata
- The location of the phreatic line
- The degree of adaptation / penetration length in the weak layer package

- The possibility of uplift and heave and the occurrence of a boundary potential under the sealing package

□

4.5.2 Points of attention in geo-hydrological descriptions

- The following aspects of groundwater flows deserve attention:
- Stationary and non-stationary flows
- Phreatic or elastic retention
- flows in the saturated capillary zone above phreatic line
- Flows along bedding planes
- Pressure flow induced by deformation (load) see also § 4.2 and § 5.3
- Hydraulic soil fracture and limit potential head

Further details are provided for a number of the above-mentioned aspects. Other aspects are considered in Annex 1.

Stationary / non-stationary flows

When considering groundwater a distinction can be made between stationary and non-stationary water flows. If the duration of the normative high water level in relation to the consolidation period of the covering layers on the water-bearing sand layer is long, stationary groundwater flows apply. If this relationship is of short duration, non-stationary groundwater flows should be taken into account.

If the boundary conditions (sea level, river level and polder level) are not stationary, in principle the groundwater flow is non-stationary. However, if the reaction of the groundwater flow system is such that the pressure flow almost immediately adapts to the variation in boundary conditions, the flow is also known as quasi-stationary. If this is the case, the flow can be seen as stationary per point in time.

Non-stationary flows are characterized by delay and reduction of the flow pattern with respect to the non-stationary preconditions. This is caused by retention of water in the soil and the consolidation in the Holocene series. This reduces the amplitude of a propagating wave.

Phreatic/elastic retention

There is a difference between phreatic and elastic retention. Phreatic retention occurs when the spaces in the granular skeleton of the soil are filled by freely rising, groundwater level. The characteristic parameter for phreatic retention is the porosity of the soil. Since the pores are never entirely filled with water, effective porosity is used here instead.

Elastic retention occurs when the groundwater pressure rises with unchanged soil pressure. As a result, the effective stress in the soil decreases and the grain skeleton that had been compressed under the influence of the effective stress will expand with roughly elastic properties. This creates extra space in the pores that can be filled by water. Compressing the pore water as a result of increased pressure (virtually nil in practice) allows additional water to be held. Elastic retention is particularly important in clay. In thick clay layers, considerably lower pressure flow may occur when a high water wave passes than in stationary flows.

In phreatic water in sand beds, elastic retention is almost negligible in comparison phreatic retention. In pressure flow in sand, elastic retention has a noticeable yet generally minor effect with respect to damping and delaying a propagating pressure wave in the sand package.

Flows in saturated capillary zones

Flow processes in a dike are not limited by the phreatic line on the upper side. Depending on the type of soil there is a particular zone above this line that also contains groundwater as a result of capillary action. The soil directly above the phreatic line is usually entirely saturated by water, while above this the degree of saturation decreases upwards. Only the fine capillaries are able to draw up water to these heights.

It is not possible to indicate a clear limit to the entire saturated zone above phreatic line. In practice, the limit is set at saturation level S_w of some 75%. Capillary action in coarse sand is

approximately 0.02-0.05 m, in reasonably fine to reasonably coarse sand it is about 0.12-0.35 metres, in silt some 0.70-1.50 metres and in clay 2-4 metres.

In the saturated capillary zone, groundwater participates in groundwater flows in the same way as the water below the phreatic line. In clay dikes, in which major capillary action is possible, flows may occur throughout the entire dike profile.

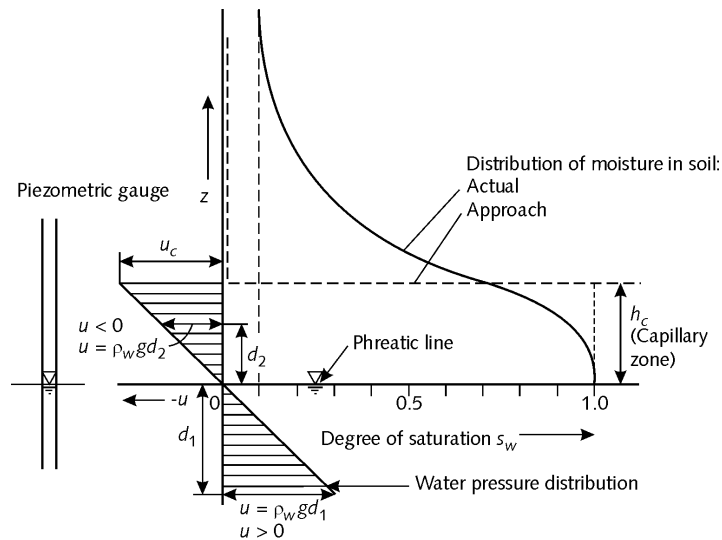


Figure 4.5.3 Capillary zone

The information above gives a generalized view of what takes place. In the unsaturated zone above the phreatic line, the flow model is complicated. It was not considered appropriate to cover it in great detail here (Ref. *Basic set-up of the technical report on water pressure. Limit potential head of the sealing package*

As special case the uplift of the sealing package under high water conditions must be taken into account. On the land side of a water defence, the potentials in the interface between the water-bearing sand layer and the sealing clay and peat package are so high that the sealing beds lift up. The upward water pressure is then equal to the downward pressure, being the weight of the soil layers behind the dike.

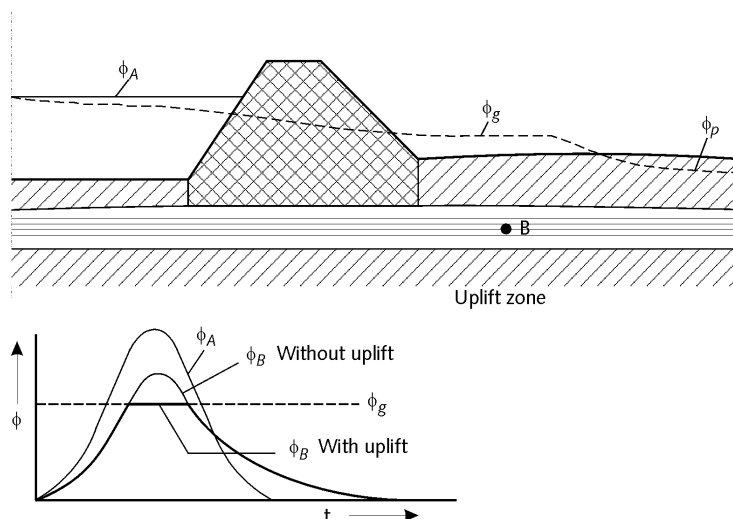


Figure 4.5.4 Pressure flow development in water-bearing layer in uplift

When considering the uplift and heave mechanisms, it is usual to express loading and strength as a hydraulic head. The hydraulic head required to cause uplift is called the limit

head potential. Subsection 5.5.3 covers the uplift and heave of a covering layer in greater detail.

In berm and dike ditches, in addition to the weight of the layers, other effects, such as bowing, play a role. The fact that a ditch does not heave can have negative effects on stability because the 'valve function' of the ditch no longer operates. It is therefore important to make a careful estimate of the layer weight at the ditch. To ensure stability this estimate should not be too low.

Naturally the length perpendicular to the dike over which uplift occurs depends strongly on the duration of high water. For the analytical solution, reference may be made to (Ref.) *Basic set-up of the technical report on water pressure*. As numeric calculations have indicated, the solution provided for the uplift length is too large (50% to 100% too high). The analytical method is a useful method for providing a first impression.

4.5.4 Modelling

The modelling of groundwater flows is primarily based on the water-bearing layers. The models mentioned in this subsection therefore relate to the water-bearing layers. The results of the calculation from these models provide the basis for the determination of pressure flow in the weak layer package.

Modelling and reality

The extent to which the result of a calculation agrees with reality is highly dependent on the calculation model used, the schematization and the accuracy of the parameters involved in the calculation. In principle, groundwater flows are a three-dimensional phenomenon. In those cases in which there are no major variations in geometry, soil parameters and boundary conditions in one particular direction, it is possible to use a two-dimensional image. However the way in which these representations are drawn, depends entirely on the problem in question. In virtually all cases two-dimensional models are adequate to solve groundwater flow problems for dikes

□

Analytical models□

Simple analytical models often provide a reasonable representation of groundwater movement. However the condition for their use is that it must be possible to solve the differential equation that describes the flow with the given boundary conditions for an analytical method. A disadvantage of the analytical solution therefore is that a clear representation of the geometry of the problem area in question is required to obtain an accurate solution. The major advantage of the analytical model however, is the qualitatively good insight that this model provides with respect to flow behaviour and the influence on the flow of various parameters

Numeric models

The major limitation of analytical models lies in the above-mentioned fact that accurate modelling is required. This disadvantage applies far less to numeric models. These can be used for multi-layer systems and dike profiles consisting of various types of soil, both stationary and non-stationary.

Possible numeric calculation models include:

- Finite differential method, explicit or implicit
- Finite elements method
- Analytical function method
- Limit integral method

Graphic approach

In two-dimensional, quasi-stationary groundwater flows through an isotropically permeable soil mass, the flow model can be approached by using the characteristic that flow lines and potential lines form an orthogonal system, the square net method. Many different models are

available, each with individual advantages and disadvantages. Consequently, in practice, a combination of models is often used. The methods most frequently used in practice are indicated in § 4.6.2.

4.5.5 Piezometric gauge observations and pressure flow measurements

Piezometric determination of hydraulic heads in the sand substratum and pressure flow measurements in the ground layers with poor permeability can provide an important source of information in for use in determining the geo-hydrological systems of a dike. Besides direct extrapolation to design conditions, observations may also be used to calibrate calculation models to realistic conditions.

Piezometric gauges are highly suitable for hydraulic head measurements in the sandy substratum. However, they are not suitable for measuring pressure flow in clay. Retention in the tube radically influences the flow model. Special water pressure measuring devices are available for measuring pressures flows in clay. The interpretation of observations demands some knowledge if one wants to extrapolate to design conditions.

When calibrating, therefore, and with the subsequent extrapolation of the model, the following influences should be taken into account:

- The influence of reaching the limit potential head in a hydraulic soil breach, or when a layer is uplifted
- The influence of foreland flooding
- The influence of retention
- The influence of the stationary or non-stationary character in changes in outer water level
- When extrapolating, account should be taken of possible changes in model parameters in the future (e.g. excavation of holes in foreland, dredging activities and so forth outside the management zone, the discontinuation of water extraction behind the dike.)
- The influence of soil formation (biological factors, etc.)

Extrapolation can be done in two ways: by assuming upper limits or by assuming momentary piezometric gauge levels / water pressure. For further information, reference should be made to the (Ref.) *Basic set-up of the technical report on water pressure*.

4.5.6 Pressure flow in water-bearing packages

The following steps are taken when determining pressure flow development under design conditions:

- Depending on the shape of the channel (water-bearing beds), a model is made of the geometry of the dike and the substratum. The channel could be modelled as follows (see § 4.6.3):
 - two-dimensional in a horizontal plane
 - two-dimensional in a vertical plane
 - three-dimensional
- Depending on the time-dependent development of the hydraulic boundary conditions, it is possible to decide whether a stationary calculation is sufficient or whether a time-dependent (cyclic) calculation would be preferable.
- The decision as to whether the model must be calibrated to actual measurements (time-dependent or otherwise), or whether a conservative estimate on the basis of a careful estimate of the hydrological conditions suffices must be partly based on the above-mentioned choices.
- Depending on the intended use of the model (e.g. extrapolation to high water), it is necessary to decide how uncertainty (now and in the future) can be included in the geological modelling.

For river dikes in the Upper River area, for example, it can be said that:

- In the event of rising outer water levels in sandy dike profiles, in the sandy substratum and with covering layers that are not too thick, the flow is stationary

- In the event of a falling water level a possible lag in groundwater movement in the sandy substratum must be taken into account

An accurate, geo-hydrological model sets conditions for management and future maintenance. If the geo-hydrological model includes the assumptions associated with the latest flow model, one must anticipate changing conditions in the future. Since the geo-hydrological zone of influence can be very large, activities a great distance away can influence the flow model in and under the water defence. This is also true of archive investigations: if geohydrological conditions have changed, historic measurements are not necessarily useable. For these reasons the choice of geo-hydrological model is an important basis for extrapolation to high water.

4.5.7 Pressure flow in the weak layer package

Pressure flow in the covering clay and peat beds is also important to the stability of a water defence. In general, these pore pressures are determined by the location of the phreatic line, the potential in the underlying water-bearing beds and occasionally, by induced (horizontal) deformation. If the groundwater flow in the weak layer is considered to be quasi-stationary and the vertical permeability in the beds is constant, linear development of pressure flow over that depth can be assumed. Development is generally not hydrostatic, but is in line with the annual average potential in the water-bearing strata. Variations in potential lead to variations in pressure flow on the underside of the weak layers.

As a result of the thickness of the weak layers covered by the water-bearing sandy layer, the groundwater flows are non-stationary when loads are of relatively short duration as a result of high water waves and tidal influences. The time required by the stationary regime to adapt depends on the hydrodynamic period of the covering series, and is generally far greater than the duration of the high water level under normative conditions.

Penetration length

The height over which the pressure flow variations are important is called the penetration length. In stability calculations, it can be assumed that the pressure flow development over this penetration length is linear. The penetration length depends on the geo-hydrological parameters and the development of the water level. When determining the penetration length, reference should be made to the (Ref.) *Basic set-up of the technical report on water pressure*. For sudden changes in water level, the penetration length in most cases (depending on the consolidation coefficient and the duration of the water level variation) is between 0.5 and 3 metres.

Above the penetration length, under high pore pressures of relatively short duration, pressure flow remains the same in the semi-permeable strata under and behind the dike; this is the result of the low permeability and the compressibility of the weak layer. In order to calculate the penetration length in the covering clay and peat layer, it is necessary to calculate the consolidation coefficient of the strata in question.

Uplift or heave of covering layers

When there is uplift or heave of covering layers, including thick layers of little permeability, there is a stationary upward groundwater flow. The pressure flow distribution in the layer is linear, from zero at ground level to the pressure flow value in the sandy substratum on the underside of the layer (see also § 5.6.3).

4.6 Pressure flow under design conditions

4.6.1 Introduction

In stability calculations, the location of the phreatic line and pressure flow in the dike and substratum play an important role. Design values for the geometric, soil-mechanical and hydrological parameters are introduced into the calculations so the calculated hydraulic head and the pore pressure measured may be seen as design values. Thus, the penetration

length for outer dike revetments in the Upper River area cannot be greater than 50 to 100 metres. However, if erosion can be reasonably excluded, greater penetration lengths can be used for broader forelands outer values.

This subsection describes three loading scenarios in greater detail, the main objective being to determine design values for pore pressure. A distinction is made between the causes of the water pressure, these being:

- Geo-hydrology (normative high water levels and extreme precipitation)
- Response to increase in soil pressure (activities to raise the height of the dike in the implementation phase, induced horizontal deformations).

In the event of high water or heavy precipitation, pore pressure is linked to the geo-hydrological situation; therefore geo-hydrological modelling is the basis for determining pore pressure. During construction, pore pressure is the result of increases in soil pressure, which directly lead to an increase in pore pressure in the weaker beds (consolidation process).

Combinations of design loading are not covered here. This means that it is assumed that the design water levels and extreme precipitation will not coincide.

4.6.2 High water

The following aspects are of importance under high water conditions:

- Normative High Water level (NHW)
- Height of the phreatic plane

Pore pressure in the water-bearing layers during NHW can be determined with the aid of models. If a rise in phreatic line should occur in the dike body under 'normal conditions', this increase should also be taken into account under high water conditions.

The models listed in Table 4.6.1 are used frequently in practice. Since each model has different properties and applications, it is essential to choose the correct model for each case. To determine the pore pressure it is often necessary to use a combination of models.

Table 4.6.1 Models used to assess water bearing layers NHW

	Geometry	Hydr. Rvw	Type	Application
Watex	Quasi 2D in vertical plane	Non-stationary	Analytical	Water-bearing beds
Micro-fem	Quasi 3D	Non-stationary	Numeric	Water-bearing beds
Seepw	2D	Non-stationary	Numeric	Phreatic line or pressure meter
Mseep	Quasi 3D	Stationary	Numeric	Phreatic line or pressure flow

The WATEX model is an analytical, time-dependent model. Micro-fem is a numeric model, in which boundary conditions can be introduced time-independently. WATEX and Micro-fem are used to predict the hydraulic head in aquifers. When this is done, the vertical hydraulic head within a water-bearing package is assumed to be constant.

Mseep and Seepw are numeric models that are frequently used to determine the phreatic line. Seepw also takes into account elastic retention. For information on the use of measurements in extrapolating to design loading reference may be made to Annex 1.

Pressure flow under high water conditions is calculated on the basis of the determination of the hydraulic head in the water-bearing beds, and the average pressure flow in the weak layers. For further details on the method used reference may be made to § 4.5.6.

4.6.3 Extreme precipitation

In addition to the influence of design level on pressure flow, the influence of extreme precipitation should also be taken into account in a balanced review of the conditions. Long-term measurements in the crown and toe of a dike provide a good model of the effect of precipitation. Subsection 4.6.2 describes a number of the models that can be used for this. Extrapolation of the relationship between the amount of precipitation and the increase in pressure flow allows the prediction of the consequences of extreme precipitation. In addition to pressure flow, the amount of precipitation in a particular wet period is measured at the location in question. In general, the following conditions will occur:

- The phreatic line at the point of the inner crown line and the inner slope of the clay dike rises by 0.5 to 1.0 metres above the phreatic line under normal conditions
- The water level in the polder rises to the average ground level height at the lowest points in the polder. This assumes that drainage of the polder ceases
- The influence of the rise in the phreatic line on the pressure flow in deeper weak layers decreases with depth

The effect of extreme precipitation has not yet been investigated in depth.

4.6.4 Outward stability

Factors that determine the stability of the outer slope of a dike include an unfavourable combination of high groundwater level in the dike after a rapid drop in water level in front of the dike, or the conditions under extreme precipitation.

For the dikes in the Upper Lower and river areas, this situation occurs when there is falling pore pressure and a delayed pressure flow model in the dike. For the Lower River area, it is recommended that a situation after a 1-day drop be assumed while for the Upper River area, it is recommended that a situation after a 10-day drop be assumed. For sea and lake dikes, a drop to Average low Water level, or TL for a period of half the storm duration ($1/2 t$), as indicated in the Guide on Sea and Lake dikes. Storm duration, t_s varies between 35 and 45 hours.

4.6.5 Activities to increase the height during the implementation phase

In principle, water overpressure in the construction phase has little to do with the geo-hydrological description, however, the geo-hydrological description does represent a boundary condition, on which water overpressure as a result of activities to increase the height can be based.

When increasing the height for dike construction or dike reinforcement, the decrease in strength is primarily determined by the increase in pore pressure. Theoretically, owing to the consolidation process, the pore pressure will decrease over time (§ 4.3.8), with a resulting increase in the effective pressure (or strength). However the pore pressure can also increase owing to precipitation or deformation, especially in structures underlain by very weak material. If the initial strength is too little to bear the total addition, phased implementation will be required, so that use can be made of the gradual increase in effective pressure in the substratum, and thereby of the strength. A summary of constructional aspects is provided in Chapter 10.

5 Failure mechanisms

5.1 Introduction

In the (Ref.) *Fundamentals of water retaining structures*, section 7.3 provides a summary of failure mechanisms for water retaining soil structures. This chapter discusses these mechanisms in detail and also indicates which calculation methods can be used in designing a dike. A distinction is made between design rules and advanced calculation methods. The design rules are covered in detail; while for the advanced calculation methods, the area of application and the additional details are described only with respect to the design rules. Where necessary, the difference between testing and designing a dike is indicated. Because the calculation methods for designing and testing are the same in principle, the chapter is based on the design of a dike. However, for some mechanisms, time may affect the mechanical behaviour of the dike. For the testing itself, reference may be made to the *Safety Monitoring Guidelines*.

This chapter covers the following subjects:

- Crown height
- Inward and outward macro-stability of the water defence
- Micro-stability
- Sand bearing springs
- Forebank stability

For each subject attention is paid to methods to make improvements. At the end of this chapter the designer should be able to make a preliminary design, after which the design can be assessed against the programme of requirements and optimized, possibly with the aid of Chapter 6.

In the assessment of the strength or the stability of earth structures the following aspects are important:

- The loading on the earth structures (Chapter 3);
- The strength properties of earth (§ 5.4.3);
- The calculation model ascertain whether the design remains within or exceeds the failure boundary condition (Chapter 5);
- The geometry of the earth structure (Chapter 6)

Various methods can be used to assess designed or existing earth structures. The classic deterministic method is still used. However a new method that which is based on the probability of inundation is being developed. This method is described in the (Ref.) *Fundamentals of water retaining structures*.

5.2 Crown height

5.2.1 Introduction

When determining construction height of earth structures, in addition to overflow and overtopping mechanisms, settlement and the expected deformation of the soil as a result of creep also play important roles. The construction height is the height of the crown immediately after completion of the construction or improvement of the defence.

Subsection 5.2.2 covers the height of the crown in a dike profile. Subsections 5.2.3 and 5.2.4 cover settlement and horizontal deformation and § 5.2.5 relates to crown construction height of special constructions.

5.2.2 Crown construction height

The construction height of the crown is determined consideration of normative high water levels (§ 3.2.2), increase in high water level, (§ 5.3.3.4) local increases in water levels (§ 5.3.3.5) and wave overtopping (§ 5.3.3.6) on the one hand and, on the other hand, on the strength properties of the soil (§ 5.2.3). Moreover the geometry of the outer slope and time are also relevant factors in the calculation of the construction height. More information about this can be found in Section 5.2 of *Guidelines on sea and lake dikes*, Chapter 6 of *Guidelines for the design of river dikes Part 2* and Section 8.1 of *Guidelines for the design of river dikes, Part 1*.

5.2.3 Settlement

As a result of the construction of a dike or an increase in the height of a water defence, two types of deformation may occur:

- Settlement (vertical deformation) as a result of settlement in the substratum and settling of the material used to raise the height of the dike
- Horizontal deformation in, under and next to the dike. These deformations lead to loading of structures in and near the water defence such as pipelines and the foundations of buildings (§ 5.2.4).

Settlement can be seen as a failure mechanism if it is considered that, in the event of overly large settlement of the crown, overflow or greater overtopping may develop than that for which the inner slope was designed. If the horizontal deformations are expected to remain limited, calculating settlement by using the method as described in § 4.3 is recommended. The compression constants required can be determined by using compression tests. With regard to the choice of the design values the following should be noted :

Crown

A decrease in crown height for a particular dike section must not be seen as an average decrease, but rather as a maximum for a certain number of points, because no point may fall below dike table height. As a result, the calculations should be performed by taking the characteristic values for the compression coefficients and the associated material factors.

If the settlement is observed during the construction period by using a settlement gauge, so that it is possible to adjust the settlement allowance calculated during the design phase during the construction phase, the average arithmetic value of the compression constant can be used. The dike profile must survive for a maintenance period of 50 years. Since settlement is a time-dependent process, the settlement calculations should be brought in line with this. Total settlement is calculated; that is the primary settlement that occurs during the hydrodynamic period, plus the secondary settlement over a period of 50 years (see § 4.3.8). The additional weight of the freeboard required to compensate for the predicted settlement must be included in the calculations.

Part of the settlement occurs during implementation. Depending on the duration of the consolidation (the hydrodynamic period) and the phasing of the construction and any soil improvement techniques applied, during the design phase a settlement value can be given.

Banks and berms

Besides settlement of the crown, it is also important to obtain a good picture of the anticipated settlement of other parts in the dike profile such as slopes and berms. When constructing a new dike on a virgin, level substratum, the problem is that the increased load on the substratum for each point in the transverse profile is different. For dike improvements, the situation is even more complex, since the new profile is laid next to and partially over the old profile. In this case, both the load increase and the initial pressure conditions are different for each point in the transverse profile. This can be taken into account in the calculations.

The finite elements method provides a useful means to determine the two-dimensional behaviour of soil. Most programs available for practical situations use elasto-

plastic soil models. The use of a finite elements program demands experience and geo-technical knowledge with regard to choice of parameters, the modelling and the interpretation of results (see § 4.3).

Residual settlement

In addition to total settlement of the crown, it is also important to have some insight into of the expected residual settlement. Residual settlement is the settlement expected, including creep, after completion of the dike reinforcement. Residual settlement is particularly large if the construction period is short. Uneven settlement in the transverse profile is particularly undesirable (consider cleft width and clamp force between the hard elements of the revetment).

In certain cases, depending on the possibilities, application of vertical drainage (acceleration of the settlement process) or lengthening the construction period can be considered. This aspect may even influence the choice of the alignment. In this context it is more difficult to achieve outward strengthening with an entirely new slope than inward strengthening whereby little or no settlement occurs in the position of the existing slope. The actual settlement may deviate from the calculated settlement. It is therefore useful to use settlement plates to continue following the settlement from the time that the increase in height is applied. It is also usual to re-predict the settlement just prior to applying the final increase in height, but this time on the basis of the information acquired from the settlement gauges.

To make an effective prediction of settlement prognosis, information on the development of pressure flow is also important in addition to the settlement gauge information. Chapter 11 covers the management of the construction in greater detail.

Settling

The potential settling of freshly applied soil can only be estimated. For the settling of clay, a value 10% of the increase in height is recommended, but with careful implementation and densification, this can be reduced to 5%. Far higher percentages should be assumed when using unripened clay. The use of clay that is not entirely ripe is considered very ill advised because serious split may develop in such clay (*ref. Technical Report Demands of Clay for Dikes*). A value of 5% is generally advised for settling in sand fill; however, in the event of good density, this settling can be neglected, certainly when compared to the settling of the substratum. If sand is applied to the structure by means of spraying (and is not subjected to further disturbance), a settling percentage of a few percent should be assumed.

Decline in the soil level

Decline in the soil level occurs as a result of the oxidation of organic components, for example, in peat. A special type of vertical deformation includes fall in soil levels as a result of water extraction (adaptation to polder level), or the extraction of minerals. The consequences of this may of such dimensions that, if this subsidence can be predicted, it is necessary to take it into account when setting the construction height of a dike body. A concrete example of this is the decline in soil level in the Province of Groningen. Here natural gas has been extracted since 1963, affecting (by decline in soil level) the soil up to and under the sea dike. Based on the most recent gas extraction forecasts for the Groningen field, by 2050 the reduction in soil level in the middle of the field will reach some 40 centimetres. At the locations of the sea defences, for example in Delfszijl, this will mean a drop of some 30 cm.

Prognoses are updated and made public on the basis of measurements every five years. The prognoses do not take into account other forms of ground level decline. If, for example, a polder level is lowered to compensate for the consequences of soil level decline for the reclamation of agricultural areas, the additional decline that results from this activity is not taken into account. Although this is usually of little consequence, it should be noted that declines in soil levels also influences the stability of the dike body under normative conditions.

5.2.4 Horizontal deformation

The construction or increase in height of an existing soil structure not only influences settlement of the substratum, but also has effects on horizontal soil movement in, under and next to the dike. It should be noted that horizontal deformation around the IJsselmeer, in the Upper River area and along the seacoast, plays a less important role than in the Lower River area, where the clay and peat series are generally thicker and weaker. There is also less construction in the direct vicinity of these flood defences.

The literature provides various analytical relationships for horizontal deformation under linear-elastic conditions with strip-like loading as induced by a dike body on the substratum. However the recommendation is to calculate the horizontal deformation with the aid of programs based on elements, in which the elasto-plastic behaviour of the soil has been included (see § 4.3). Horizontal displacement may influence the structures and objects already present. This is covered in greater detail in Chapter 8.

5.2.5 Crown construction height in special structures

Translating the overtopping capacity into a construction height can be extremely complex for special structures. Chapter 3 provides some information on how to achieve this for slopes and, to a limited extent, for vertical walls. Further information can be found in (Ref.). *Guidelines on Water-retaining hydraulic structures and special structures*). In many cases, specialist assistance and model investigation will be required.

5.3 Macro-stability of the water defence

5.3.1 Introduction

In this section, the stability of the soil structure is defined as: resistance against shearing of large sections of the soil structure along straight or curved slip planes, or loss of equilibrium as a result of the generation of major plastic zones. In other words, 'the ability to resist loading without loss of function as a result of excessive deformations'.

The following information is required for the analysis of the macro-stability:

- Geometry, that is, the measured transverse profile of the water defence
- Layer structure of the substratum and the dike body
- Weight by mass and the representative strength characteristics for each layer
- Loading: the normative location of the phreatic line and the pressure flow development in the substratum

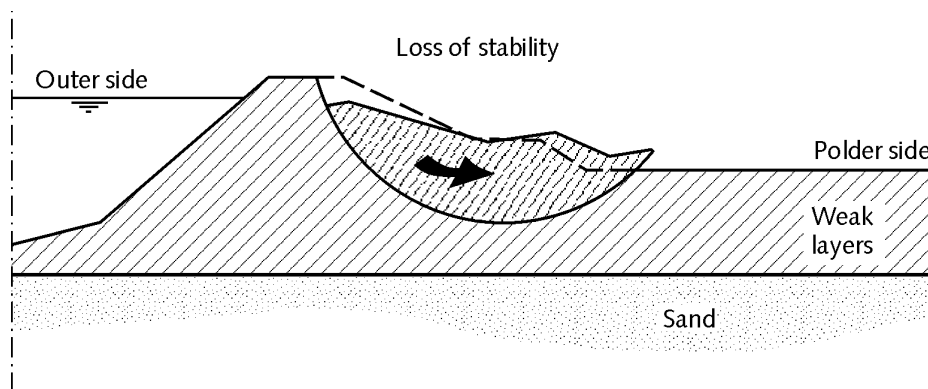


Figure 5.3.1 Shearing on the inner side

The strength characteristics and pore pressure in and under the dike determine the resistance to shearing (See § 4.3, § 4.5 and § 4.6). This chapter covers the methods by which strength and loading can be weighed against each other when investigating the stability of

a water defence: where necessary, references are made to the standard literature list for further details. Subsection 5.3.2 provides a more general view, which is followed by consideration of the methods to be applied in subsections § 5.3.3, § 5.3.4 and §5.3.5.

5.3.2 Further consideration on macro-stability

General

Macro-stability relates to the resistance to shearing of major parts of a soil body. Shearing occurs along straight or curved slip planes or through plastic zones where, as a result of overloading, there is no longer an equilibrium of forces. The degree of deformation plays an important role, owing to the often major differences in stiffness between the dike material and the underlying substratum on the one hand, and the soil layers behind the dike on the other.

There are two possible ways to assess stability:

- a) slip plane calculations
- b) finite elements method

Re a):

In practice, slip plane calculations are often used for a more or less routine assessment of existing or newly designed dike profiles. Under normal conditions, this approach provides a sufficiently reliable result and, owing to the relative simplicity of the method of calculation compared to the finite elements method, it is a very attractive method.

Re b):

In the finite elements method, the pressure conditions must be determined for each point in the soil. As a result of the statically undetermined character of soil structures, deformations of the soil must also be taken into account. The pressure-deformation relationships of the soil must therefore be known. The advantage of this approach is that the calculation itself indicates the area where the potential slip plane develops, as well as giving a fairly comprehensive picture of the pressures and deformations in the soil mass.

Loading scenarios

There are several loading scenarios that may affect the macro-stability, as shown in the flowchart in Figure 5.3. Both the stability of the inner slope and the stability of the outer slope must to be taken into account. Another differentiation that must be made is that between stability in the construction phase and the stability in the end phase.

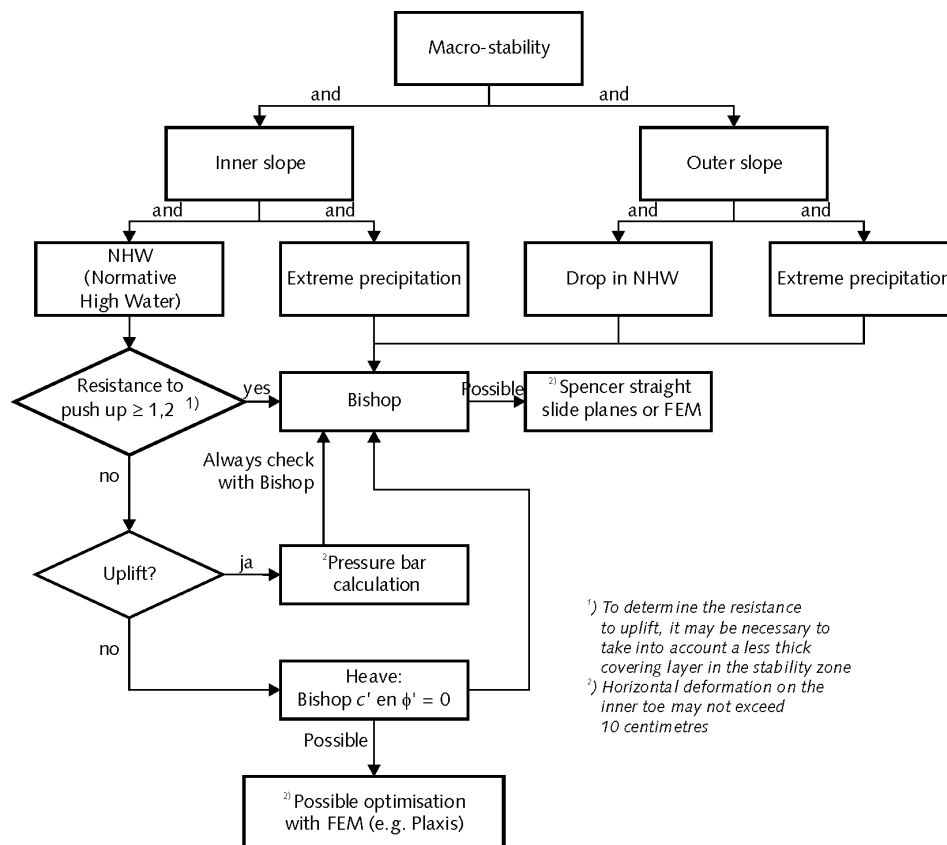


Figure 5.3.2 Flowchart for the assessment of macro-stability

Stability inner slope

For the stability of the inner slope, either the conditions under normative high water, or the conditions under extreme precipitation are determinative. Section 4.5 shows that the time-dependent character of the groundwater flow can be taken into account. The water level development under normative conditions can be used as a boundary condition here, as described in § 3.3.3. This can have a major impact, in particular along the coast and in the Lower River area. When considering the macro-stability of the inner slope under normative conditions, it is necessary to make a distinction on the basis of whether or not covering clay and peat layers are pushed up under the influence of high water potentials in the water-bearing series (see stability in uplift and heave in § 5.3.3 and § 5.6.3).

□

Stability outer slope

The stability of the outer slope is adversely affected by the factors such as an unfavourable combination of high groundwater level in the dike under rapidly falling water levels in front of the dike, or conditions arising under extreme precipitation. The relationship between the macro-stability of the outer slope on the one hand and the settlement flow and shearing of the forebank on the other, are covered in § 5.7. Reference should be made to § 4.5.4 for the pore pressure conditions.

Stability during implementation

In addition to the assessment of a dike reinforcement design for the fully consolidated final situation, stability during the various implementation phases must also be calculated. This is important for two reasons. Firstly it is necessary to ascertain whether there is sufficient guarantee of safety with respect to collapse as a result of instability in the construction phase. Secondly it is necessary to determine whether the target design can be realized from a technical perspective, and at what rate the various layers required to heighten the dike can be applied. Implementation stability is covered in further detail in § 10.5 and § 10.6. If

should be noted that the end situation, 100% consolidation, is only achieved after a long period of time. The water defence is therefore less safe for a considerable time.

5.3.3 Slip plane calculations

General

The slip plane calculations (analytical methods) do not give a direct indication of the potentially most dangerous slip plane. The slip plane is included in the calculations as a boundary condition. From the results of the calculations using a number of slip planes it is possible to determine the most dangerous slip plane. This is the plane with the lowest resistance to shearing.

There are various methods of calculation, distinguished according to the shape of the slip planes in question and the shear tension distribution that need to be taken into account along the slip plane. Most of these methods assume circular slip planes. In general, the calculation methods are two-dimensional; perpendicular to the cross-section, the slip plane is assumed to be of infinite length. These methods are often used in practice. CUR 162 (ref. *Construction with soil*) describes how to calculate the macro-stability of soil structures using circular slip planes according to the Bishop method.

Situations may develop in which shearing along a circular slip plane is not the least favourable situation. If the substratum contains a thin layer with relatively poor strength characteristics, stability checks will have to assess the risks of both shearing along circular slip planes, and of shearing along a more or less horizontal slip plane through the relatively weak layer. This situation may also occur if the weak sealing layers lift up as a result of a high hydraulic head in the water-bearing series. Although in most cases, depending on uplift safety (<1.20), the latter effect is negated by choosing a non-circular slip plane calculation model, it should be noted that non-circular slip planes can be normative under higher uplift safety. Examples of this approach include dike or berm ditches with a steep slope (see §5.6.3).

Compared with respect to circular slip plane calculations, the major difference is that the horizontal equilibrium is considered instead of the moment equilibrium. This is called the Spencer method and is described in CUR 162 (*Construction with Soil*). As in the Bishop method, the slip plane is a boundary condition that is included in the calculation.

It should be noted that the deformations along the slip plane are the same throughout in this analytical slip plane approach. For heterogeneous soil profiles with major differences in stiffness, the actual shear tensions that are mobilized are not the same everywhere. In many cases, this will mean that it is necessary to use the residual shear tension of the relatively stiff material in and under the dike (active zone) in calculations. An alternative is the ADP method, although there is little experience with this.

Soil behaviour in shearing

There are three analytical methods to describe soil behaviour in shearing:

1. Drained pressure analysis
2. Undrained total pressure analysis (on the basis of the C_u method)
3. Undrained effective pressure analysis on the basis of measured or predicted water pressure or overpressure

Re1:

A drained analysis is valid in those situations in which both the dike body and the substratum are of highly permeable soil, in which water overpressure does not develop as a result of loading. This means that undrained analysis may also be used in the assessment of dike structures on poorly permeable soil on condition that the substratum is entirely consolidated (end situation) and does not develop water overpressure under extreme conditions. The drained cohesion (c') and the drained angle of internal friction (ϕ') are used for shear strength.

Re 2

Undrained analyses are used in locations with soil that is not very permeable. In such soils, as a result of loading water overpressure develops and there is little or no consolidation during

the construction period. The undrained shear strength (f_{undr} or C_u) is determined before the occurrence or application of a change in loading. The methods used to determine the undrained shear strength is described in § 4.3. It should be noted that in the finite elements method a C_u analysis which uses the actual values of C_u could be performed.

Re 3

An undrained analysis can be seen as a drained analysis combined with introduced water overpressure. In this analysis, drained strength properties (c' and ϕ') are introduced in combination with measured or predicted water overpressures.

The water overpressure resulting from the loading can be expressed as adaptation percentages for the various layers. Since no account is taken of pressure distribution, this results in values that are too high under the raised area, and too low next to the raised area. However in spite of this, in practice, on average this method provides good measurements. It should be noted that this soil behaviour could also be applied in the finite elements method. In such an analysis, drained strength characteristics c' and ϕ' are introduced. After this an undrained calculation is made (no run-off of overpressure-pore water). Changes in time can be deduced by using the 'consolidation' option.

Stability in uplift and heave

When considering the macro-stability of the inner slope under conditions with normative high water, it is necessary to investigate whether or not covering clay and peat layers have been lifted up as a result of high potentials in an underlying, water-bearing sand layer. If uplift has to be taken into account, (if resistance to uplift is less than 1.2 see also § 5.6.3), a distinction should be made between:

- a) Uplift (Figure 5.3.3): In uplift, a situation may develop in which the entire covering series starts lifting.
- b) Heave (Figure 5.3.4): In heave, a situation may arise in which the layer is damaged as a result of overpressure (heave). Heave occurs if the weak layer package is relatively thin.

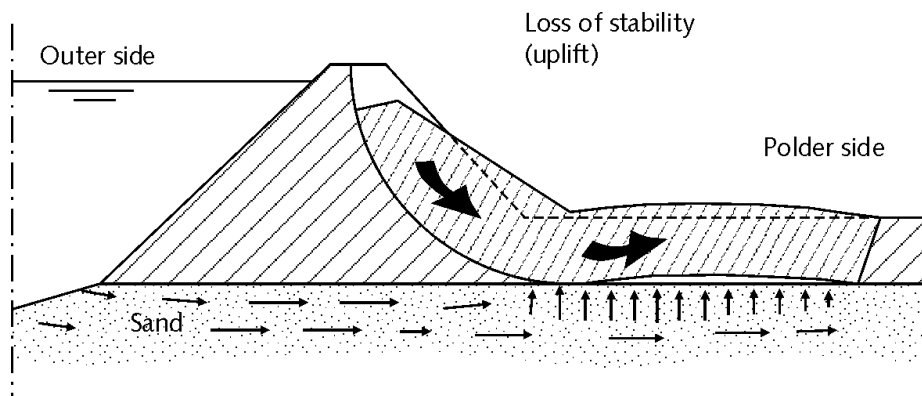


Figure 5.3.3 Loss of stability (uplift)

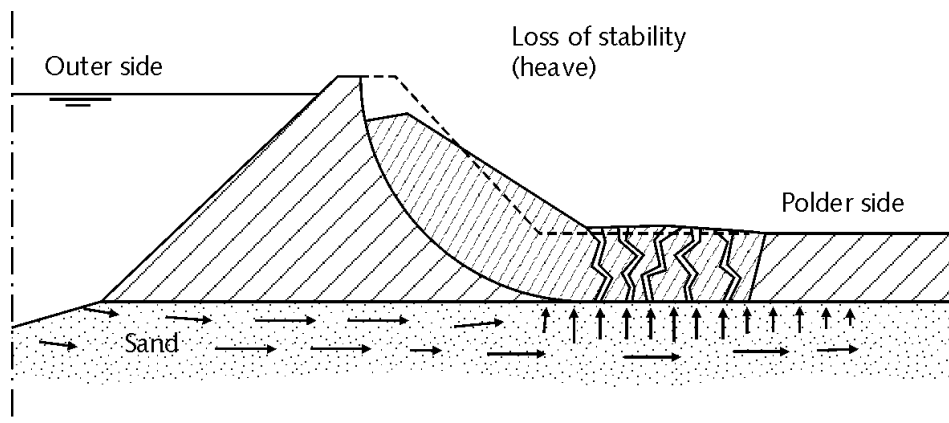


Figure 5.3.4 Loss of stability (heave)

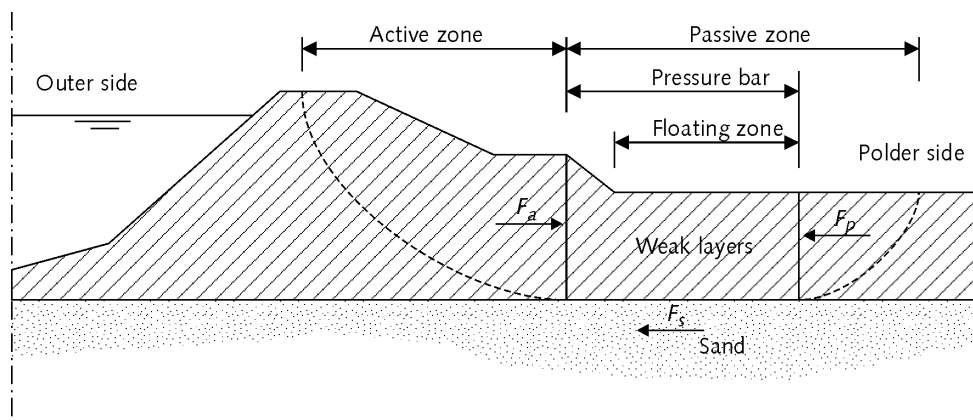


Figure 5.3.5 Forces taken into consideration in the 'pressure bar' method

Re a) This describes a situation with uplift (see Figure 5.3.5). In a normal situation with low water levels, the dike body is supported to a certain degree by the soil series behind the dike. With high water, this soil layer package will lift up. The shear resistance in the active zone, as residual shear strength, will then be almost entirely mobilized. In the passive zone, the horizontal force exerted by the potential shearing soil layer F_a is transferred to the Pleistocene sand by shear pressure F_s on the one hand and, on the other hand, is counteracted by horizontal soil pressure F_p outside the uplift zone. Under high water conditions, the upward potential in the sand series increases and reduces the effective stress on the dividing plane between the weak layers and sand, and after a certain period of time also in the lowest zone of the weak layers. As a result, the maximum shear resistance that can be mobilized in this zone, F_s , decreases.

In local uplift, the resistance of the dividing plane is reduced to zero: the soil layer behind the dike that is lifted up behaves like a weak 'press bar', as the horizontal shear pressure can no longer be transferred to the sandy substratum. Additional horizontal pressure will have to be mobilized in the area behind the uplift zone. This category includes the situation in which uplift may not yet be relevant, but in which there is a considerable reduction in shear resistance between the weak layer package and the deep sand as a result of an increase in potential.

Annex D of (Ref. Guide on Designing River Dikes, part 2, Annexes) explains how the 'Improved Pressure Bar Method' can be used to calculate stability in this situation. This

method is still under improvement. The *Guide on Structural Design* indicates how the Spencer method is used in calculating the stability in uplift.

Re b) Stability in heave

If heave occurs in the clay and/or peat layers located on the inner side, it can be assumed that the strength of the heaved layer has been seriously impaired. A normal Bishop analysis using $c = 0$ and $\phi = 0$ in the uplift zone is a safe approach. In this case on the passive side, only the eigen weight of the layers as opposing force (see Figure 5.3.6) is taken into account. The shear strength in the passive part of the slip circle is equal to zero. This approach gives a safe, yet somewhat oversized design.

Deformation calculation

In situations requiring accurate calculations, the strength contribution of the uplifted 'pressure zone' to the stability of the water defence should also be taken into account. In such cases, however, in addition to equilibrium, deformation must be included. This is indicated in Annex D with (Ref. *Guide on Designing River Dikes, part 2, Annexes*). A stability analysis using a finite elements method (Plaxis, for example) provides a relatively full indication of the pressures and deformation in the soil mass (see § 5.3.4).

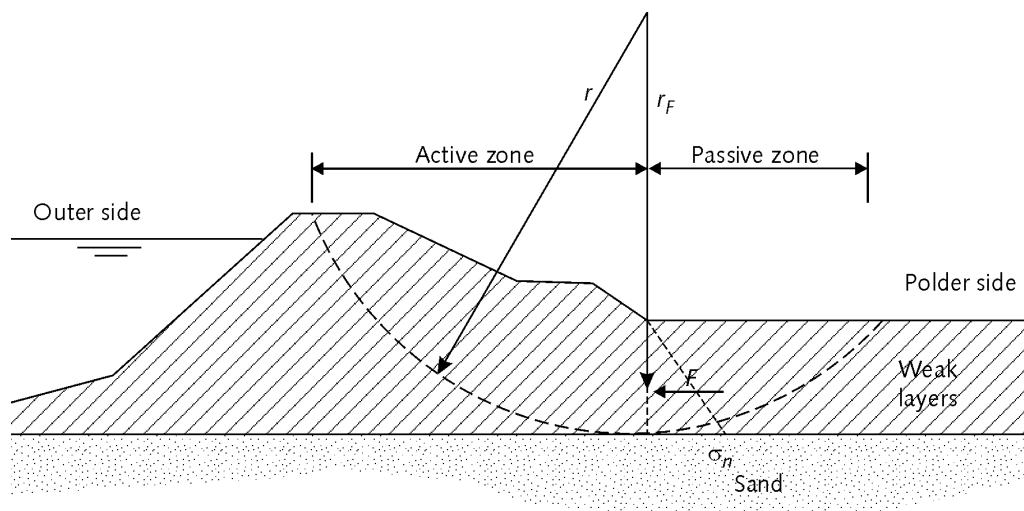


Figure 5.3.6 Diagram for heave: Bishop with $c' = 0$ and $\Phi = 0$ in passive zone

End phase

As mentioned above, if the differences in stiffness in, under and next to the dike are large, or when the weak layer series behind the dike is lifted up, or almost lifted up, an adapted stability approach should be considered. In these cases, the soil outside the toe of the dike must undergo a relatively large amount of deformation in order to build up resistance, for example to the changes in loading resulting from extreme high water. The approach described below therefore concentrates on the deformations in a dike. The calculation can be refined step by step, depending on the optimization required.

First a calculation using the neutral soil pressure behind the berm is used. This assumes equilibrium without deformation behind the dike. If the resulting stability is insufficient, a calculation is made using passive soil pressure. If this calculation indicates that no equilibrium is possible with passive soil pressure, calculating the deformation is pointless; the design will have to be adapted by, for example, increasing the size of the berm. If there is equilibrium with passive soil pressure, a deformation calculation is performed, based on an approach of the deformation according to the press bar principle. A practical magnitude for the maximum deformation on the inner toe may never exceed 0.10 metres.

If the deformations exceed the design values the design will also have to be adapted, for example by making the berm higher or wider. The adapted calculation is shown by means of a flow chart in Figure 5.3.7.

Design

When designing a dike for a location where uplift plays a role, it is natural to compensate for the expected loss in shear strength along the contact plane between the covering layer and the deep sand by adding the weight of a sufficiently wide and heavy berm. From a structural perspective, the occurrence of a gradient that is critical for soil collapse is not in itself a reason for the construction of a berm. Under normative conditions a dike may experience uplift or heave while the macro-stability is assured.

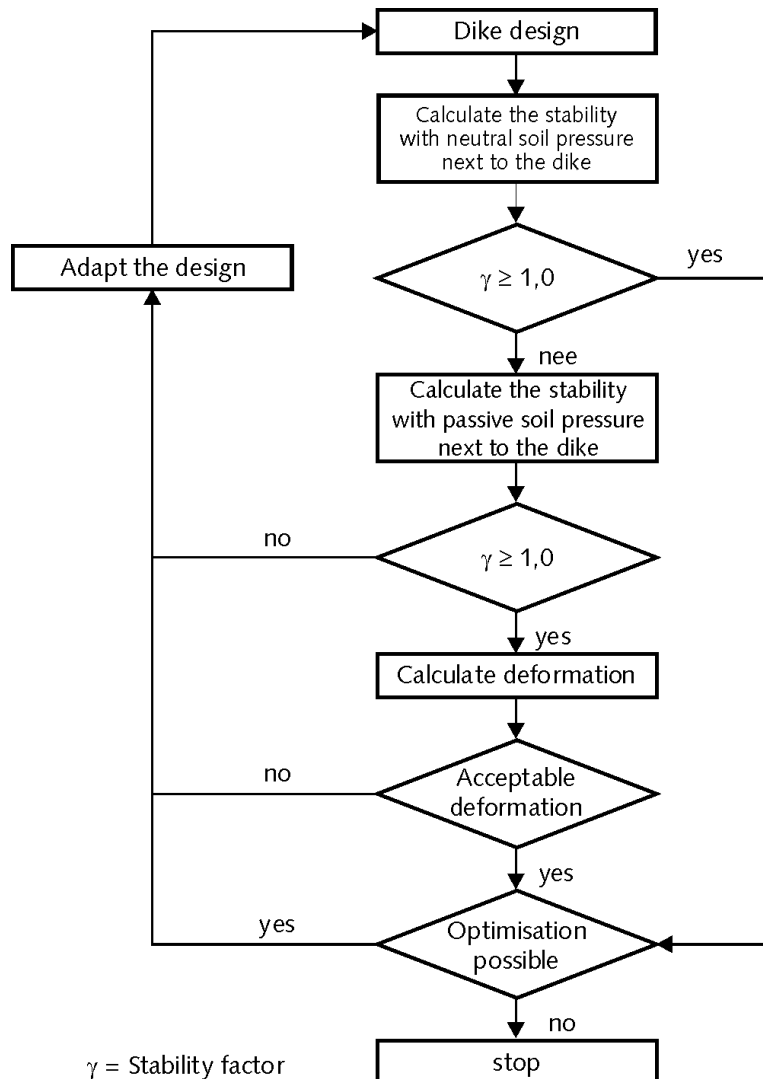


Figure 5.3.7 Flowchart for the calculation of uplift

5.3.4 Finite elements calculations

As indicated earlier, the advantage of a stability analysis using the finite elements method is that the potential slip circle is not set beforehand. It also provides relatively clear insight into the pressures and deformations of the substratum. A finite elements calculation is useful if it is not possible to obtain a sufficiently clear picture with a simpler calculation:

- If, for example, a soil layer of limited thickness that has poor strength characteristics (c and ϕ) is present, stability calculations will require not only assessments of shearing along

circular slip circles, but also of shearing along a more or less horizontal slip circle through the relevant relatively weak soil layer.

- This may also be necessary if, as a result of relatively high water overpressure, it can be expected that a circular slip circle is not the most unfavourable assumption.
- In a heterogeneous soil profile with major differences in stiffness, the mobilized shear stress is not the same everywhere. However, the deformations along the slip circle are the same everywhere. This is taken into account in a finite elements analysis.
- Performing a finite elements calculation is also recommended if the deformations in uplift have to be calculated in detail.

Over recent years, the possibilities of the finite elements method have increased greatly. As a result, the methods are also suitable for calculating stability in uplift conditions. Programs are available (Plaxis) for the application of a load in the weak layer series as a result of an increase in potential in the Pleistocene (water-bearing series). In calculations with a finite elements method, special attention has to be devoted to the way in which the soil structure, soil characteristics and pore pressure are represented in the model. The manner in which the eigen weight and loading is applied must be given the necessary attention in the calculation process.

There are a number of differences between methods used to calculate slip circles:

- If the stability is of interest, the Mohr-Colomb model will suffice, however if the deformations are important, the Softsoil Creep model is recommended (see § 4.3.7 and §. 4.3.8 for the available material models). Current experience is primarily based on a linear-elastic perfectly plastic model (Mohr-Coulomb).
- For a slip circle analysis, calculation values for the shear strength parameters are used (see § 5.3.5). To obtain realistic deformations, the use of a finite elements calculation with average values for strength parameters is recommended. This means that the testing value must be increased by multiplying by a factor which in principle, is the quotient of the average strength parameters and the values of the strength parameters. However the problem is that when the strength parameters are increased by 10%, for example, the stability factor calculated with the finite elements method is not 10% higher. This is still the subject of study. Since the determination of the factor in question is relatively complicated, when determining a stability factor it is preferable to use the values of the shear strength parameters. The deformations calculated in this way are therefore not realistic.
- The results of the finite elements calculation depend on the manner in which the calculation process takes place. Depending on the program used to implement the calculation it is necessary to select certain settings for the calculation parameters. At the moment (2000), stability in uplift can only be calculated by using the Plaxis elements method. This program is the only one that has the option (called c- ϕ -reduction) of determining the stability factor.
- Usually the calculation process entails the following phases:
 - Calculating the initial pressure situation resulting from the eigen weight of the structure; the drained application of the eigen weight of the soil layers and pore pressure. This is done for the current situation under normal conditions.
 - If a dike improvement is included in the calculation: the drained calculation of the pressure situation resulting from of the application of a dike improvement. When analyzing stability during construction, this phase can be seen as follows: firstly, the dike improvement is applied undrained, with or without phased stages of increases in height, and either consolidated or unconsolidated.
 - Calculation of the pressure situation resulting from an increase in the outer water level and the potential in the sand of the substratum (Pleistocene), as well as the adjustment/consolidation of this water overpressure in the clay/peat layers. Prior to this phase, the deformation can be set to zero in order to permit calculation of the deformation of the phase in question and in the following phase.
 - The undrained application of traffic loading.
 - The calculation of the stability factor by implementing a c- ϕ -reduction analysis.

□

Method used to calculate stability during implementation

The general method for determining the implementation phasing is as follows:

- a) A calculation is made to determine at what consolidation percentage the planned total increase in height gives a sufficiently high equilibrium factor.
- b) Once the hydrodynamic period has been calculated, it is possible to determine how long it will take to attain the full increase in height.
- c) If the period calculated in stage b) is too long, the effect of vertical drainage is investigated.
- d) Within the calculated total construction period, how the various phases can be most effectively distributed as effectively as possible over time can be established.

Re a) In practice, this means that for a number of consolidation percentages (for example, 30, 60 and 90%), the equilibrium factor of the entire increase in height is calculated. Interpolation to the desired equilibrium factor gives the required consolidation percentage (for example, 50%).

Re b) Using calculations in the soil-mechanics handbook, it is possible to deduce at what stage of the hydrodynamic period the required consolidation percentage is reached (for example, 50% adaptation is reached after 10% of the hydrodynamic period).

Re c) If according to the calculation the work cannot be completed in the available period, vertical drains are usually employed, in which case if necessary a smaller centre to centre distance than usual is accepted. To determine the effect of vertical drainage, it is necessary to possess insight into the horizontal k-value. These days, synthetic drains are almost always used. In exceptional cases, the design is changed on the basis of the constructional stability. Changes to the design may include increasing the height or the width of the berms or the use of lighter material to increase the height. If too many problems are encountered with respect to construction time, the design can be adapted by using alternative materials or structures. Chapter 9 provides a summary of possible solutions.

Re d) On the basis of calculations, the adaptation percentage required for the various phases in the work is determined, taking into account the safety level required. Next, the adaptation percentage is translated into a construction period and waiting times. The calculated adaptation is generally introduced as averaged values over the entire series; it is preferable, (as done in cases requiring optimization), to introduce the change as a function of the distance to the permeable edges.

5.3.5 Resistance to shearing: stability factors

In the deterministic safety approach a conservative assumption is used for the strength R and loading S. Then an overall-safety factor is demanded as a margin is between the calculated strength and the loading.

[EQUATION] 5.3.1

$$\gamma_0 = R/S \geq 1$$

in which:

R	average strength along the shear plane
S	driving moment (loading)
γ_0	stability factor based on average values

In the pressure bar calculation, stability factors $\gamma_{0, \text{pressure bar}}$ is defined as follows:

[EQUATION] 5.3.2

$$\gamma_{0, \text{drukstaaf}} = (F_p + F_s) / F_a \geq 1$$

in which:

F_a driving force from active slip plane

F_p opposing force along dividing plane F

F_s opposing force in hinterland – end of pressure bar

Partial stability factors

For the stability analyses, a semi-probabilistic method is recommended (Ref. *Fundamentals of water retaining structures*). As indicated above, the stability of a dike body against shearing is investigated by comparing loading and strength (see following equations). The maximum opposing moment that can be mobilized (or the opposing forces) and the driving moment (or the driving force) are calculated on the basis of calculation values. The variable that represents the strength is divided by γ_R so that a safe calculation value can be obtained for the strength. Conversely the variable that represents the loading is multiplied by γ_S . The following indicates how the TAW meets the ISO system of safety factors for the macro-stability mechanism. The system is similar for other failure mechanisms.

[EQUATION] 5.3.3

$$\gamma_S = 1 \text{ and } \gamma_R = \gamma_d \gamma_m \gamma_n$$

in which:

γ_d partial safety factor related to the model used (also known as the model factor)

γ_m partial safety factor relating to material parameters (also known as the material factor)

γ_n partial safety factor related to damage (also known as the damage factor)

γ_R strength safety factor

γ_S loading safety factor

A characteristic value for a material property is a value determined on the basis of statistical analyses for which there is a certain probability of exceedence and deficiency. It can be determined according to the method given in Annex 1.

For the Bishop method the stability factor (γ_{Bishop}):

EQUATION (5.3.4)

$$\gamma_{\text{Bishop}} = R_d / S_d \quad \text{or} \quad \gamma_{\text{Bishop}} = \gamma_0 / (\gamma_d \gamma_m \gamma_n)$$

in which

R_d Maximum opposing moment that can be mobilized calculated on the basis of calculation values ($R_d = R / \gamma_d$)

S_d Calculated value for the driving moment ($S_d = \gamma_d S$)

The stability value $\gamma_{\text{pressure bar}}$ for the press bar method, (see Figure 5.3.5) is:

[EQUATION] 5.3.5

$$\gamma_{\text{drukstaaf}} = (F_{p,d} + F_{s,d}) / F_{a,d} \quad \text{or} \quad \gamma_{\text{drukstaaf}} = \gamma_{0, \text{drukstaaf}} / (\gamma_d \gamma_m \gamma_n)$$

In which:

$F_{a,d}$	calculated value for the driving force from active slip plane
$F_{p,d}$	calculated value for the opposing force in hinterland = end pressure bar
$F_{s,d}$	Calculated value for the opposing force along the dividing plane

Assuming that the calculation was made using calculation values (the deformations are not then correct, see § 5.3.4), the stability factor (γ_{em}) is deduced from this as follows:

[EQUATION] 5.3.6

$$\gamma_{em} = R_d / S_d \quad \text{or} \quad \gamma_{em} = \gamma_0 / (\gamma_d \gamma_m \gamma_n)$$

In the finite elements method usually no stability factor is calculated. However the finite elements model Plaxis does have the option of determining the stability factor. This option is called c- ϕ -reduction.

Material factors

Material factor γ_m takes into account uncertainties in the description of the shear strength. These depend on the type of soil and also on the method of testing. The material factors used in current practice are given in Tables 5.3.1 and 5.3.2, which are taken from the (Ref.) *Guidelines for the design of river dikes, parts 1 (Upper Rivers) and 2 (Lower Rivers)*.

Table 5.3.1 Material factors for pressure independent approach¹

Material factors	γ_{m1}	γ_{m2}	$\gamma_m = \gamma_{m1} \cdot \gamma_{m2}$
Volumetric mass – wet/dry (ρ)	1.00	1.00	1.00
Internal friction ($\tan\phi$)			
• Sand (CP) ¹	1.00	1.15	1.15
(TP-CD)	1.00	1.15	1.15
• Clay (CP)	1.00	1.15	1.15
(TP-CU-5%)	1.05	1.15	1.12
(TP-CU)	1.10	1.15	1.25
• Peat (CP)	1.00	1.20	1.20
(TP-CU-5%)	1.05	1.20	1.25
(TP-CU)	1.10	1.20	1.30
Cohesion (C)	1.00	1.30	1.30
Compression constants			
• Terzaghi (CA)	1.00	1.10	1.10
• Buisman-Koppejan (c_p : c_s)	1.00	1.10	1.10

¹Source: *Guidelines for the design of river dikes Part 1 Upper Rivers*

Table 5.3.2 Material factors for pressure independent approach²

Material factors			
Volumetric mass – wet/dry (ρ)	1.00	1.00	1.00
Internal friction ($\tan\phi$)			
• Sand (CP) ¹	1.00	1.10	1.10
(TP-CD)	1.00	1.10	1.10
• Clay (CP)	1.00	1.10	1.10
(TP-CU-5%)	1.05	1.10	1.15
(TP-CU)	1.10	1.10	1.20
• Peat (CP)	1.00	1.15	1.15

(TP-CU-5%)	1.05	1.15	1.20
(TP-CU)	1.10	1.15	1.25
Cohesion (C)	1.00	1.25	1.25
Undrained strength (C)			
Slip modulus (G)	1.00	1.20	1.20
Compression constant	1.00	1.10	1.10
Permeability (K)	1.00	1.25	1.25
Consolidation (C)	1.00	1.25	1,25

²Source: *Guidelines for the design of river dikes Part 2 Lower Rivers*

¹)CP	=	Cell test
TP-CD	=	triaxial test, consolidated and drained
TP-CU	=	triaxial test, consolidated and undrained
TP-CU-5%	=	triaxial test, undrained, with maximal 5% deformation

²) When using these tables and equally when using the strength parameters based on NEN 6740, Table 1), it is wise to take into account the uncertainty in relation to the calculated difference between cell and triaxial tests used in this table.

Pressure dependent/independent approach

The pressure dependent approach provides a better schematization of the true material behaviour. As a result the material factor can be lower.

Cell or triaxial tests

With cell tests the shear strengths found are lower than those found with triaxial tests. This means that to obtain the same calculation values the material factor for cell tests must be lower. However there is uncertainty about what material factors should be used. From recent comparisons between collections of test results it appears that the differences between the results of cell and triaxial tests results can be significantly greater than in the current material factors (Tables 5.3.1 and 5.3.2). Because of this when the current factors used with the results of cell tests a lower calculation value will be found for the shear strength than is found when results of triaxial tests are used with current factors. This difference is at its maximum if for the triaxial tests the shear strength parameters are determined by failure. If the shear strength parameters are determined by a smaller deformation this difference will be smaller. For triaxial tests it is stipulated that the axial strain (in %) is reported.

Grey area

At present the magnitude of the above-mentioned differences is under investigation. Amongst other things the type of soil and the strain criterion used in the triaxial tests are being considered. This will lead to the replacement of the material factors in Table 5.3.1 and 5.3.2 by new material factors. It is anticipated that these will be presented concurrently with the introduction of the new *Guidelines for river dikes*.

The point of departure is that the improved material factors are set within a 'grey area'. On one hand these are limited by current methods using the results of cell tests (upper limits) and, on the other hand, by the current method of using the results of triaxial tests with failures (lower limit). This means that when the material factors given in Tables 5.3.1 and 5.3.2 are used – in anticipation of the new material factors – with the cell tests (CT) a dike can be approved with certainty with regard to macro-stability, and be rejected with certainty when the triaxial test (TP-CU) is used for failure.

Strength parameters

If the strength parameters are estimated on the basis of Table 1 (Ref. NEN 6740) those for clay should be considered as being determined on the basis of triaxial test with maximum deformation (TP-CU), assuming a pressure independent approach. It is advisable to take a minimum material factor of 1.5 for the cohesion and 1.2 for the angle of internal friction, or

one as much higher as follows from Table 5.3.1. These values should also be considered as the lower limit of the above-mentioned grey area.

Model factor

The model factor γ_d takes into account different uncertainties with regard to the calculation model. The model factor can be taken as the product of two parts. When using the method of Bishop a model factor of $\gamma_d = 1.0$ is calculated.

When using the pressure bar method the model factor is dependent on the uplift safety, which is:

EQUATION (5.3.7)

Note this equation number (5.3.6) appears twice in the Dutch text.

$$\gamma_d = 1,0 - \frac{1}{2}(n_{\text{opdruk}} - 1,0) = 1,5 - \frac{1}{2}n_{\text{opdruk}}$$

In which $n_{\text{upward pressure}}$ represents the resistance to upward pressure (uplift). In the pressure bar model the model factor varies between 0.9 and 1.0. For a calculation model according to the finite elements method it is recommended that a value of $\gamma_d = 1.0$ should be taken.

Damage factor

The damage factor γ_d takes into account the basic assumption for the safety analysis, namely that the probability of failure resulting from causes such as instability is negligible. The damage factor includes three partial aspects: the exceedence frequency of the design level, the consequences of soil-mechanical instability and the length of the water defences around a ring dike (the latter aspect derives from a ring dike approach). For the damage factors and model factors to be used reference should be made to (Ref.) *Guidelines for the design of river dikes: Part 2 Lower River area*.

Safety and material factor in implementation

Depending on the question of whether the pore pressure is monitored, a material factor of 1.5 or 1.25 is used for the consolidation coefficient in dry soil fill additions.

When calculating the expected consolidation of the substratum in wet soil fill additions a partial material factor of 1.5 should be used for consolidation characteristics, in combination with careful assumptions with respect to drainage of the fill material. As in the case of the safety assessment in the end situation, the analysis of the critical situations during construction also uses the semi-probabilistic method as set down in the (Ref.) *Guidelines for the design of river dikes: Part 2 Lower River areas*. This means that characteristic estimates of soil parameters, material factors and damage factors are used. When calculating water pressure or over pressure, the material factors mentioned in this section are used.

One stipulation is that the damage factor for slip planes that affect the water-retaining capacity of the dike, is at least 1.0. For slip planes that do not affect the water-retaining capacity, a damage factor of at least 0.9 is recommended. The water-retaining capacity is affected when the water defence is damaged or fails, or if failure occurs for water levels lower than those occurring before construction started. When there is danger of damage or failure as a result of non-stationary aspects, the water-retaining capacity is also affected if damage occurs in a shorter period of loading than the period of loading after which damage (or failure) would occur in the original situation.

Differentiation of the damage factor in shearing

When assessing the macro-stability of the inner slope, it is possible to apply a further nuance to the damage factor to be used. Not every instability necessarily affects the water-retaining capacity of a dike. This has led to a differentiation between damage factors depending on the location of the entry point of the slip plane. Depending on the degree of direct danger of changes in the water bearing capacity resulting from the initiation of a slip plane, a lower

damage factor can be recommended. Consideration of the location of the sheared soil and the occurrence of new slip planes in the remaining dike body has led to a division of the dike profile into three zones (zoning) for inward-oriented slip planes. For the outward-oriented slip planes, no zoning has been applied as a lower damage factor already applies for these slip planes. Depending on the zone in which the entry point for the slip circle is located, a lower damage factor may be applied, if designed under certain conditions as mentioned in the *Structural Design Manual*, Annex 2H). This Annex defines the zones and indicates how to determine the size of the damage factor. It shows, for example, that if a damage factor of 1.20 is required for a slip plane in zone 1, a factor of 1.12 is permissible in zone 2 and a factor 1.0 is allowed in zone 3.

5.3.6 Improvement measures

If the macro-stability of the dike is insufficient this can be improved by either making a gentler slope or adding an inner berm. Naturally it is possible to combine these measures. In practice, from the point of landscape and LNC (landscape, nature and cultural) considerations there may be a preference for a particular solution. Possibly soil improvement will be necessary (see § 6.5). If very weak layers are responsible for major settlement (and over height) and/or space is required for wide berms, the removal of the weak layers and their replacement by sand can be considered.

□

5.4 Micro-stability

□

5.4.1 Introduction

Micro-stability relates to the stability of soil layers of very limited thickness on the surface of a slope under the influence of groundwater flowing through the dike. In this case the threat to stability comes from within. A high phreatic line in the soil body may cause problems. The stability differs from that of the inner slope under the influence of wave overtopping (see § 5.5); here the infiltrating water causes a saturated top layer that may shear as a result of the extra weight of the water. Wave overtopping and micro-stability are therefore different failure mechanisms and although the modelling of them is to some extent comparable it is not the same (see Figure 5.4.1).

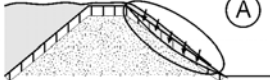
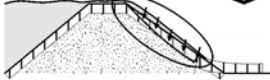
Type of dike	Micro-stability	Stability with wave overtopping
Clay dike	 No problem	 Infiltration and shearing
Sand dike	 Out-wash	 Erosion of inner slope
Sand dike with clay covering	 Push off /shearing	 (A) Infiltration and shearing
		 (B) Infiltration and micro-instability
		 Shearing of upper slope

Figure 5.4.1 Comparison between micro-stability and stability with wave overtopping

Micro-instability concerns very local instabilities that nevertheless can give rise to the collapse of the entire water defence. The term micro-stability more or less indicates that this relates to the degree of granular equilibrium. This contrasts with macro-stability, which concentrates on the dike body as a whole, or on large sections of the dike. Micro-stability is no less important a facet of the water defence; however since micro-instability it can also lead a dike to collapse. Micro-instabilities resulting from groundwater currents through the dike include:

- the pushing off of the revetments of the inner slope by water pressure in the core of the dike (see Figure 5.4.2): this can occur when a less permeable top layer lies over a permeable core, for example a top layer of clay on a core of sand (§ 5.4.2)
- Shearing of the inner slope revetment (or a combination of shearing and pushing off) by water pressure in the core of the dike: this can occur when a less permeable top layer lies over a permeable core, for example a top layer of clay on a core of sand (§ 5.4.2).
- Shearing of the inner slope as a result of an increase in height of the phreatic line in the dike (see figure 5.3.2). It is assumed that the structure of the dike is uniform (a sand core with a sandy top layer) and horizontally expelled groundwater from slopes above water level (§ 5.4.3) or ground water expelled at right angles for slopes under water (§ 5.4.3).
- Washing out of soil particles from the inner slope as a result of exiting groundwater (Figure 5.2.3). This may occur when the permeability of the top layer is approximately equal to that of the core of the dike. Assuming that the groundwater is expelled horizontally for slopes above water (§ 5.4.3) and at right angles for slopes that are under water (§ 5.4.4).

Design rules have been developed for the above-mentioned collapse mechanisms, as discussed in § 5.4.5. Improvement measures are considered in § 5.4.6.

5.4.2 Sand dike with a top layer of clay

When assessing this type of dike, two opposing parameters are considered. These are the pressure of the groundwater against the inner slope revetment and the opposing force of the weight of the inner slope revetment. When determining the water pressure under the revetment in the following consideration this amounts to the fact that the phreatic line in the dike body must be determined. Next it is assumed that a hydraulic pressure develops under the point where the phreatic line reaches the revetment. The latter assumption contains several simplifications such as the omission of the flow pressures. These simplifications are considered to be admissible.

If it is assumed that in the initial situation the inner and outer water levels are the same, the following model can be drawn up. At a certain time a sudden long-term increase in the outer water level. The position of the phreatic line in the dike body will now change because water penetrates into the dike through the revetment of the outer slope and from the substratum. As a result of velocity and geometry the phreatic line on the outer side rises the most quickly. This rise continues and extends to inner slope (see Figure 5.4.2).

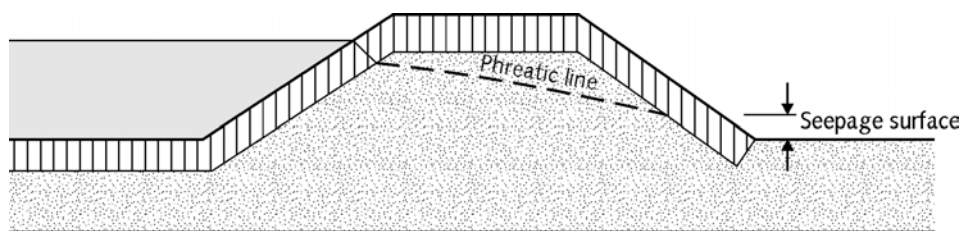


Figure 5.4.2 Push-off slope revetment in combination with internal erosion

If the phreatic line under the inner slope rises to above ground level, the inner slope revetment may be pushed off. In this case it is assumed that there is no significant groundwater current through the inner slope revetment. On the basis of this assumption, the inner slope revetment is lifted up when the hydrostatic pressure in the dike is greater than the weight of the revetment. Once the revetment has been lifted up off, the soil particles located under the revetment may be washed out.

A second failure mechanism that may occur with this type of dike is the shearing of the inner revetment. When there is a high water level water pressure develops under the less permeable inner revetment. As a result of this, the effective stress (soil pressure minus pore pressure) on the boundary between the core and the inner revetment is reduced, even if the water pressure is not yet high enough to force off the inner revetment. In consequence of the reduced effective stress the shear resistance is also lower. The inner revetment has a weight component parallel to the inner revetment. If the water pressure under the revetment becomes too high the shearing of the inner revetment may follow.

5.4.3 Sand dike with a sandy top layer

With this type of dike it is necessary to consider the equilibrium at right angles to equilibrium parallel to the slope. Equilibrium at right angles to a slope is also termed the washout of soil particles. Equilibrium parallel to the slope concerns shallow slip planes in the sandy upper layer. The washing out of soil particles is particularly prevalent on the underside of the inner slope during and shortly after high water levels. In practice, this phenomenon only occurs with non-cohesive granular materials. With cohesive materials, deeper shearing is more likely to occur (see § 5.3).

The occurrence of micro-instability does not necessarily have to lead to loss in water-retaining function of the dike. The washed out material will be deposited a short distance away. Moreover the washed out material will also (certainly in theory) be limited to the zone in which the stability criteria are not met.

For slopes above the water line, a global estimate of the damage profile can be made on the basis of the following assumptions (see Figure 5.4.3):

- The material cannot wash out at a point higher than the original seepage surface;
- The material washed out is deposited with an equilibrium slope of approximately 1 : 5;
- The material located at a higher point may slip off as a result of the natural angle of slope;

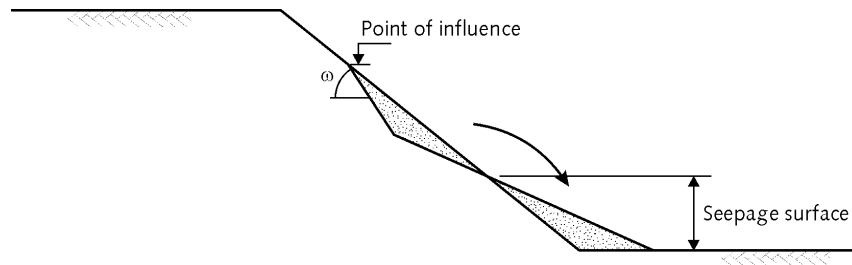


Figure 5.4.3 Damage profile

In defining failure of a water defence as a result of material being washed out it can be assumed that this occurs when the point of encroachment reaches a minimum crown height. The dike manager should determine the height of this minimum, or the degree to which erosion may be permitted to increase.

A second failure mechanism in this type of dike is the occurrence of shallow sliding planes in the sandy upper layer (equilibrium parallel to the slope). Under the influence of the slow pressure of the water that is expelled, the effective stress of the soil particles is reduced. This may lead to the occurrence of small shallow sliding planes. In this type of dike the direction of the groundwater flow is important. In slopes above water the groundwater is expelled almost horizontally from the inner slope. In slopes under water (ditches and waterways behind the dike) the equilibrium potential lines are almost parallel to the slope and the water is expelled at right angles to the slope. This is expressed in a difference between the calculation values for slopes above or below water level.

5.4.4 Calculation rules

The calculation rules are based on the stability of the soil layers of limited thickness on the surface of a long slope under the influence of flowing water. This concerns both the equilibrium along straight slip planes parallel to the slope and the equilibrium in a direction at right angles to the slope.

Sand core with clay revetment: testing for push up

The core of the dike body is saturated to a height h and covered by a layer of clay. On the side of the inner slope the phreatic line is high, as a result of which a pressure difference occurs on the underside of the slope close to the toe. This difference in pressure will push up the top layer of the core, while the weight of the top layer will resist this action. If the top layer is heavy enough it will remain in place, The requirement for pushing up is satisfied if:

$$\text{EQUATION} \quad (5.4.1) \\ \cos \alpha \geq \frac{\gamma_d \gamma_{m,p} \gamma_n \rho_w (h - z)}{\rho_g d}$$

In which

d assumed layer thickness (perpendicular to the slope) [m]

h groundwater level in relation to a reference level [m] ^{2*)}

*) The hydraulic head can be determined for the calculation models given in Table 4.661

z	height of the toe in relation to a reference level [m]
α	angle of the slope [°]
$\gamma_{m,p}$	(=1.0) material factor for the volumetric mass [-]
ρ_w	volumetric mass of water [kg/m ³]
ρ_g	volumetric mass of soil [kg/m ³]

Equation 5.4.1 is the result of a simple assumption in which the friction resistance that will push up a slice of soil on both sides is neglected. If this effect is included in the calculation the following equation is obtained (Ref.) *Micro-stability: overview of modelling*:

EQUATION (5.4.2)

$$\frac{2cd}{\gamma_{m,c}} + \frac{\rho_g g}{\gamma_{m,p}} \Delta x d \cos \alpha + \frac{\rho_g g}{\gamma_{m,p}} \Delta x d \sin \alpha \frac{\tan \phi}{\gamma_{m,\phi}} \geq \gamma_n \gamma_d \left(\Delta h - \frac{1}{2} \Delta x \sin \alpha \right) \frac{\rho_w g}{\gamma_{m,p}} \Delta x$$

This equation can be solved if $\Delta h = (h-z)$ is known. It is necessary to determine whether there is a value for Δx that does not meet the above requirement. The term Δx stands for the width of a slice of ground, measured parallel to the slope and beginning at the toe of the slope. For the total safety ($\gamma_d \gamma_n$) with perpendicular stability 2.0 is assumed (Ref. *Safety Monitoring Guidelines*).

c	cohesion for the clay [kN/m ² per m ¹]
g	force of gravity [m/s ²]
$\gamma_{m,c}$	(= 1.250) material factor for the cohesion [-]
$\gamma_{m,\phi}$	(= 1.1) material factor for the angle of internal friction [-]
ϕ	angle of internal friction of the clay [°]

The equation is considerably extended and therefore it is better to program it into a spreadsheet. To check this for errors the following numbers are used

c	=	2 kN/m ² per m ¹
d	=	0.8 m
g	=	9.81 m/s ²
tan ϕ = 0.4		
ρ_g	=	1630 kg/m ³
ρ_w	=	1000 kg/m ³
slope	=	1:3

The method of procedure is as follows: choose a hydraulic head (h-z) for example 1 m. Vary Δx between 0 and the maximum value $\Delta x_{\max} = \Delta h \sin \alpha$ (in this case 3.16 m) and see if a value can be found that does not meet the requirement of the equation. This is the case for $\Delta x = 1$ for example. This means that the chosen (h-z) must be less, for example 0.95 m. With this there is no value for Δx that does not meet the requirement. If this must be examined even more precisely it can be seen that with $\Delta x = 0.98$ m and $\Delta x = 0.9$ m it only just meet the requirement. With $\Delta h = 0.99$ m and $\Delta x = 1$ the requirement is no longer met. The maximum permitted difference in hydraulic head over the clay layer is thus 0.98 m.

If the one-dimensional approach (equation 5.4.1) in this example only a difference of 0.23 m in the hydraulic head is permissible.

In Figure 5.4.4, for covering top layers of varying thickness, an angle of slope is given that, with the constants given in the numbered example and the associated safety factors, gives the permissible hydraulic head.

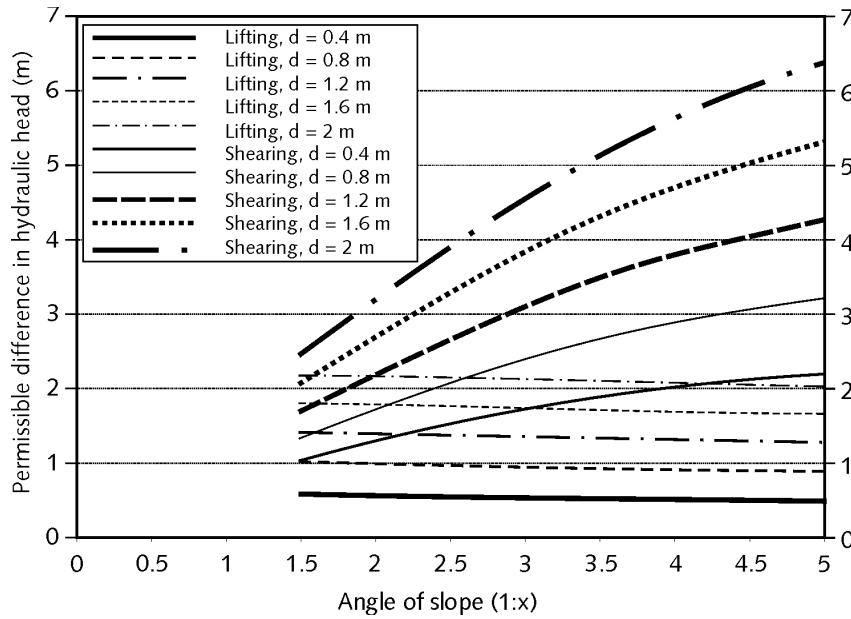


Figure 5.4.4 Admissible head for sand cores with clay covering layers

Sand core with clay covering: testing for shearing

On the basis of an equilibrium of forces the following requirement can be derived:

EQUATION (5.4.3)

$$\cos \alpha \geq \frac{\gamma_n \gamma_d (h - z) \rho_w}{\gamma_{m,\rho} d \rho_g}$$

This equation is the result of a one-dimensional approach (infinitely long slope). If the friction resistance that must be overcome at the toe of a dike is also taken into consideration the following equation is derived (Ref. *Micro-stability: overview of modelling*):

EQUATION (5.4.4)

$$\gamma_n \gamma_d \left(\frac{\Delta h d}{\gamma_{m,\rho}} \frac{\rho_g g}{\gamma_{m,\rho}} - \left(\frac{cd}{\gamma_{m,c}} + \frac{c \Delta h}{\gamma_{m,c} \sin \alpha} + \left(\frac{\Delta h}{\tan \alpha} d \frac{\rho_g g}{\gamma_{m,\rho}} - \frac{1}{2} \frac{\rho_w g \Delta h^2}{\gamma_{m,\rho} \sin \alpha} \right) \frac{\tan \phi}{\gamma_{m,\phi}} \right) \right) \leq \frac{cd}{\gamma_{m,c} \sin \alpha} + \frac{1}{2} \frac{d^2 \rho_g g}{\sin \alpha \gamma_{m,\rho}}$$

In which:

- c cohesion of the clay {kN/m² per m¹}
- g force of gravity
- $\gamma_d = 1.1$
- $\gamma_{m,c}$ (= 1.25), material factor of the cohesion [-]
- $\gamma_{m,\phi}$ (= 1.1) material factor for the angle of internal friction [-]
- $\gamma_n = 1.1$
- ϕ angle of internal friction of the clay

(Ref. *Guidelines for the design of river dikes; Part 1 Upper River area*)

This is a long equation that is best used when programmed into a spreadsheet. To determine whether any errors have been made a numeric example is given:

c	=	2 kN/m ² per m ¹
d	=	0.8 m
g	=	9.81 m/s ²
tan ϕ	=	0.4
ρ_g	=	1630 kg/m ³
ρ_w	=	1000 kg/m ³
slope	=	1:3

If these values are filled in it appear that a difference in hydraulic head of $\Delta h = 2.42$ m just meets the requirement. When using the one-dimensional equation in this numeric equation it is 1.0 m. In Figure 5.4.4, for covering top layers of varying thickness, an angle of slope is given that, with the constants given in the numbered example and the associated safety factors, gives the permissible hydraulic head.

Sand dike with sandy inner slope above water: testing for washout

This is the most dangerous situation for the loss of micro-stability in slopes above water if water is expelled horizontally. On the basis of an equilibrium of forces, in which the perpendicular stability is considered, the following equation can be derived for the angle of the slope:

EQUATION (5.4.5)

$$\tan \alpha \leq \sqrt{\frac{\rho_g - \rho_w}{\rho_w \gamma_d \gamma_{m,\phi} \gamma_n}}$$

With perpendicular equilibrium 2.0 is used for the total safety ($\gamma_d \gamma_n$). (Ref. *Guidelines on safety Monitoring*. If this requirement is not met it is possible investigate whether, with the aid of the damage profile (see Figure 5.4.3) it can be demonstrated that this leads to only limited damage (see § 5.4.3).

Sand dike with sandy inner slope above water: testing for shearing

In a sand dike with a sandy inner slope, if the phreatic line in the core is high, water will be expelled almost horizontally from the inner side. On this assumption and assuming $c = 0$ the for the equilibrium parallel to the slope can be derived from:

EQUATION (5.4.6)

$$\tan \phi \geq \frac{\gamma_n \gamma_d \gamma_{m,\phi} \rho_g g \sin \alpha}{\rho_g g \cos \alpha - \frac{\rho_w g}{\cos \alpha}}$$

With $\gamma_d = 1.1$ and $\gamma_n = 1.1$

(Ref. *Guidelines for the design of river dikes: Part1 – Upper River area*)□

Sand dike with sandy inner slope under water: testing for wash out

With an underwater slope the groundwater will be expelled more or less at right angles. This leads to the following equation.

EQUATION (5.4.7)

$$\cos \alpha \geq \frac{\gamma_n \gamma_d \gamma_{m,\phi} i \rho_w}{\rho_g - \rho_w}$$

The hydraulic head i must be determined by using a groundwater flow calculation. This test is only necessary if the inner slope is partly under water, for example because there is a ditch or watercourse behind the dike. It is also possible to investigate the stability of the slopes of ditches near an inner berm, but then it is necessary to consider whether this forms a threat to the stability of the dike (see § 5.3.5 Assessment of the damage factor in shearing). With perpendicular equilibrium 2.0 is used for the total safety ($\gamma_d \gamma_n$). (Ref. *Safety Monitoring Guidelines*).

Sand dike with sandy inner slope under water: testing for shearing

For ground water expelled at right angles to the slope, from slopes under water assuming that $c = 0$, the following equation for equilibrium parallel to the slope is derived:

EQUATION (5.4.8)

$$\tan \phi \geq \frac{\gamma_d \gamma_{m,\phi} \gamma_n (\rho_g - \rho_w) g \sin \alpha}{\rho_g g \cos \alpha - \rho_w g (\cos \alpha + i)}$$

With $\gamma_d = 1.1$ and $\gamma_n = 1.1$

(Ref. *For the design of river dikes: Part 1 Upper River area*)

The hydraulic gradient i must be determined by using a groundwater flow calculation. This test is only necessary if the inner slope is partly under water, for example because there is a ditch or watercourse behind the dike. It is also possible to investigate the stability of the slopes of ditches near an inner berm, but then it is necessary to consider whether this forms a threat to the stability of the dike (see § 5.3.5 Assessment of the damage factor in shearing).

5.4.5 Improvement measures

There are various ways in which micro-stability can be improved. Sometimes a phreatic line can be taken up in a berm that in any case is required for macro-stability. At the level of the intersection of the phreatic line a slope reduction to 1:5 can be introduced to prevent the washing out of material. Drainage can also be used to prevent the phreatic line from reaching the inner slope. Finally, it is also possible to apply a thicker layer of clay on the inner slope so that no push-up or shearing can occur. On the outer side of the dike it may also be possible to take action on the water management, for example by applying a less permanent revetment.

5.5 Stability and overtopping

5.5.1 Introduction

Overtopping is merely water passing over the crown of a dike in the event of high water levels. The water forces its way into the structure by the process of infiltration (See § 5.3.1) and this may lead to shearing of inner slope.

In the 1953 Flooding catastrophe, many Zeeuwse dikes collapsed as a result of inner slope shearing following overtopping. For this reason, in current practice it is common to choose a crown height that is sufficiently high to ensure that very little overtopping occurs, even under normative conditions. Therefore it is not necessary to take wave overtopping into account when calculating the dimensions of the dike.

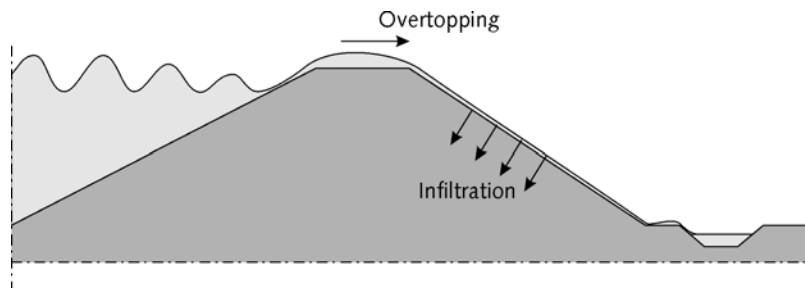


Figure 5.5.1 Infiltration caused by wave overtopping

In complex situations, however a decision may still be made to allow a larger overtopping capacity. This must be a conscious choice, the consequence of which is that the dimensions of the crown and the inner slope are such that they can withstand the effects of overtopping. The choice must not lead to difficulties with the water management in the polder.

The mechanisms that occur during infiltration due to wave overtopping are described in § 5.5.2. A distinction is made between two characteristic situations for dike structure. The flowing water may also cause erosion and this process is described in § 5.5.3 (see also Chapter 7). In § 5.5.3 an indication is given of how earth structures can be dimensioned in such a way that they are secure against loss of stability resulting from infiltration. Subsection 5.5.5 suggests some measures for improvement.

5.5.2 Infiltration and shearing

In infiltration due to overtopping or precipitation, water enters the dike via the crown and inner slope. During this infiltration process, the mass by unit volume increases from γ_{dry} to γ_{wet} . The pore pressures in the infiltration zone are of almost no significance as long as the infiltrating water flows further into the dike. The degree of infiltration depends on the amount of overflowing water (overtopping capacity), the total duration of overtopping and the permeability of the covering clay layer. The time factor is therefore very important.

In a saturated infiltration zone, pore pressure increases and the effective stress decreases, thereby reducing the possible shear stress. The driving force increases because the soil is saturated with water. This has a negative impact on the stability of the inner slope. Instability of the inner slope is shown by the deformations that occur there. Initially the slope will move in the direction of the dike toe and take on a rounded shape. A strain zone is created near the crown line in which a split may develop parallel to the crown. When this occurs, the dike is considered to have collapsed. Prototype tests have indicated that the formation of a split and the rounding and extrusion of the underside of the inner slope occur at the same time. At the moment that a split forms, a large part of the water that has overtopped the dike enters the split and ensures that the dike is quickly saturated, starting the shearing process of the inner slope. This is usually a surface shear in which the entire grass cover shears off the dike, exposing the dike core to erosion by the overtopping water. In almost no time a breach is formed in the dike.

These parallel splits were seen in the dikes in the southwest of the Netherlands in 1953. In some cases, the managers spread tarpaulins over the splits. This often slowed down the process of collapse. The collapse mechanism initiated by infiltration is strongly dependent on the configuration (angle of slope) and the structure of the dike. If slopes of less than 1 in 3 are used, experience shows that there is little danger of instability as a result of infiltration by wave overtopping. In principle, there are two characteristic situations to describe the infiltration process:

- The dike consists of a clay core, covered by a clay layer permeated by the roots of the vegetation growing on it.
- The dike has a sandy core, covered by a clay layer.

In practice, situations may occur in which the heterogeneous nature of the structure plays an important role. Other types of mechanism may occur locally. In this context, it can be established that the holes created by the digging activities of rabbits and moles represent a risk and further reinforce the process of other mechanisms.

Clay core

Clay on the surface is usually far more permeable than the clay in the core. Splits and root canals create a relatively high permeability for clay in the order of 10^{-5} to 10^{-4} m/s. During overtopping, the water infiltrates mainly into the top layer causing an increase in the pore pressure. In contrast, effective stress gradually decreases to such an extent that a slip plane may be created on the border between top layer and core at a depth of around 1 metre below ground level. On a steep slope, the top layer shears off.

Sandy core

Infiltration can also cause collapse as a result of overtopping in dikes with a sandy core. The permeability of the top layer is highly anisotropic. Splits and vegetation often make the permeability only slightly less than that of the core. Directly at right angles to the slope, the permeability value is far greater than it is parallel to the slope. As a result of this anisotropic property, relatively little water flows through the top layer along the slope; most of the water seeps into the sandy core. If the core is not drained or if the top layer is less permeable than the core, for example if there is a top layer of clay over a core of sand, water infiltrate near the toe causing higher phreatic line under the revetment. This will cause upward pressure on the top layer. In this case when assessing the shearing of the inner slope as a result of overtopping the extra force on the inner slope must be taken into account. It is recommended that in these cases a groundwater flow calculation should be made to determine the phreatic line in the dike. The duration of the infiltration is important in determining the total amount of water that can infiltrate via the inner slope.

Once the height of the phreatic line has been determined is also necessary to investigate whether it is possible for a combination of two failure mechanisms to occur. First micro-instability is created in the lowest part of the inner slope after which shearing of the upper part of the revetment of the inner slope occurs. If this phenomenon of a combined failure mechanism forms a threat the same measures can be taken as have been mentioned in relation to macro-stability.

5.5.3 Erosion of the inner slope

Erosion on the inner slope of soil structures is primarily caused by water that flows over the crown. This water derives from wave overtopping or over flow. The combination of a high water level and wave run-up on the outer slope may result in overtopping. Because of the irregular nature of waves in overtopping a phenomenon that only occurs in extreme conditions must be taken into consideration. With a high overtopping flow erosion of the inner slope can be caused by water flowing along or off the slope. This erosion appears to be a function of the overtopping flow, the angle of slope, the types of soil covering the slope and the quality of the vegetation.

5.5.4 Methods of calculation

Joustra and Edelman

For ground water flowing parallel to the slope above water Joustra and Edelman devised the following analytical equation for a homogeneous slope of infinite length:

[EQUATION] (5.5.1)

$$\tan \phi \geq \frac{\gamma_d \gamma_{m,\phi} \gamma_n \rho_g g \sin \alpha - \gamma_{m,\phi} \gamma_{m,p} \frac{c}{\gamma_{m,c} d}}{\rho_g g \cos \alpha - \rho_w g \cos \alpha}$$

In which:

c	Cohesion [kN/m^2 per m^1]
d	Layer thickness measured perpendicular to the slope [m]
g	force of gravity m/s^2
α	Slope angle [$^\circ$]
γ_d	(=1.1) Model factor [-]
$\gamma_{m,c}$	(= 1.25) Material factor for cohesion [-]
$\gamma_{m,p}$	(= 1.1) Material factor for the angle of internal friction [-]
$\gamma_{m,\rho}$	(= 1.0) Material factor for weight by unit volume [-]
γ_η	(= 1.1) Damage factor [-]
ϕ	Angle of internal friction [$^\circ$]
ρ_w	Mass per unit volume for water [kg/m^3]
ρ_g	Mass per unit volume for soil [kg/m^3]

Stability with infiltration

The simplest numerical method is a slip plane calculation according to Bishop (§ 5.4) that is preceded by a groundwater current calculation. Using the finite elements method allows improved modelling of the behaviour of the soil since the shape of the slip circle is not necessarily circular as in the Bishop method. It is often assumed that the phreatic plane coincides with the slope and that the hydraulic head is constant at right angles to the slope. For a sandy core, the hydraulic head distribution is generally more generous as the infiltration front behaves as a water bubble and does not immediately penetrate into the substratum. In the actual design, the process that is described in Figure 5.5.2 can be used. This approach assumes a relatively homogeneous layer structure with a clay revetment on a sandy core or clay revetment on a clay core. In the event of a relatively impermeable revetment (clay revetment on a sandy core), it is important to know the extent to which water overpressure can develop in the core. In this case, the capacity and duration of the overtopping period are important boundary conditions. The result is that the design calculations and the resulting solutions may differ somewhat from the 'clay on clay' scenario.

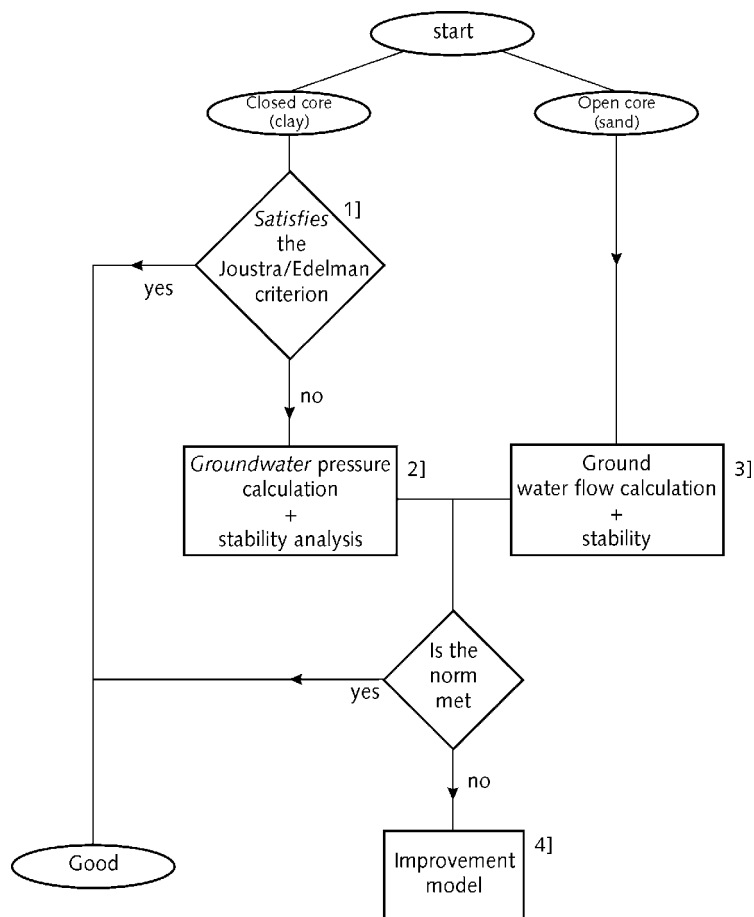


Figure 5.5.2 Flow chart showing processes for analyzing stability in infiltration

- (1) Test the angle of the inner slope with the analytical solution from equation (5.5.1). For non-homogeneous (layered) structures of the slope, the characteristics of the layer which give the lowest angle α in the design are chosen. If cohesion is apparent this must be taken into consideration. For the thickness of the shearing layer in a clay dike, the thickness of the structured top layer (often about 1 metre) is used. Further information on friction characteristics of this structured clay More information about the friction coefficient can be found in (ref. *Technical Report on Demands of Clay in Dikes*).
- (2) Perform a two-dimensional stability analysis, preceded by a groundwater flow calculation (see Table 4.6.1). The stability analysis can be performed by using the Bishop method or with a finite elements program. In the latter case, no restrictions are imposed on the shape and location of the slip plane. This shape and the degree of deformation that is likely to occur are used to determine whether the slope angle is permissible. In all cases, it must be assumed that the slope is entirely saturated and that the phreatic surface coincides with the slope. No calculations that assume an unsaturated condition of the clay and water streaming cover the slope surface can provide a better picture of events are available yet.
- (3) Although testing according to equation 5.5.1 is possible for dikes with a permeable core, it is not recommended. Infiltration via the crown and the inner slope and groundwater flows through the outer slope would first have to fill the sandy core before saturation of the revetment of the inner slope could occur. It can be assumed that before the core is completely saturated a reduction in effective stress between the revetment layer and the core close to the inner toe or failure will occur.

From the above it appears that the degree of saturation of the top layer is very important. If the overtopping capacity is less than 0.1 l/m/s it is unnecessary to take overtopping into account, since the top layer is not considered saturated. With higher rates it is assumed that saturation occurs. In the *Structural design manual* a global indication resistance to shearing is given. On the basis of estimated calculations the shaded area in Figure 5.5.3 shows where, given complete saturation, shearing will occur.

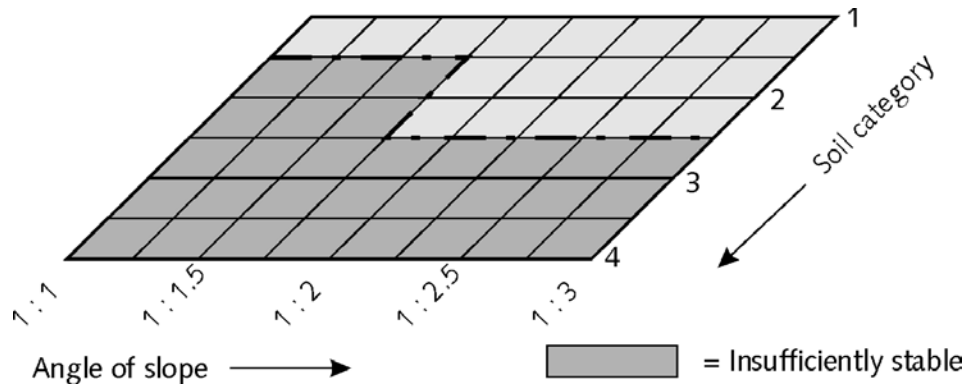


Figure 5.5.3 Indication of the area of shearing

The 'category of soil' used in the figure relates to erosion resistance of clay as stated in *Standard RAW Regulations, CROW 2000*.□

5.5.5 Measures for improvement

There are various alternatives for adapting the design. The first is to reduce the angle of the lower part of the slope. The second is to increase the height of the dike crown to reduce the overtopping capacity. A third possibility arises in the event of a dike with sandy core. In this case, the phreatic line can sometimes be kept low by means of a drainage structure. This drainage structure becomes an essential part of the water defence and a point of attention in management and maintenance. There must be security with respect to functionality of the structure. It is also possible that LNC-conditions necessitate other solutions (special structures, improving the characteristics of the covering layer).

Anisotropy of soil structures makes it necessary to have some reserves with regard to the acceptance of overtopping. The acceptance of a higher rate means that the design can have a lower crest height. A lower crown height means that the longer period during which the overtopping occurs will be longer and that the chance of complete saturation becomes greater. Studies on the mechanism of stability with overtopping are still in progress.

5.6 Sand-bearing springs

□

5.6.1 Introduction

The development of quicksand when water that exits vertically (heave) and the formation of sub-surface channels (piping) may threaten the stability of soil structures. The lifting up of soil particles and retreating erosion may occur as a result of seepage water carrying away soil particles in water-bearing sand layers in the event of major hydraulic gradients over soil structures. This may result in either the appearance of continuous channels under the soil structure, or a loss of effective stress as a result of too large a vertical hydraulic gradient. The report on *Sand-bearing Springs*, pays detailed attention to possible safeguarding measures in threatening situations and to management aspects relating safeguards against possible emergency situations. Finally, a number of case studies in which classical calculation rules

are applied on the basis of stationary groundwater flows and non-stationary calculation models are presented.

This section briefly covers the above-mentioned aspects. Here, the emphasis lies on the description of the various phases that may arise in the event of springs under the soil structure (§ 5.6.2). Attention is also devoted to calculation rules for checking for the occurrence of heave and piping (§ 5.6.3 and § 5.6.4). In Section 5.6.5 possible structural solutions and measures involving management techniques to ensure safety in relation to sand carrying springs is adequate are considered.

5.6.2 Process description

In principle, it is possible to distinguish two characteristic configurations in which seepage water penetrates under and into the base of the soil structures:

1. There is a covering layer consisting of clay and/or peat on the water-bearing sand layer inward of the soil structure
2. Inward of the soil structure there is little or no covering soil layer, on the water-bearing sand layer that, for example is dissected by a wide ditch.

□

Inward covering layer

The following seepage phases are defined for impermeable soil structures, the inward top layer of which is watertight:

- Uplift of the covering layer. As a result of high outer water levels, the pore pressure in the water-bearing sand layer will increase. If the water pressure at the point of the covering layer on the inside is larger than the weight of that layer, the covering layer will be subject to uplift. This phenomenon is a form of hydraulic soil collapse. In practice, uplift can sometimes, though not always, be detected from small wave motions of the ground when trodden on.
- Heave of the covering layer. Uplift will cause the covering layer to split, as a result of which the seepage water seeks a way to ground level via vertical channels.
- Erosion in the water-bearing sand layer. The occurrence of horizontal channels in water-bearing sand layers is highly dependent on the velocity of the flow in these channels and the critical flow speed (= the average flow velocity of the water at which the sand grains remain more or less in place). The hydraulic gradient over the soil structure determines the acting flow velocity, while the critical flow velocity is highly dependent on material properties, such as grain size. In principle, there are two possibilities, these being: (1) the flow velocity is lower than the critical flow velocity; the erosion process stops, or (2) the flow velocity is greater than the critical flow velocity; the erosion of sand from the channel continues. The eroded material is carried away by the seepage water and deposited around the hole near ground level. If the hydraulic gradient is steep enough, the channels will continue to grow (retreating erosion). If these channels reach the outer water, this is referred to as piping. This will eventually lead to the formation of hollow spaces under the soil structure that develop to such dimensions that weakening and breach, and therefore collapse of the soil structure will occur.

No inward covering layer

If the composition of the soil on the inner side of soil structures is sandy, the vertical grain stress may fall away under the influence of seepage. This is defined as heave, or the formation of quicksand. This occurs primarily in groundwater that exits vertically. Consequently, sub-surface channels develop that increase in size as a result of the eroding action of seepage water, with the result that the structure collapses. Since both heave and piping can occur on the surface in water-bearing layers, these two aspects should be assessed separately. To prevent these phenomena, seepage berms or seepage screens (dam walls, clay coffer and cofferdams) are constructed. These increase the distance that the water must cover, thereby decreasing the velocity of the seepage water and the hydraulic gradient of the expelled water (Figure 5.6.1).

5.6.3 Calculation rules for uplift and heave

Research indicates that for uplift and heave of a covering layer, it is essential to know the thickness of this layer. In order to estimate the occurrence of quicksand/fluidized sand near the exit point, information must be available that relates to seepage length, permeability of the sand and the thickness of the water-bearing sand package: the thinner this layer, the lower the probability that the soil will become fluid.

A classic deterministic approach (*ref. Fundamentals of water retaining structures*) is used as a guideline to predict hydraulic soil breach for two characteristic configurations (see § 5.6.2). Advanced calculation models are available for the safety test with respect to heave. The WATEX program calculates the hydraulic head in the sand layer under a relatively impermeable layer from the theoretical entry point to the critical uplift zone for both stationary and non-stationary ground water flows. If there is no covering layer inward of the soil structure, the hydraulic heads can be determined by using a semi-analytical method based on the 'fragment method'. For more information on these models, reference can be made to (*Ref. Technical Report and bearing springs*). In order to ensure that there will be no danger of uplift and heave, the maximum acting loading should be smaller than the strength resulting from the material properties. When considering the mechanisms that cause uplift and heave, it is usual practice to express loading and strength as a hydraulic head, while for the investigation of the possible occurrence of quicksand downstream from earth structures, the hydraulic gradient is generally used as characteristic parameter. To summarize:

[EQUATION] 5.6.1

$$\phi_{strength} \geq \gamma \phi_{loading} \text{ or } i_c \geq \gamma i$$

In which, γ (>1) is a safety factor, i_c is the critical hydraulic gradient ($=0.5$) and i is the hydraulic gradient.

Inward covering layer

On the basis of comparison (§ 5.6.1), for a soil structure located on an impermeable layer, the following relationship can be deduced in the uplift zone (see Figure 5.6.2):

[EQUATION] 5.6.2

$$\frac{(\gamma_{nat} - \gamma_w)}{\gamma_w} d \geq \gamma (\phi_z - h_p) \text{ or } \phi_{z,g} \geq \gamma \phi_z$$

In which:

h	groundwater level or surface water level
d	thickness of the impermeable layer (see Table 5.6.2)
γ_{wet}	wet volumetric weight of the covering layer [kN/m ³] (see § 4.4.1)
γ_w	volumetric weight of the groundwater [kN/m ³]
γ_z	hydraulic head directly under the covering layer [m] (WATEX)
$\phi_{z,g}$	limit potential [m] (hydraulic head that gives rise to uplift)

If the hydraulic head in the sand layer is smaller everywhere than the calculated limit potential, heave will not occur. The local soil structure and soil properties are very important, although in practice it is almost impossible, or entirely impossible, to estimate these in detail. For the calculation to ensure that there will be no danger of heave, the covering layer should have a minimal thickness (low representative value); see Table 5.6.2 (or Annex II, Sand

bearing springs). In determining the safety coefficient to be used, a distinction must be made between the use of a safety test and the designing of a berm to prevent heave.

- If the acting hydraulic head under normative outer water conditions is measured by using only a geo-hydrological model, uncertainties with respect to the parameters can play an important role. In that case, a safety of $\gamma = 1.2$ is used. In situations in which uncertainty with respect to the acting hydraulic head is relatively small, $\gamma = 1.1$, (in accordance with geo-technical standard NEN 6740) will suffice.
- With support berms, the acting hydraulic head is partly determined on the basis of reliable water pressure response measurements. Should the water-bearing layer under the soil structure be sensitive to piping (see § 5.6.4), the introduction of a berm may provide a solution. This increases the seepage length, moving the heave location to the toe of the new berm. Under failure conditions, the limit potential is then located in the sandy layer. This is the boundary condition for accurately determining the course of the hydraulic head in the berm. For this reason, a safety coefficient/factor of $\gamma = 1.1$ according to NEN 6740 will suffice when the design thickness of the berm is being determined. In the (Ref. *Structural design manual*), a safety coefficient of $\gamma = 1.05$ is mentioned; although it is recommended that it is necessary to ensure that the uncertainty of the estimated acting potential is small, or that conservative estimates are used. If the criterion for heave is met in a safety test (see § 5.6.2), it is still necessary to ascertain whether springs are formed during high water (see § 5.2.8, Ref. *Technical report sand-bearing springs*). If the uplift criterion is not met, further tests for piping will have to be performed. For this, the heave location must be determined in order to calculate the acting seepage length for piping. This normative location is the point located closest to the dike, at which the heave criterion is not met (see Figure 5.6.1).

Influence of ditches and waterways

If there is a clearly less thick covering package locally in the stability zone, for example where there is a waterway (dike and berm ditches), the risk of uplift may be lower (increased safety). Figure 5.6.2 shows how to calculate the uplift safety using the loss in weight of the underlying layer under the ditch assuming a distribution of 2:1. With a layer thickness greater than ditch width B , no reduction in soil weight is applied. The maximum pore pressure remains the same as the weight of layer H_1 . If the layer thickness under the ditch is less than the width of ditch base b , in the calculation the maximum weight of the of the layer underlying ditch base H_2 is used, including the weight of the layer of water in the ditch. For layer thickness under the ditch base that is less than B and thicker than b , the maximum pore pressure against the base of the impermeable layer is equal to the weight of layer H_3 including the weight of the layer of water in the ditch. If at the location of the ditch, the limit potential head is reached, it can be assumed that the potential is reduced so much at that point that uplift of the hinterland areas can no longer occur. The thickness of the weak layer package in the hinterland areas is thicker than that on the ditch bottom. For this reason, the zone that has to be taken into account for uplift is limited to ditch width B .

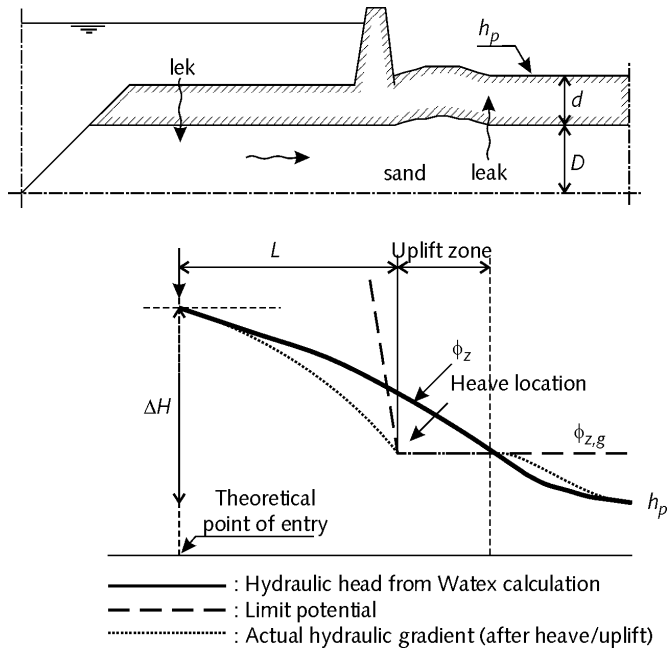


Figure 5.6.1 Dike with foreland and substratum; hydraulic head in sand layer, limit potential, heave location and uplift zone

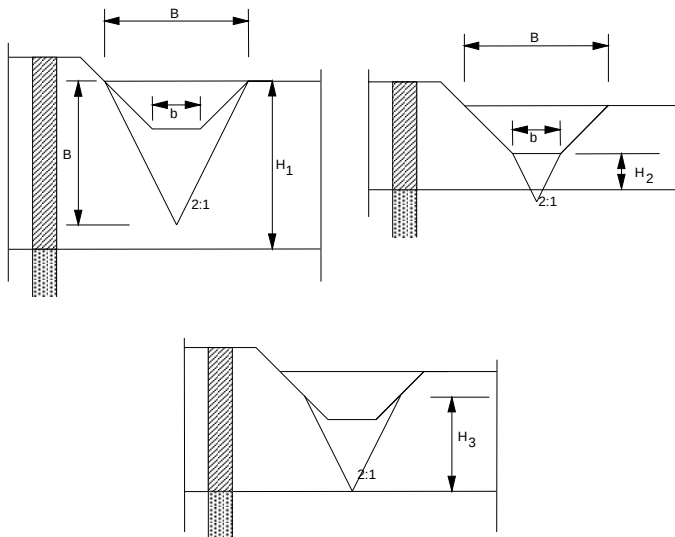


Figure 5.6.2 Determination of the effective layer thickness for uplift near ditch

No covering layer under the surface

A similar relationship as in (§ 5.6.2) can be deduced for vertical seepage currents in sandy ground behind a seepage screen on the inner side of a soil structure (see Figure 5.6.3)³:

[EQUATION] (5.6.3)

$$\frac{(1-n)(\gamma_k - \gamma_w)}{\gamma_w} \geq \gamma \frac{(\phi_0 - h_p)}{x} \quad \text{or} \quad i_c \geq \gamma i$$

³ Note that $\gamma_{wet} = (1 - \eta) \gamma_k + \eta \gamma_w$

In which:

- H_p polder level or the surface water level [m]
- η porosity [-] sand layer
- x distance over which the exit hydraulic head is maximal [m]
- γ_k volumetric weight of the grain material [kN/m³]
- γ_w Volumetric weight of the groundwater [kN/m³]
- ϕ_o Hydraulic head at the point of the base of the seepage screen where the exit hydraulic head is maximal [m]

If the soil has a layered structure, the volumetric weight underwater can be calculated by adding layer by layer. The exit hydraulic head (i) is highly dependent on the total hydraulic head of the structure and the geo-hydrological configuration and location and depth of any seepage screens. For soil structures on a sandy substratum with seepage screens, it is assumed that when the vertical exit hydraulic head (i) is greater than 0.5, sand-bearing springs are formed and therefore additional testing for piping is required (see § 5.6.465\4 Depending on the grading and packing of the sandy layer in the vicinity of the exit point, hydraulic heads greater than 0.5 are allowed. For this reason density measurements of the sandy layer are required (see Appendix I of *Technical Report on sand-bearing springs*, 1999).

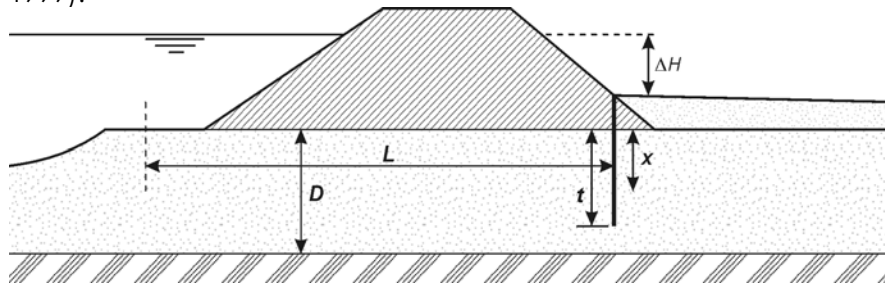


Figure 5.6.3 Seepage screen in a dike

The critical hydraulic head (i_c) depends on the porosity and has a theoretical value between 0.9 and 1.1. If the above-mentioned is taken into account, the safety coefficient γ has a value between 1.8 and 2.0; however, when the soil is homogenous in nature this safety factor is often not reached. Figure 5.6.4 shows a number of results of calculations of the 'Fragment Method' for a simple test for heave. For more information on this method, reference can be made to (Ref. *Technical Report on sand-bearing springs*).

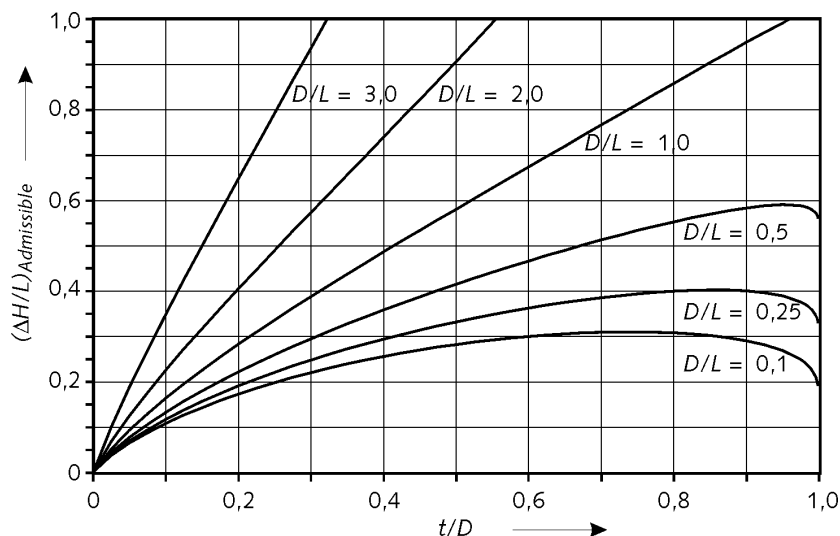


Figure 5.6.4 Calculation results 'Fragment Method' (see Figure 5.6.3)

5.6.4 Calculation rules for piping

The calculation rules of Bligh and Sellmeijer are used in the Netherlands to test for piping in which only horizontal seepage is taken into account, while in addition to horizontal seepage, the method of Lane takes the vertical seepage water into account. In recent years, when calculating the hydraulic head for river dikes in the west of the Netherlands, it has become common practice to take into account the reduction in loading resulting from resistance in vertical channels from the sand layer to ground level or base of the ditch. In laboratory tests, the potential hydraulic head has been measured for a column of sand that started to fluidize as a result of vertical seepage (*Ref. Piping due to flow towards ditches and holes*). Various tests indicate that the average potential hydraulic head over the fluidized column of sand is 0.6 times the height of the column. In a prototype situation in which the eroded sand from the sand layer is transported to ground level via vertical channels, on average, this is the hydraulic head that will occur. For the piping mechanism, the potential hydraulic head between the outer water and the exit point in the sand layer is normative. This hydraulic head is equal to the total hydraulic head over the soil structure, minus the hydraulic gradient over the vertical channel in the covering layer. If this phenomenon is taken into account along with the introduction of a safety factor 2 and the testing rule is then:

[EQUATION] 5.6.4

$$\Delta H_{strength} \geq \gamma \Delta H_{loading} \left(= \gamma (\Delta H - 0.3 d) \right)$$

In which:

$\Delta H_{strength}$	critical hydraulic head [m]
ΔH	hydraulic head over the soil structure [m]
d	thickness of the covering layer (Table 5.6.2) [m]
γ	safety factor [-]

The use of vertical seepage screens is currently under consideration. Seepage screens on the downstream side of soil structures are in general most effective, because usually a greater hydraulic head is permitted as a result of sizing for heave. For seepage screens on the upstream side of a soil structure, only the resistance of the flow is increased. For effective operation, it is often necessary to use seepage screens that seal the water-bearing sand layer.

The Lane method is only used when the methods of Bligh and Sellmeijer cannot be used. Although the above-mentioned method can be used for testing and designing soil structures for heave for new soil structures the less stringent rules for testing for heave (see § 5.6.3) are recommended. In some situations, the use of these rules is not possible, for example, when water penetrates behind the structure. The calculation model for two-dimensional seepage currents does not suffice in this situation but a three-dimensional analysis of the groundwater provides a solution.

Bligh

Bligh listed and analyzed a number of small dams that collapsed through piping (*ref. Dams barrage and weirs on porous foundations*). According to Bligh, the critical hydraulic head is a function of a material constant C_{creep} (see Table 5.6.1) and the 'line of creep' (also called the continuous line) from entry point to exit point (L).

[EQUATION] (5.6.5)

$$\Delta H_{sterkte} = \frac{L}{C_{creep}}$$

When applying Bligh's rule, the safety coefficient = 1.0; the required safety, of the size which is as yet unknown, is already included in the material constant.

Lane

One or more vertical seepage screens are sometimes used to extend the seepage line for hydraulic structures or soil structures located on a sandy substratum. In structures with seepage screens to prevent under and rear seepage in particular, the normative seepage lines can be very complex. Naturally, seepage water follows the way of least resistance. In (Ref.) *Technical report on sand bearing springs* an approach is given in which these types of seepage routes are systematically investigated. Lane (Ref. *Security from under-seepage in masonry dams on earth foundations*) has set down a summary of collapsed structures of which the seepage line consisted in part of vertical sections in the following rule:

[EQUATION]

5.6.6

$$\Delta H_{sterkte} = \frac{\frac{1}{3}L_h + L_v}{C_{w,creep}}$$

In this equation, L_h is the total length of the horizontal sections of the seepage line and L_v is the total length of the vertical parts of the seepage line. Note that the vertical seepage line along a screen is equal to twice the length of the screen. The material constant $C_{w,creep}$ is based on an upper limit that implies that when Lane's rule is used: $\gamma = 1.0$; Table 5.6.1 indicates the values given by Lane for various types of material in the soil layer.

Sellmeijer

In the context of Dutch TAW research Sellmeijer (ref. *On the mechanism of piping under impervious structures*) developed a calculation model to test for piping. It is valid for an idealized soil composition, or in other words for situations in which the water-bearing sand layer with homogeneous permeability and constant thickness extends far beyond the exit point. In reality, however, soil compositions are frequently encountered, in which the sand is anything but homogeneous. Nevertheless the model can still be used on condition that the product of layer thickness and permeability in the model is so chosen that the seepage current is simulated in the prototype situation.

The above mentioned calculation model was used to perform various numeric calculations, in which the hydraulic head and soil parameters differed. The results from these calculations were then used to calibrate an analytical calculation model. Sellmeijer's calculation model was validated on the basis of a large-scale model test in the Delta channel of the Waterloopkundig Laboratory. On the basis of the model tests in the piping guideline the calculation model of Sellmeijer was validated. From the model test in the guidelines on piping (Ref. *Technical Report on sand carrying springs*) the nominal values were determined for roll-resistance angle (θ), and slip force (η) these being $\theta = 41$ degrees and $\eta = 0.25$ respectively. There is no simple way to determine these parameters for different cases (expensive laboratory studies). With these parameters, however, the model error in the original model of Sellmeijer can be corrected. If these nominal values are used, Sellmeijer's original equation gives:

.)

[EQUATION]

5.6.7

$$\Delta H_{sterkte} = D^\beta L^{1-\beta} \left(\frac{gd_{70}^3}{\nu k L} \right)^{\frac{1}{3}} \left(0,25 - 0,037 \ln \left(\frac{gd_{70}^3}{\nu k L} \right)^{\frac{1}{3}} \right) \quad (5.6.7)$$

with

$$= \frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1}$$

In which:

$\Delta H_{\text{strength}}$	critical hydraulic head (=H _{sterkte})
D	thickness of the sand layer [m] (see Table 5.6.2)
D ₇₀	70-percentile value of the grain distribution [m] (Table 5.6.2)
g	gravitational acceleration [m/s ²]
k	permeability [m/s] (see Table 5.6.2)
L	length of seepage line length (measured horizontally) (m) (Table 5.6.2)
v	(1.33 * 10 ⁻⁶ m ² /s at 10 ⁰ Celsius) kinematic viscosity of water

Table 5.6.1 Creep factors for the rules of Bligh and Lane

Type of soil	Median Grain diameter μm	C _{creep} (Bligh) (γ = 1)	C _{w, creep} (Lane) (γ = 1)
Extremely fine sand, silt	<105		8,,5
Very fine sand	105-150	18	
Very fine sand (mica)		18	7
Moderately fine sand (quartz)	150-210	15	7
Moderately coarse sand	210-300		6
Very/extremely coarse	300-2000	12	5
Sand			
Fine gravel	2000-5600	9	4
Moderately coarse gravel	5600-16000		3.5
Very coarse gravel	>16 000	4	3

¹ Indications in accordance with NEN 5104 (September 1989)

If necessary it is possible, to interpolate between the medians of class of median grain diameters.

Table 5.6.2 Representative choice of parameters

Parameter	Description	Type of representative value	Comments
-----------	-------------	---------------------------------	----------

D	Layer thickness covering layer	l.r.v.	$V_c = 0.10$
D ₇₀	70-percentile sand	l.r.v.	$V_c = 0.25$
D	Thickness sand layer	h.r.v.	$V_c = 0.10$
K	Permeability	h.r.v.	(see text)
L	Length seepage line	l.r.v.	$V_c = 0.10$

Representative or characteristic values:

h.r.v. $\mu(1 + t_{N-1}^{0.95}V_c)$ high representative value or 95% upper limit)

l.r.v. $\mu(1 - t_{N-1}^{0.95}V_c)$ low representative value or 95% lower limit)

μ average value from sample, or best estimate

V_c variation coefficient from sample or default variation coefficient; see table

$t_{N-1}^{0.95}$ student t-factor (if no sample is available: 1.65)

Example of the determination of k if no random sample is available;

$$k_{\text{calculation value}} = k_{\text{average}} (1 + 1.65 \cdot 0.1) = 1.16 k_{\text{average}}$$

Indicative values for the permeability of sand containing negligible amounts of silt (see also § 4.3.9).

Grain size d_{50} mm [mm]	Permeability k (mm/s)
0.1	0.06
0.15	0.14
0.2	0.24
0.3	0.54
0.4	1.00

In contrast to the approach of Bligh and Lane, which was derived from practice, in testing the experimental observations and the results of calculations Sellmeijer used the most practicable estimate. Attention has also been devoted to partial safety. On the basis of this extensive study into reliability, in validating Sellmeijer's equation a safety coefficient of $\gamma = 1.2$ was derived (ref. *Technical report on piping in river dikes*).

It may occur that, with Sellmeijer's formulation, a larger required seepage length is given than the maximum required seepage length, $18 \cdot \Delta H$, according to Bligh's rule. In such situation, Sellmeijer's rule has not been validated and in current consultancy practice there is still consensus for choosing Bligh.

5.6.5 Improvement and management measures

Measures for new or existing structures are usually designed according to the Sellmeijer (Equation 5.6.7) or according to the Heave Criterion (Figure 5.6.4). For structures in tidal areas in particular, it is useful to take into account non-stationary aspects. Various possible measures can be taken to prevent the formation of sand carrying springs. The differences primarily relate to the fact that the measures affect different aspects of the failure mechanisms. The possible measures are:

- Extending the seepage line, either horizontally or vertically
- Preventing heave of the clay layer within the critical seepage line
- Reduction of the hydraulic head
- Prevention of the washing out of sand

The above mentioned aspects are explained in (Ref.) *Technical report on sand bearing springs*

Observations during periodic inspections or at high water can provide valuable supplementary information. All relevant aspects are investigated, such as height of the inner and outer dike ground surface levels, the functioning of filters or filter constructions and the functioning of seepage screens. With regard to sand carrying springs, during high water it is necessary to observe whether water is expelled on the inner side of the dike and, if this is the case, whether the water is transporting sand. If spring formation, with or without sand transportation, is observed while on the basis of the available data it is not to be expected (at least at the observed rate), all the bases of the calculations should be carefully adjusted. If no explanation can be found it is advisable to carry out a local investigation. If the flow of springs increases in comparison with earlier high waters, it is also advisable to commission further investigations. In figures 5.6.5 and 5.6.6 points of special interest in relation to heave and piping are indicated.

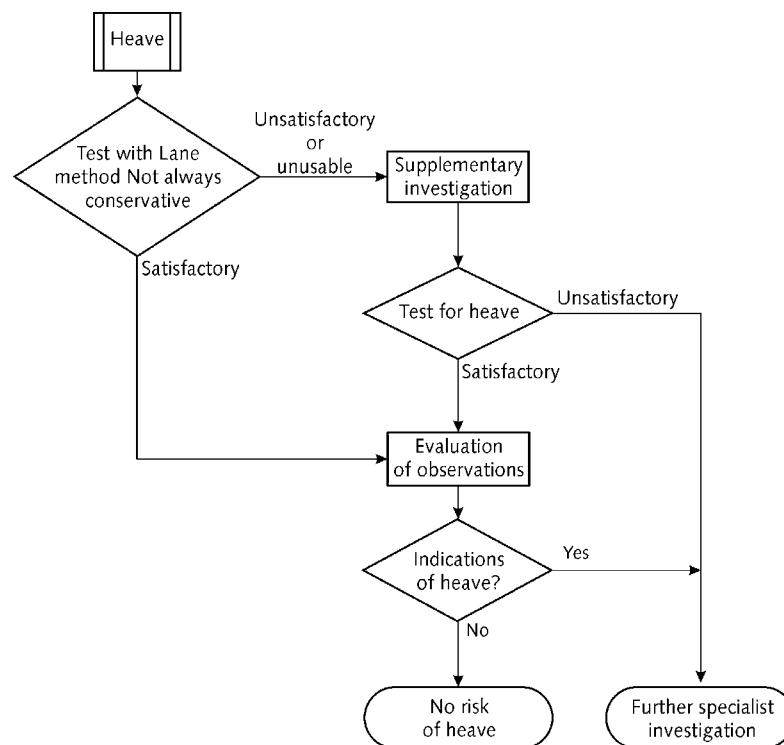


Figure 5.6.5 Heave in soil structures

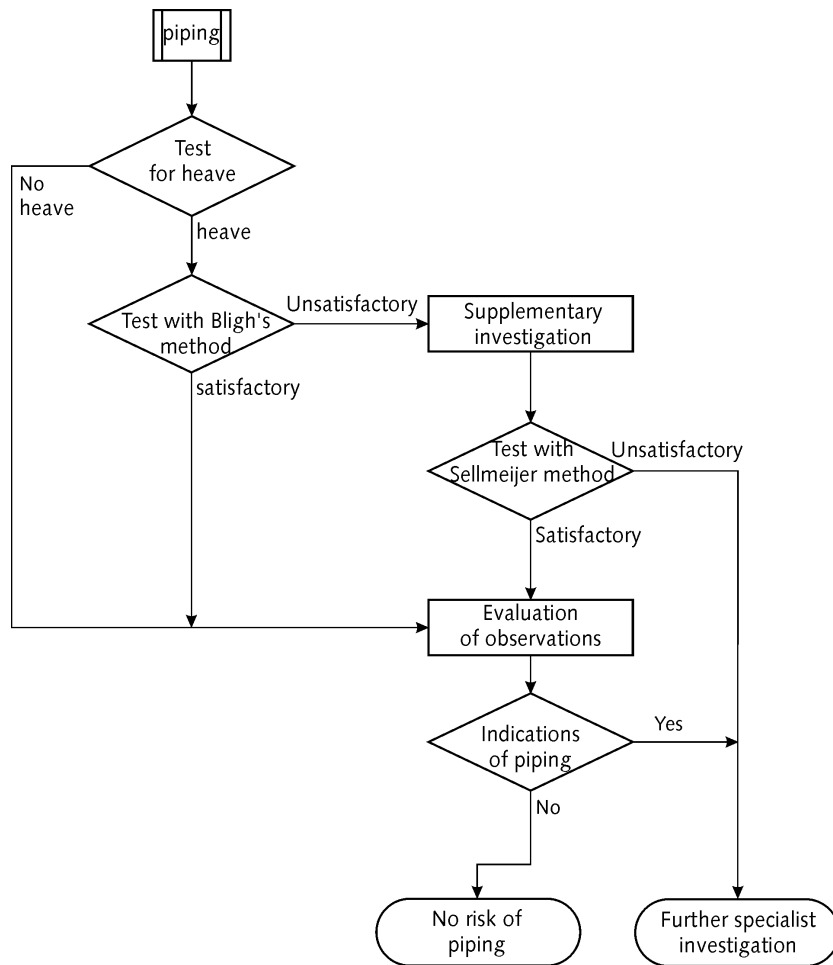


Figure 5.6.6 Piping in soil structure

5.7 Stability forebank

5.7.1 Introduction

For earth structures, whether or not they have a forebank of limited length along a slope that is located under the waterline, account should be taken of the occurrence of instability of this forebank. This affects stability of the soil structure and therefore the water-retaining capacity. In the event of possible instability, a distinction is made between instability as a result of shearing (§ 5.4) and instability as a result of settlement flow (§ 5.7.1). In § 5.7.2 the processes involved in these phenomena are described while § 5.7.3 covers the calculation rules for describing the above-mentioned instabilities.

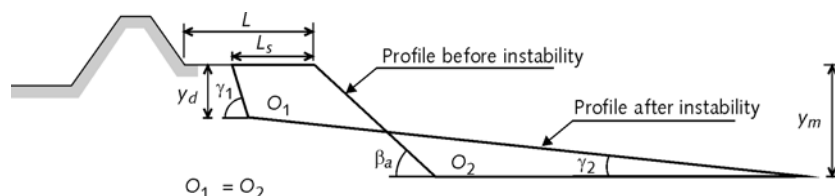


Figure 5.7.1 Two-dimensional absorption model with unlimited absorption

y_d height of breach
 L width of foreland

L_s	erosion length
y_m	maximum depth of water in flow channel
O_i	area
β_a	average angle of slope occurring before the instability
γ_1	side slope close to the bank/earth structure
γ_2	sedimentation angle of slop after the instability

5.7.2 Flow □□

□

Flows along soil structures can cause erosion of the foreland and in other ways exert forces on a revetment layer (see Chapter 7). The water itself and also the materials or objects that it carries can erode the revetment; the forces exerted by currents can transport loose rip-rap and the edges of the fascine collars and/or mattresses can be lifted and folded over, as a result of which the underlying layers are exposed to erosion.

Natural flow caused by the discharge of water from the upper river areas and the tide is often a hydraulic boundary condition for the design of water retaining structures that is of subordinate importance. In places where erosion is frequently caused by flowing water usually measures to control this are taken by the water management. Examples of such measures are dikes adjacent to waterways (schaar dikes), and forebanks for earth structures along the edge of a tidal channel. For forebanks the development of the flow channel must be kept under observation: if the slopes of the forebank are too deep and too steep they may themselves be the mechanisms that initiate settlement flows or shearing. With instabilities geotechnical aspects (strength), and hydraulic and morphological conditions (loading) all play a role (see Figure 5.7.2).

□

5.7.3 Shearing and settlement flow

Shearing can occur in forebanks that consist of cohesive soil such as clay and peat and/or sand that is not prone to weakening. If there is a relatively weak layer of limited thickness strength properties (cohesion and angle of internal friction) under the foreland, it is necessary to pay attention to possible shearing along a straight slip plane. This is particularly true if the bed in question is sloping down to the underwater slope.

For soil structures, whether or not they have a forebank of limited width along an underwater slope, in loosely packed sand account must be taken of the collapse mechanism settlement flow. This is a mechanism in which a mass saturated with water undergoes very large displacements, or 'flows' as a result of weakening. Weakening of sand on a slope is caused by an unfavourable combination of loose packing and slope geometry. There must also be a causative factor: the weakening occurs after an increase in shear stress that is sometimes only very small. When this occurs the grain skeleton is reorganized, resulting in a decrease in volume and causing an increase in pore pressure of such proportions that the contact pressure between the grains is reduced to a critical degree. At this point the sand mass starts to behave like a heavy liquid, resulting in settlement flow.

The causes of settlement flow include:

- steepening of the underwater slope in the channel as a result of erosion
- an increase in height or loading from above
- vibrations, for example resulting from pile driving, explosions, earthquakes, the collision of a vessel and dredging activities.
- wave loading for example, during a severe storm; this loading causes variations in pressure and grain skeleton deformations in the soil (wave penetration process). These may be so severe in nature that the sand weakens, giving rise to settlement flow.
- rapid decrease in water level.

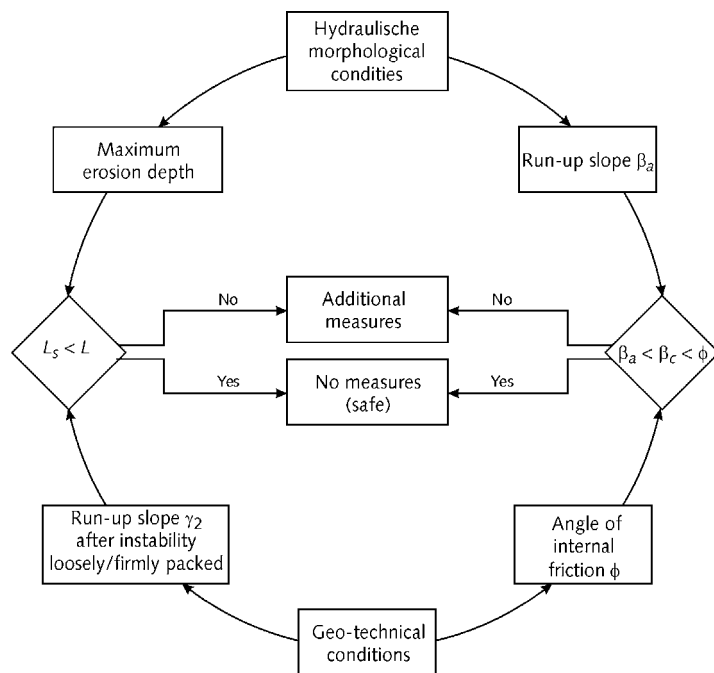


Figure 5.7.2 □ Relationship between strength and loading parameters

There is a clear difference between settlement flow and shearing, although there is a grey area between these phenomena. From the point of view of soil mechanics, settlement flow arises as a result of water overpressure in the soil. Shearing, on the other hand, is the result of exceedence of the shear resistance of the soil. Shearing can cause settlement flow.

To prevent the formation of slopes in the channels, which can cause instability of the water defence, the position of the slope can be fixed by dumping or sinking material onto it, or by a combination of these. Fixing the slope does not affect the sensitivity to weakening of the loosely packed sand: its only purpose is to prevent the development of an even steeper slope. Other measures, such as compaction, are either extremely difficult to carry out in practice, or are very expensive. The detection of increasing steepness of the forebanks, followed by the dumping of material onto these forebanks, which almost entirely prevents settlement flow and shearing, is a successful method used in Zeeland.

To summarize, both geo-technical (strength) and hydraulic and morphological (loading) issues play a role in shearing and settlement flows (see Figure 5.7.2). If the side slope (β_a) of a flow channel exceeds a critical value $\beta_c (= \phi/\gamma_\eta)$ in which γ_η is a safety factor, see § 5.7.3) and/or when the width of the foreland (L) is smaller than the erosion length (L_s), measures have to be taken to safeguard the stability of the soil structure.

5.7.4 Calculation rules

The density of the soil in the foreland determines whether settlement flow will occur. In addition to density, the angle of the side slope of the channel also plays a role in settlement flow. If this occurs, the resulting end slope γ_2 and the erosion length are characteristic features. On the basis of various recorded plate and bank collapses that have occurred in the south-west of the Netherlands, calculation rules for predicting settlement flows and shearing were developed in the context of the Delta works (*Ref. Orientation study into soil mechanical aspects of scour holes*).

The stability of the forebank with regard to shearing along circular or straight slip planes can be investigated in the way described in § 5.4. For settlement flows, scenarios with circular slip planes cannot be used. The SLIQ2D calculation model predicts the occurrence or non-occurrence of settlement flows in loosely packed sand for specified geometrical and soil conditions. The model assumes 'static' weakening and is not suitable for assessing

cyclical weakening (earthquakes, wave loading). Further information regarding SLIQ2D can be found in the *Settlement Flows Handbook*).

Monitoring settlement flow

Attention must be paid to the possibility of settlement flow occurring if:

- The depth of the channels exceeds 9 metres but the dike is relatively close to the underwater bank of the channel (see text on channel depth).
- The angle of slope of the channel is too steep and the erosion length L is too long (see text on side slopes of the flow channel and erosion length).

In these cases it is necessary to determine whether the forebank consists of sand, whether this sand is loosely packed and to what degree this sand is predisposed to weakening (see text on packing density).

The phenomenon of settlement flow is often considered in relation to the geological history. In natural soil beds that were relatively rapidly deposited loose packing may easily occur: there is a good chance that settlement flows have previously occurred in such beds. Examples of this can be found in the estuaries in Zeeland and at IJmuiden and Eemshaven.

It is a mistake to think that settlement flow along rivers is impossible. Forelands can consist of sand fill that has partially placed under water. When dikes are strengthened on the outer side much depends on the structure of the underwater foundation. In the past rough material such as mine stone was often used. Now that owing to environmental considerations, mine stone is less often used, there is a tendency to use sand for this purpose. Therefore in most cases it is necessary to densify this sand to sufficiently limit the risk of settlement flow

Packing density

If tests have to be performed for the collapse mechanism causing settlement flow, in view of the existing geometry, it is necessary to investigate the degree to which the in situ sand is predisposed to weakening. Sampling and, if necessary, grain distributions from drilling samples can provide an initial picture of the packing (loose, medium, solid) and the gradation of the sand, and thus provide a general impression of the sensitivity of the sand to weakening. For a more accurate picture of the sensitivity to weakening, a density survey consisting of drilling sampling and electrical density measurements (see § 4.4), in combination with calibration values to calculate the pore content of the sand in situ can be carried out. The grain distribution, maximum and minimum density, the dry and/or wet critical density, and also the calibration values for the electrical density measurements can be determined from sand samples from the drilling. The electrical density measurements, in combination with the calibration values, can be used to determine the pore content of the sand in situ.

Comparing the pore content of the sand in situ with the maximum and minimum density gives an impression of the relative density of the sand present. The relative density (n_r) is defined as the proportional value that is obtained by dividing the difference between the maximum ($n_{\max}$) and the pore content present (n) by the difference between the maximum and minimum pore content ($n_{\min}$).

EQUATION (5.7.1)

$$n_r = \frac{n_{\max} - n}{n_{\max} - n_{\min}}$$

As shown in this equation, the relative density can vary between 0 for the most loosely packed sand and 1 for the most solidly packed sand. The area between these two limits can be divided into three equal components, in which the relative density is classified as follows (Table 5.7.1):

Table 5.7.1 Density of fine sand

Degree of density of fine sand	Relative density
Loose packing	0.00 - 0.33
Medium packing	0.33 - 0.67
Solid packing	0.67 – 1.00

Comparing the density in situ with the dry and wet critical density gives an impression of the propensity of the sand to weakening. The pore content associated with the wet critical density can be considered to be an upper limit, while that associated with dry critical density considered to be a lower limit. The following criteria can be set:

- If the pore content in situ is greater than the wet critical density, the sand is certainly predisposed to weakening
- If the pore content in situ is smaller than that of the dry critical density, sand is not predisposed to weakening
- If the pore content in situ is between that of the dry and the wet critical density, the sand may be predisposed to weakening. The closer the density is to the dry critical density, the smaller the predisposition to weakening, and the closer the density is to the wet critical density, the greater the predisposition to weakening.

It should be noted that the dry and wet critical densities are a function of the pressure level. The greater the average pressure (the higher the slope), the greater the predisposition of the sand to weakening. Dry critical density tests show that the looser the packing of the sand the higher the average pressure, the greater the decrease in volume with increasing shear pressure, and therefore the greater the probability of weakening. These influences are quantified in calculation model SLIQ2D.

Channel depth

As limit of the channel depth above which settling flow forms a serious problem a maximum channel depth of 9 metres is given in the (Ref.) *Safety Monitoring Guidelines*. This is a somewhat arbitrary figure, for which it is difficult to give a better value, and which has been included in this Technical Report to maintain some agreement, although it is important to take careful note of this fact.

Since a foreland or forebank bordering on a dike is under consideration there is a chance that the pressure condition of the underwater slope is influenced by the presence of the part of the dike that is above water, especially if the foreland is not very wide. This is even more likely because wet sand above water level weighs about twice as much as sand below water level. This can be taken into account in the following way (Figure 5.7.3):

- *Step 1*: make the slope above water twice as long as it really is; assume an extremely low water level.
- *Step 2*: make a shallow slope for the dike and the foreland by joining the most left and right points where the slope changes.
- *Step 3*: determine the fictive underwater slope that has to be taken into account in the equation.

EQUATION (5.7.2)

$$h_{fictief} = h_{onder} + h_{boven} \frac{\cot \alpha_{onder}}{\cot \alpha_{boven}}$$

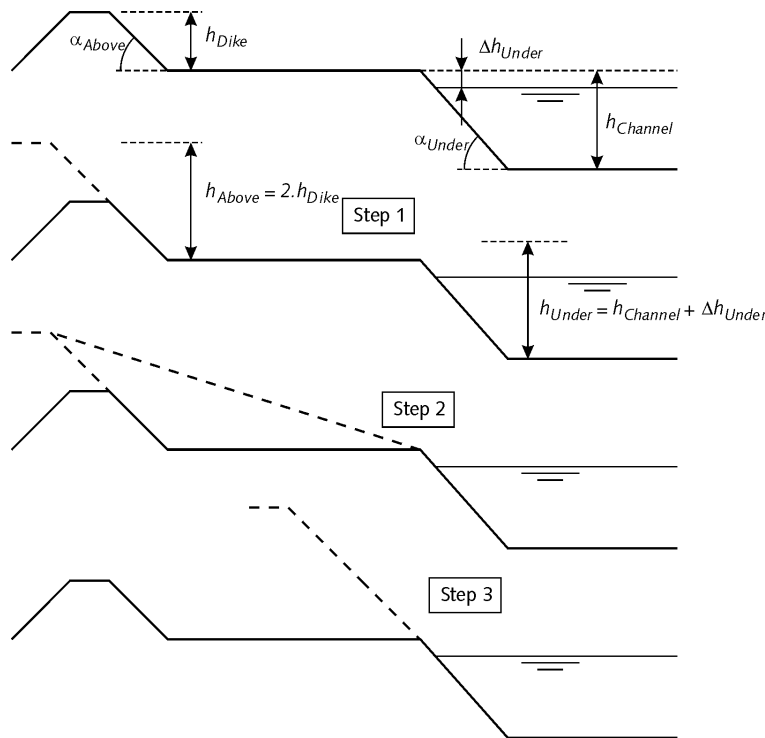


Figure 5.7.3 Schematization of saturated dike body in the vicinity of a channel

In the following example there is a 5 metre high dike with an outer slope gradient of 1:3. In front of the dike is a 25 metre wide foreland and in front of the foreland a 5 metre deep channel and a gradient of 1:3. With extremely low water the foreland is 1 metre above water. If attention is restricted to the depth of the channel, this is less than 9 metres and it is not necessary to consider the possibility of settlement flow. However that this is not so is revealed when steps 1, 2 and 3 are followed.

- *Step 1*: the 5 metre high dike is given a fictive height of 10 metres. One metre of the lower slope is above water. The height of the lower slope is therefore 6 metres and not 5 metres.
- *Step 2*: the dike (10 metres high and with a gradient of 1:3, that is 30 metres wide), the berm (25 metres wide) together make a single slope that is 10 metres high and 55 metres wide, in other words a gradient of 1:5.5.
- *Step 3*: the fictive underwater slope is:

EQUATION without a number

$$h_{fictief} = h_{onder} + h_{boven} \frac{\cot \alpha_{onder}}{\cot \alpha_{boven}} = 6m + 10m \frac{3}{5,5} = 11,5m$$

Thus it is indeed necessary to take into account the chance of settlement flow. The situation is considered as an 11.5 metres high underwater slope with a gradient of 1:3.

Side slope of the current channel

Two parameters are generally used when describing the shape of the current channel; the maximum depth of the water in the channel and the angle of the side slope. The angle of the side slope (β_a) is determined by the current and the waves (see ref. *Scour Manual*) and must at all times be smaller than the critical side slope (β_c). Likewise, (β_a) must be smaller than the angle of internal friction (ϕ). In the *Safety Monitoring Guidelines*, the following test criteria are used for a channel depth greater than 9 metres. Shearing occurs when the maximum slope is steeper than 1:2 (clay without sand 1:1) over a height of at least 5 metres, or the total slope from channel edge to channel base is steeper than 1:4.5). The occurrence of

settlement flow is possible if *either* the maximum slope is steeper than 1:4 over a height of at least 5 metres, or the total slope(channel edge to channel base) is steeper than 1:7.

Erosion length□

The erosion length depends closely on the storage capacity in the current channel next to the foreland (Ref. *Results of the preliminary investigation of Zeeuwse currents*). If the base of the current channel is assumed to be horizontal, the erosion length can be approached as follows (Ref. *Orientation study into the soil mechanical aspects of scour holes*)

[EQUATION] (5.7.3)

$$L_s = y_D \left(\frac{1}{2} \frac{y_D}{y_m} - 1 \right) (\cot \gamma_2 - \cot \gamma_1) + \frac{1}{2} y_m (\cot \gamma_2 - \cot \beta_a)$$

This equation is based on a two-dimensional retention model in which the volume of the potential shearing sand is equal to the amount that has sheared off (so $O_1 = O_2$, see Figure 5.7.1). Although, in reality, instability is a three-dimensional phenomenon and not a two-dimensional phenomenon, the statistics applied have been converted into a two-dimensional model. Table 5.7.2 provides a summary of the average and characteristic upper limits for the parameters in the storage model in question. These results are based on some 200 instabilities that have occurred in the last 150 years as a result of shearing and settlement flows (Ref. *Results of the preliminary investigation of Zeeuwse currents*). If it is assumed that $y_D = 0$, this gives the following rough upper limit:

[EQUATION] 5.7.4

$$L_s = \frac{1}{2} y_m (\cot \gamma_2 - \cot \beta_a)$$

When assessing the stability of the forebank, the erosion length calculated must be less than the width of the foreland present. The use of equation (5.7.3) in which for $\cot \gamma_2$ the characteristic upper limit must be taken ($\cot \gamma_2 = 20$) is recommended.

Table 5.7.2 Statistical parameters for settlement flow and shearing

Loading parameters	Settlement flows and shearing	Settlement flows	Shearing
	Average values	Characteristic upper limit	Characteristic Upper limit
$\cot \beta_a$	3.7	2	2
$\cot \gamma_1$	2.7	-	-
$\cot \gamma_2$	16.3	20	8
y_d / y_m	0.43	-	-
L_s / y_m	1.7	9	3

6 Optimizing the transverse profile

6.1 Introduction

The basis for optimization of the transverse profile is the analysis of the structural design (see Chapter 2). Dike elements are used in the method and the entire process of optimization is an approach that proceeds from the generalized to the detailed. The objective of this iterative process is that the user makes optimal choices, which continue to meet the reliability demands. The following issues play a role in optimization:

This chapter gives supplementary information about the optimization of the transverse profiles of a soil structure. Section 6.2 describes the design variable, time dependent aspects and optimizations directions that form part of optimization scope. Optimization is described from the points of view of both hydraulic loading and the strength of the soil body in § 6.3. Section 6.4 covers the traditional soil investigation and how actual soil investigations must be carried out. In § 6.5 and § 6.6 attention is paid to the optimization of the subsoil and dike improvement. In the same way two optimization problems are worked out (§ 6.3. and § 6.6).

6.2 Scope for optimization

6.2.1 Introduction

The optimization differs from phase to phase of a project. The table below provides a picture of the characteristics for each phase.

6.2.2 Design characteristics

The designer should bear safety in mind throughout the entire process. On one hand this is governed by the loading and on the other hand by the strength. Table 6.2.1 gives a general outline of this process.

Table 6.2.1 Relation between design characteristics

Safety norm	
Loading (boundary condition)	Strength
Above ground level Choices	Below ground level Boundary conditions
- soil elements - dimensioning - management and maintenance - execution	

It is important that in each subsequent phase, the freedom to optimize the design is reduced. The design process must not be restricted to the construction of the dike (by generating alternative structures), but should also take into consideration the nature of the substratum. The subsoil is an essential starting point for the design. In contrast to the loading, the degree of precision with which this process is followed determined by the designer. The following issues play a part in the optimization process:

- The choice of a combination of soil elements and materials, with a view to their structural functions.
- It is possible to use various combinations of elements in order to meet the demands with respect to water-retaining capacity. Various elements can be used per failure mechanism; for example, resistance to piping can be enhanced by the application of an impervious layer on the foreland, by sealing the water-bearing package, or by applying a piping berm. However, elements often influence more than one failure mechanism. A piping berm, for example, also affects the stability of the inner slope. On the basis of Table 2.7.1 in Chapter 2, it is possible to ascertain whether it is necessary to optimize the design with respect to a particular aspect and determine which dike elements influence this. The same table indicates which collapse mechanisms are affected if the configuration of one dike element in the design is adapted.
- The sizing of the elements, bearing in mind the soil properties. The design must be based on the layer structure of the substrata and the dike body, the layer properties to be used and the associated predicted time-dependent development of pore pressure. As the reliability of the soil-mechanical and geo-hydrological investigation increases, it will be possible to estimate these parameters less conservatively and thus improve the accuracy of the sizing process.
- Optimization also involves management and maintenance procedures. The management and maintenance must be attuned to maintaining sufficient strength in the various elements. This must take into account the importance of the element in the design and the sensitivity of the element to a possible reduction in strength. For example, this may include maintaining an adequate covering layer on the foreland, keeping drainage free of obstruction and monitoring drainage.
- The implementation phase may be a critical factor in the choice and size of elements.

There are roughly three stages in the optimization process that can be further refined in the phasing of the project (see § 6.2.3). With regard to earth structures, when considering the height of the crown it is possible to envisage problems relating to erosion and stability. These concern both the loading and the strength of the system characteristics (see Table 5.2.2. The calculation of the safety falls into two parts: a norm to which the overtopping capacity must conform when the height of the crown and a norm to which the remaining parts of the structure must satisfy. In the year 2000 the 10% of the norm was used for stability assessment of the water retaining structures. To date no simple norms have been established for erosion.

The above-mentioned optimization of the transverse profile can be made on the basis of several grounds such as cost, space required and LNC values. In addition to the constructional requirements the design should take into account the foundations for an integrated design and possible changing in conditions,

Table 6.2.2 Relation between loads and failure mechanisms

FUNCTION	FAILURE MECHANISM	LOADS	MOST IMPORTANT LOADING PARAMETERS	SYSTEM CHARACTERISTICS	RESPONSE CHARACTERISTICS
HEIGHT	Settlement	Weight	Density of materials Degree of saturation pore water pressure	Compressibility Permeability Layer thickness	Lowering Horizontal displacement
	Wave overtopping	Waves Currents Ice			

STABILITY	Shear	Weight	Material weight Angle of slope Pore water pressure (under the influence of waves)	Angle of friction Cohesion and Permeability of the core, substratum and top layer	Shearing part of structure Failure
	Settlement flow	Waves Earthquakes	Wave height Wave period Pore water pressure Shear stress Frequency and number of changes in loading	Permeability Compression grade Layer thickness Angle of friction	Large d Failure
EROSION	Migration of under layers and/or filters	Waves (tides) Bow waves (ships) Falling water levels	Hydraulic gradient Internal water movement	Permeability and layer thickness	Internal materia
	Piping	Decline/drop	Flow velocity in the pipe	Pipe length Hydraulic resistance grain diameter	Internal materia
	Erosion	Waves Ice	Orbital velocity Flow velocity Turbulence	Grain diameter Angle of slope Permeability	Erosion c

6.2.3 Project phasing

The freedom, (that is the means), which make optimization possible, clearly differs from phase to phase. Table 6.2.3 gives an overview of the characteristics of each phase. It is important that for each subsequent phase the freedom to optimize the design is reduced. At the same time the uncertainty about the conditions in the following phase also decreases. In practice, when proceeding through the project phases an increasingly more reliable (detailed) image of the type of subsoil will be required. A more reliable and detailed understanding of the subsoil is needed because with a fixed safety criterion it is possible to take into account the uncertainties in the design. One freedom that remains until the last phase is the reduction in the uncertainty in the subsoil model. The balance between the gathering of extra information about the subsoil and the design advantage that this yields (or testing) is considered in § 6.4.

Testing

At the testing stage, there is no longer any freedom because the water defence already exists and can only either pass or fail. This means that, with respect to soil investigation and knowledge of the substratum, the freedom remains limited to charting the upper structures and substratum. The reduction of the uncertainty in the model of the substratum is the last opportunity to prevent failure caused by design errors. While detailed measurements and calculations are of little use in the design, they are very useful in testing. If, despite additional soil surveys, the dike still fails the test, this information can be included in the design.

Table 6.2.3 Characteristics project phasing for water defences

Project phase	Objective of the	Criterion	Design
---------------	------------------	-----------	--------

	project phase		freedom
Definition/plan development	To draft the design	Sufficiently accurate design with respect to types of dike	Very large
Pre-design	Pre-design	Sufficiently accurate design with respect to possible design variants	Large
Design	Designing	Safety demand with respect to sizing	Limited
Construction	Realization	Safety demand as test	Very limited
Use/management	Testing	Safety demand as test	Zero
Demolition/reinforcement	Demolition or adaptation	Safety demand for the improved design	Limited

6.2.4 Possibilities for optimizing the transverse profile

□

On the basis of the individual and integrated role of a soil element of which the dike is composed, there are a number of possibilities for optimizing the transverse profile. Some of these, such as reduction in slope angle, the use of lighter reinforcement materials and more erosion-resistant slope revetments are the obvious choices (see § 6.3). Other possibilities are less obvious, but under certain circumstances may certainly be worth considering. These include:

- Additional soil surveys to better determine the layer structure (see § 6.4.4, example 1)
- Additional soil surveys to better estimate the soil parameters (see § 6.4.4, example 2)
- Demarcating problem processes (with additional soil surveys) (this is covered in § 6.4.4, example 1).
- Soil improvement for weak or permeable substrata (see § 6.5)
- When reinforcing a dike the following choice is also important: the position of the reinforcement inside, outside or on both sides of the dike (see § 6.6)]
- Advanced calculation models, reliability analysis (see Chapter 5). Upgrading parameters with current strength with local soil survey, or proven strength on the basis of past loading situations (*Ref. Technical report on actual strength of dikes*. Still in preparation)
 - Pseudo characteristic values (see Annex 1)

6.3 Soil elements and materials of a dike

6.3.1 Introduction

A dike protects the hinterland and the population from calamities such as dike breach and flooding. It consists of a number of soil elements (crown, core, substrate, outer slope, inner slope, revetment, berm, foreland and hinterland) that play both integrated and individual roles in defence.

A closer scrutiny of the functional role of the soil elements mentioned with respect to threats, in conjunction with the local situation, provides prior knowledge of the optimization options for a transverse profile that is considered representative for a dike section. This is a

clear association with Chapter 2. This section is restricted to consideration of the optimization of the transverse profile of a dike in which the following variables are considered:

- Gradient and shape of the outer slope (§ 6.3.2);
- Strength of crown and inner slope (§ 6.3.2);
- Revetment of outer slope (§ 6.3.4);
- Gradient of inner slope and shape of inner berm (§ 6.3.5);
- Structure of the dike and provisions for drainage (§ 6.3.6);

6.3.2 Gradient and shape of the outer slope

If wave run-up is reduced the crown can be lower. This can be achieved by:

- A gentle outer slope
- A high foreland
- A wave retarding construction
- A rough outer slope
- An outer berm of the correct height

A gentle slope

Figure 6.3.1 gives an example of a berm at the level of the NHW. For this example the consequences of varying the conditions for the dike are shown in tabular form. The tables include the height of the dike table, the area (volume/m³) of the transverse profile, the width in relation to the cost of purchasing ground, the area of the revetment and a general comparison of the costs. It is emphasized that this example serves only an illustration and that in practice it is unlikely that a grass cover would be used on the lower part of the outer slope. The prices mentioned are also fictive. The conclusion that can be drawn from this example is that if options without a berm or with a steep slope are not considered, all dikes with revetments are equally expensive. If the need to use revetments can be avoided by choosing a slope of 1:8, a cheaper option can be found. Thus it does not seem to be possible to make a choice on the basis of this simple comparison. Much more detailed consideration is necessary.

A high foreland

With a high foreland (if h/H , < 3 or 4) the highest waves break on the foreland. Here h is the depth of the water and H the significant wave height, (both at the toe of the structure).

A wave retarding construction

Because of the very high cost, wave-retarding structures are only considered if a low crown is required for a very short distance. For the design of these special structures it is necessary to enlist specialist aid or to carry out model tests.

A rough outer slope

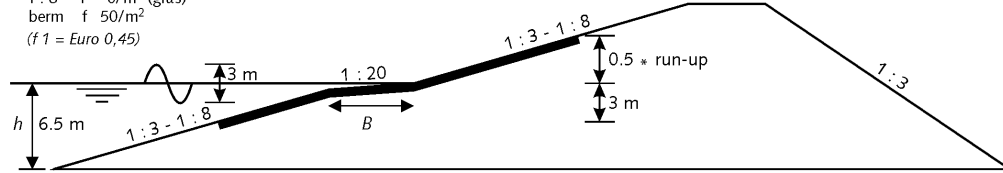
A rough outer slope can be made by using special types of block for the revetment, by applying roughness elements on smooth revetments and by the use of rubble. The latter is the most efficient, giving wave run up reduction of up to 50%. The reduction coefficients are given in the *Technical report on wave run-up and overtopping of dikes*.

An outer berm of the correct height

An outer slope works optimally in reducing wave run-up when it lies at approximately the same height as the waterline, which for design conditions is at the level of NHW + possible local additions. The *Technical report on wave run-up and overtopping of dikes* explains how the reduction can be calculated for different widths and heights.

Boundary conditions/assumptions:

- Short crested waves ($H_s = 3$ m; $T_p = 8$ s)
- Shallow water ($d = 6.5$ m)
- Wave approach perpendicular to dike
- Smooth revetments for slopes 1:3 and 1:5 ($\gamma = 1$)
- Grass covered revetments for slopes 1:8 ($\gamma = 0.95$)
- Revetment extends from 3m under the berm to half the wave run up height above the berm
- For slopes of 1:8 a cover of grass is used
- Price of revetment material:
 - 1:3 f 120/m²
 - 1:5 f 80/m²
 - 1:8 f 0/m² (gras)
 - berm f 50/m²
- (f 1 = Euro 0,45)



schematic illustration of boundary conditions/assumptions

Dike table height (m)									
	1 : 3			1 : 5			1 : 8		
	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10
Berm width B	0 m	17,9	15,4	12,9	13,2	11,7	10,2	9,5	8,6
	5 m	15,7	13,7	11,7	12,1	11,1	9,8	10,2	9,3
	10 m	14,3	12,6	10,8	11,9	10,7	9,5	10,0	9,2
	15 m	13,4	11,9	10,4	11,4	10,3	9,2	9,8	9,0

Volume of the transverse profile per metre of dike (m ³ /m) (Price of soil at f15m ³)									
	1 : 3			1 : 5			1 : 8		
	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10
Berm width B	0 m	997 (f 14.955)	742 (f 11.134)	525 (f 7.875)	723 (f 10.850)	437 (f 6.548)	616 (f 9.235)	515 (f 7.731)	424 (f 6.360)
	5 m	803 (f 12.051)	623 (f 9.345)	467 (f 6.999)	642 (f 9.635)	548 (f 8.213)	625 (f 9.377)	527 (f 7.902)	447 (f 6.703)
	10 m	707 (f 10.606)	567 (f 8.497)	437 (f 6.548)	655 (f 9.829)	544 (f 8.165)	635 (f 9.525)	549 (f 8.234)	470 (f 7.048)
	15 m	663 (f 9.945)	546 (f 8.192)	443 (f 6.642)	640 (f 9.602)	543 (f 8.136)	645 (f 9.680)	561 (f 8.415)	493 (f 7.395)

Width of revetment (Price at f17/m ²)									
	1 : 3			1 : 5			1 : 8		
	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10
Berm width B	0 m	109 m (f 820)	94 m (f 708)	79 m (f 596)	108 m (f 807)	96 m (f 717)	84 m (f 627)	116 m (f 873)	107 m (f 799)
	5 m	101 m (f 759)	89 m (f 669)	77 m (f 579)	104 m (f 778)	96 m (f 718)	85 m (f 640)	119 m (f 894)	109 m (f 820)
	10 m	97 m (f 734)	88 m (f 657)	77 m (f 576)	107 m (f 804)	98 m (f 732)	88 m (f 660)	122 m (f 915)	113 m (f 849)
	15 m	97 m (f 730)	88 m (f 663)	79 m (f 596)	108 m (f 812)	99 m (f 746)	91 m (f 680)	125 m (f 936)	116 m (f 870)

Wave run-up, area of slope revetment, cost of slope revetment									
	Slope	1 : 3			1 : 5			1 : 8	
Berm width B	0 m	8,3 m 22,6 m ² (f 2713)			5,0 m 28,0 m ² (f 2244)			3,0 m 0 m ² (f 250)	
	5 m	6,7 m 19,8 m ² (f 2630)			4,4 m 26,3 m ² (f 2351)			2,8 m 0 m ² (f 250)	
	10 m	5,6 m 17,8 m ² (f 2641)			4,0 m 25,0 m ² (f 2500)			2,6 m 0 m ² (f 250)	
	15 m	5,0 m 16,6 m ² (f 2747)			3,6 m 23,7 m ² (f 2648)			2,5 m 0 m ² (f 250)	

Total cost									
	Slope	1 : 3			1 : 5			1 : 8	
	overtopping	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)	10	0,1	1,0 (l/s/m)
Berm width B	0 m	f 18.489 (100 %)	f 14.555 (79 %)	f 11.184 (60 %)	f 13.901 (75 %)	f 11.525 (62 %)	f 9.419 (51 %)	f 10.108 (55 %)	f 8.529 (46 %)
	5 m	f 15.439 (84 %)	f 12.643 (68 %)	f 10.207 (55 %)	f 12.765 (69 %)	f 11.283 (61 %)	f 9.536 (52 %)	f 10.521 (57 %)	f 8.972 (49 %)
	10 m	f 13.980 (76 %)	f 11.795 (64 %)	f 9.765 (53 %)	f 13.132 (71 %)	f 11.397 (62 %)	f 9.835 (53 %)	f 10.940 (59 %)	f 9.583 (52 %)
	15 m	f 13.422 (73 %)	f 11.602 (63 %)	f 9.984 (54 %)	f 13.062 (71 %)	f 11.530 (62 %)	f 10.144 (55 %)	f 11.366 (61 %)	f 10.035 (54 %)

Figure 6.3.1 Example of the influence of the angle of the outer slope on the transverse profile

6.3.3 Strength of crown and inner slope

If more wave overtopping is accepted the crown height can be kept lower but the strength of the inner slope must be increased and measures must be taken for drainage on the inner side. These aspects are considered in § 3.3. The width of the crown is related only to a limited extent to the safety aspect. This probably partly explains why difference practices have grown up in various regions. In the *Structural design manual*, from the point of view of the other functions of the dike (especially traffic carrying), recommendations are made for the width of the crown and the shape of the inner slope, with or without an inner berm.

A dike ditch takes up the water that flows from the dike and also the excess precipitation that falls onto it. In addition the ditch takes water that flows over, thorough and out of the dike profile, whether or not this has been collected in a longitudinal drainage in the inner toe. Often the ditch is also intended to take water from the adjacent properties

6.3.4 Revetment of outer slope

For the optimization of the dike the type and weight of the revetment material must be assessed. With tidal waters the main considerations are as follows:

- With a high foreland (above MHW + 0.5 m) a good grass turf may suffice. In other cases, at least in the zone of daily tide, it is necessary to protect the outer slope with stony material. Local experience will indicate the height to up which it is possible to use grass turf.
- Under MHW + 0.5 m the defence can only be applied in the form of a deposit such as a rubble covered fascine collar, loose stone (rubble) with a poured asphalt seal, packed material (gabions) or colloidal concrete.
- Between MHW + 0.5 m and NHW a revetment is of placed stone, packed materials, block mattresses, plates or colloidal concrete is used.
- Between MHW and NHW (+ additions), either a heavier version of the revetment used for levels under MHW or an asphalt revetment that can be lighter above the wave run-up zone, can be used, possibly with a grass cover at the higher levels.

The undersides of slopes constructed above water level must be supported and if there is a foreland the slope can be supported by this. However, for earth structures without a foreland (thus with an underwater slope), the slope must be supported by a berm that is placed close to the transition between the under and above water sections of the slope. This berm usually lies at about the level of 0.5 m above average low water (transitional berm) to ensure that the construction thickness of the revetment on the slope can be achieved at low tide. This berm may also be necessary to the outer stability of the dike. If there is no question of tides, the above is also valid, in which case lake level must replace average low water (MLW). Between lake level + 0.5 m and NHW a lighter revetment may be used that that required for sea dikes. The selection and sizing of the revetment is considered in Chapter 7 and in the *Technical report on revetments* (not yet published),

6.3.5 Gradient of inner slope and shape of inner berm

Within the available space it is possible to vary the magnitudes mentioned above. For example a gentle inner slope can provide a stable profile, but so can a steeper profile in combination with a berm. For a stability berm both high and narrow and low and wide structures provide good solutions. The nature of the stability problem usually leads to the most

effective solution. In the case of uplift, for example, a low wide berm is often more effective than a narrower higher berm. Spatial limitations on the inner side sometimes make an outward movement of the axis of the dam an attractive solution. With this the space for the inner berm is obtained from the partially excavated dike body. The dimensions of an inner stability berm also depend on the crown height. This means a complex interaction between the gradient of the outer slope – crown height – inner stability – can also bring about spatial optimization. For example the minimization of wave run-up by using a gentler outer slope gives a lower crown height. Although the crown requires more space, in this case the inner berm can be narrower and this compensates to some extent for the extra room that is needed.

6.3.6 Structure of the dike and provisions for drainage

Aspects such as macro stability and micro-stability (see Chapter 5) are highly dependent on pore pressure (see § 4.5 and § 4.6) in the dike body and thus indirectly on the structure of the dike. For example in a clay dike on sandy subsoil piping may be a problem. In the case of a sand dike on sandy subsoil piping will not be a problem, but the water level in the dike may become so high that there is a danger to stability. With good harmonization between the structures of the subsoil and the failure mechanisms that may occur, the structure of the dike can be optimized. It is necessary to consider effects of actions such as the application of an extra-thick clay revetment on the outer slope (to keep the phreatic line low), sealing off the dike body from the foundation or substratum, using an open or closed outer toe or constructing a drainage system on the inner side.

6.3.7 Optimization options with ice loading

Heavy ice loading, in particular that caused by piling ice, may damage the revetments and toe structures of a dike. The moving ice can carry away the revetment, particularly if there are sufficient points of contact in rough surface or protrusions (rows of poles), or if the slope is steep (steeper than 1:3). In fresh water ice loading is a greater threat than in salt water, since fresh water freezes more often and fresh water ice is harder than sea ice.

There is as yet insufficient knowledge available to be able to dimension a dike to withstand heavy ice loading although it is possible to take measures to restrict the consequences of ice loading. It is essential that there is a smooth surface without protrusions. A rough surface has the benefit that it reduces wave run-up, and thereby reduces the required crown height, but this positive aspect should be weighed against the possible increase in ice loading on the revetment. Transitional structures must not project from the slope. When the material of the revetments lacks cohesion, for example especially with loose stone in an underwater plasberm (transitional berm), it is necessary to take into account the fact that piling ice can carry loose structural elements up the slope.

The presence of a berm restricts ice loading on the upper slope and crown. On dikes without a berm above the slack water level, the piling ice can move across the dike. In such cases, it is necessary to assess whether this ice represents a threat to the crown revetment and the inner slope, and to any structures directly behind the dike. A berm above the high water level is favourable because it can accommodate the ice blocks. The ice floes will break up as they rise up against the lower slope on the front edge of the berm. The floes on the berm can no longer be pushed up the upper slope because at the front edge of the berm a form of buckling and cracking occurs. The floes buckle upwards and pile up.

The managers of the water defences know on which stretches of dikes where piling ice may cause problems. In the Dutch situation, this does not give rise to any situations that need to be included in the design of water defences. The consequences of ice formation on the rivers are manageable, while in general, the probability of a combination of drifting ice, a high water level and heavy wave attack is small. For dike stretches where piling ice does occur, this is taken into account in sizing in order to avoid unnecessary damage.

6.4 Soil investigation

6.4.1 Introduction

According to consultant engineers, the relationship between the cost of a soil survey and its rewards is unclear. Clients often wish to spend as little money as possible on soil investigation, but still want good advice. Consultants always want more samples and investigations at their disposal, and this requires greater expenditure. When optimizing the soil investigation, the issue is whether the benefits of a soil survey can be expressed in numbers.

6.4.2 Normal process

In order to estimate the strength of a water-retaining soil structure, it is first necessary to collect information on the structure and on the strength of the substratum. Depending on the progress of the project, the information required is more detailed or less detailed. The process is generally as follows:

Definition phase

In the definition phase, information is required on the basis of which it is possible to choose the alignment of the dike, as well as the principles that will govern the technical solution. At this stage, general knowledge of bedding in the substratum and a classification of the soil layers will generally suffice. The information required can usually be derived from existing documents such as soil maps, ground water maps, and so forth, and where necessary can be supplemented by relevant archive material from prior soil investigations in the area and/or exploratory soil investigations.

Preliminary design phase

In the preliminary design phase, information is required on the basis of which it is possible to make a choice with respect to the technical and economic consequences of a design. This means that there must be a reliable picture of the layer structure, the classification of the soil layers and possible local disturbances such as channels that may influence the design. Usually, the soil investigation will involve drilling and sampling, and where possible will be supplemented by additional geo-physical investigation to reveal any discontinuities in the pattern of the soil structure. In addition, reasonable indications of the physical and mechanical properties of the substratum must be available for geo-technical design calculations. To determine the soil properties, either laboratory or in situ tests can be performed. Another possibility at this stage is to use careful estimates that can be derived from Table 1 in NEN 6740, or estimates on based on correlation with the samples or test collections that are available for similar types of soil under similar conditions (see Constructing with soil: Earth structures on and in subsoil with little bearing capacity and with highly compressed subsoil).

Design phase

In the definitive design phase, the final uncertainties that may have remained after the preliminary design phase must be resolved, and the design must be made as cost-efficient as possible. This will demand a great deal of detailed soil investigation. In general, uncertainties with respect to the substrata will have been compensated in the preliminary design phase by over-sizing. This means that the nature and scope of the detailed investigation will depend on the expectation of possible gains to be achieved in the design. This may include stricter determination of the layer transitions at the most critical locations, determining the limits of local disturbances (channels, and so forth) and more detailed determination of the soil properties by means of laboratory and/or in situ tests. Detailed design calculations are also made in the detailed design stage, including calculations for several cross-sections in the dike section under consideration.

Table 6.4.1 gives rough indications of the nature and scope of a normal soil investigation. The details are from the CUR-C68 Report (Ref. Cases, *nature and extent of field and laboratory research*) and are based on the inventories and insight of a group of experienced consulting engineers. The consultants were asked to give their opinions on a responsible preliminary approach for one hypothetical case. Table 6.4.1 provides a summary of the recommended approaches. In practice, several engineering firms are asked for quotation for soil investigation. As a result of competition, the quotations given are as competitive as possible and for economic reasons the cheapest quotation is generally chosen, with the result that recommendations are made on the basis of fewer surveys. The resulting uncertainties can lead to costly oversizing of the design.

Table 6.4.1 Example of soil investigation efforts in various project phases. Source: Cases nature and scope soil investigation

Phase Definition	Data Archive	Source of information	Scope	Comments
		Maps Old measurements	Entire stretch of Dike	Interim layer Structure
	Geological background knowledge	Advisor	Entire stretch of dike	
	Experience			
	Local knowledge			
Preliminary y Design	Preliminary investigation	Geo-physical	3 survey lines	Layer structure, presumption of parameters on the basis of classification
		Continuous drilling	Per 200 to 1000 metres	
		Penetration tests	Per 50 to 150 metres	
		Hand drilling	Per 50 metres	
		Classification	8 per drill	
	Detailed investigation	Additional (special) drilling and sampling	Depending on the situation	Any layer structure adjusted per location. Parameters based on field and laboratory tests
		Triaxial tests of drilling samples	3 per drilling sample/at least 4 per layer	
		Compression tests of drilling samples	2 per drilling sample/ at least 3 per layer	

It should be emphasized that the table above is an example and is not for general use. The actual scope of the soil investigation also depends strongly on the type of work to be done.

Detailed investigation includes the placing of piezometric gauges, particularly in the implementation phase and end situation. Water pressure measurements are performed in soil layers that are important to the stability of the structure. Whether water pressure measurements are performed and if so how many depends on the geo-hydrological system.

In the test phase, first the available soil investigation data is considered. Only when simple tests fail is actual strength used or is possibly detailed investigation carried out (*Technical report on the actual strength of dikes*, not yet published). In dike reinforcements in Zuid-Holland, the test collection is used. Table 6.4.1 is not related to this. Further information on test collections can be found in § 4.5.4.

6.4.3 Optimal soil investigation

The definition of 'optimal soil investigation' depends on the project phase in question and the role played by the reliability of the substratum model.

Definition phase

In the definition phase the alignment of the structure and often also possible options that will provide initial solutions are selected. It is expected that these will meet the requirements and conditions (for example with respect to secondary functions, environmental demands, loading and safety demands) applicable for the design of the water-retaining structure. It is important that the information on the substratum is so reliable that it will not become apparent at a later stage in the process that other choices should have been made on the basis of further information on the properties of the bedding. It is therefore necessary to determine how much the choice of the process and principle solution depend on the assumed knowledge of the substratum, and what the probability is that the substratum may deviate in a negative sense from what was expected. The greater the dependence and the greater the probability of possible crucial deviations, the more necessary it is to refine the model of the substratum and increase its reliability by making additional investigations.

Preliminary design phase

In the preliminary design phase, the initial solutions are worked out more fully in order to reach a technically and economically viable draft design. Here too it is true that the reliability of the model of the substratum must be such that no surprises occur at a later stage. The test for this is analogous to the test in the design phase: test for sensitivity to possible deviations in the assumed substratum model and estimate the probability of such deviations on the basis of knowledge of the geology of the area.

From the mathematical point of view, in the definition and preliminary design phases, it is difficult to define optimal soil investigations. For this purpose a 'maximized user function' (or its equivalent, a 'minimized cost function') is required, in which the marginal costs of supplementary soil investigation are compared with the marginal rewards. Strictly speaking, the benefit of additional soil investigation in these project phases is a reduction in risk of 'incorrect decisions' (= probability of making the wrong decision multiplied by the result of the wrong decision). As yet, it is difficult to determine these risk components explicitly. At this stage of the project, the optimization principle provides a model that helps to direct ideas and not a concrete calculation that has to be solved. In practice, the only claim made is that in these project phases soil investigation is optimal only when the probability of an incorrect process or design choice is 'small enough', and when the nature and scope of the consequences of such an incorrect choice are also considered.

Design phase

The design phase, with the choices made in the previous phase, is concerned with the development of a design that is optimal from a cost perspective and that meets all the demands and conditions. The question of whether a more reliable substrate model is desirable depends in part on the degree to which eliminating uncertainty leads:

- A reduction in the cost of the design or the planned implementation
- A reduction in the implementation risks or further maintenance risks and/or
- Reductions in undesirable environmental influences (for example, requiring less of space and the associated reduction in landscape values).

The last example cannot always be expressed as a monetary value. In a cost/risk analysis, the anticipated return on the cost of additional investigation can be expressed as a monetary value and therefore allows a direct comparison of the costs of additional research. Subsection 6.3.4 gives two examples if this.

Management phase

In the management phase of a water defence, its safety is assessed on the basis of periodic safety testing. An important difference between the test and design phases is that for the first there is almost no freedom to take into account uncertainties with respect to the substratum by relatively cheap oversizing. This means that additional soil investigation, as a result of which the water defence can just pass the test, can give a relatively higher return than it could have done in the design phase.

6.4.4 Influence of additional soil investigation

The factors that must be taken into account when considering the use of further investigation are illustrated on the basis of two examples. The first relates to soil investigation to reduce uncertainty with respect to the structure of the substratum. The second example relates to soil investigation designed to obtain better estimates of soil properties.

Example 1: Layer structure

Assume the following situation. According to available reliable information (probes every 200 metres), the substratum under a dike over a length of 700 metres consists of a thick clay series that is not prone to uplift or heave. According to this information, piping does not play a role in the design of the dike. In probabilistic terms: according to the information, the probability of heave and piping is lower than the permissible probability. The permissible probability is related to the stipulated protection of the area. For areas with, for example, an NHW-exceedence frequency of 1/1250 per year, the permitted probability of piping in a dike section is equal to, for example, 1% of 1/1250 per year, and therefore approximately 10^{-5} per year. Assume that calculations have given a probability of uplift and piping of 10^{-6} per year.

However, this relates to a river area in which the possible presence of sand-filled channels in the clay series cannot be discounted. According to geologists, in this area there are channel indentations that are 50 to 80 metres wide: the estimated frequency is one channel per kilometre. At the location of such a channel, the probability of heave and therefore piping when the water level is close to the design water level is approximately 1/10 (assuming that the dike profile was designed on the basis of a continuous equally thick clay series). If there is a channel in the dike section, the probability of heave and piping (that is the chance of piping under design conditions multiplied by the probability of reaching the design water level) is about 10^{-4} per year. This is a factor 10 too large.

The substratum model, in which the thick clay series is present everywhere, is called O_1 . The alternative scenario, one in which there are one or more channels in the substrate model, is called O_2 . The probability of heave and piping, given models O_1 and O_2 is then $P[\text{piping} \setminus O_1] = 10^{-6}$ per year, or $P[\text{piping} \setminus O_2] = 10^{-4}$ per year respectively. The total probability of piping in the dike section is therefore as follows:

$$P[\text{Piping}] = P[\text{piping} \setminus O_1] P[O_1] + P[\text{piping} \setminus O_2] P[O_2]$$

assuming that O_1 and O_2 are the only possibilities. The actual probability of piping depends on probabilities $P[O_1]$ and $P[O_2]$. These two probabilities indicate the degree of certainty that the relevant substratum model is the correct one. If, for example, the probability of the presence of one or more sand channels is estimated at 10%, $P[O_2] = 0.10$ and $P[O_1] = 0.90$ (together making 1.0). The question is at which probability $P[O_2]$ the safety demands with respect to piping are just fulfilled. In that case:

$$P[\text{Piping}] = P[\text{piping} \setminus O_1] P[O_1] + P[\text{piping} \setminus O_2] P[O_2] \geq 10^{-5}$$

so, $P[O_2]$ must be smaller than approximately 10^{-1} . In other words, if the presence of a channel can be excluded with 90% certainty, the safety criterion is met. If not, the design will have to be adapted, for example by making a piping berm across the entire length of the dike profile. Assume that a piping berm is required, the costs of which are estimated at € 200 000 per kilometre with construction over 500 metres.

An alternative is to carry out additional soil investigation; for example, probes every 50 metres. This investigation will almost certainly determine whether there is no channel in the dike section, or whether there are one or more channels present. These are then also

immediately located. The cost of such an investigation is estimated to be € 10 000 fixed costs and € 25 per kilometre of dike. The investigation is expected to show one channel on the dike section. This means that a piping berm will have to be constructed over a 150-metre stretch. The costs of this are estimated at € 50 000.

The expected net result of the soil investigation for a dike section length of 700 metres) is therefore:

$$0.7 * 200\,000 - €\,10\,000 - 0.7 * €\,25\,000 - €\,50\,000 = €\,62\,500.$$

It is also possible (although not very probable) that two channels are found and that a berm will therefore have to be placed in two locations. The remaining net gain is then considerably less, at €12 500. If in reality three or more channels are found, the net result is negative. The obvious choice then is to construct a piping berm along the entire length, meaning that no savings are made. Therefore, with hindsight, the research is proved to have been unnecessary although previously the probability of this had been estimated as being very small.

With a presumed distribution of the probabilities of results from the soil investigation $P_1 = 0.05$; $P_2 = 0.90$; $P_3 = 0.04$ and $P_4 = 0.01$ (together 1.0), the expected saving (with respect to alternative 1) is:

$$0.05 * €\,112\,500 + 0.9 * €\,62\,500 + 0.04 * €\,12\,500 - 0.01 * €\,27\,500 = €\,62\,100$$

Now a random point L in the dike section is taken and the length, at which the breakeven point is reached, is determined, assuming that one channel will be found. The breakeven point is the point at which the costs of investment are equal to the return. The breakeven point found is at a length of 340 metres. This naturally depends strongly on the fixed costs in the calculation. For shorter dike sections in this example, it is therefore pointless to try and achieve savings on the piping berm to be built by making additional soil investigations

Table 6.4.2 Consideration of the alternative 'Additional soil investigation' versus the alternative 'Over-sizing'

		Costs additional soil investigation	Costs construction berm	Savings respect Alternative 1	with to
Alternative 1					
No soil investigation, berm construction over the full length of the dike section			€ 140 000		
Alternative 2					
Additional soil investigation, with local berm construction if necessary					
Result investigation (number channels present)	soil of	Probability result of soil investigation			
None		P1 = Small	€ 27 500		€ 112 500
One		P2 = large	€ 27 500	€ 50 000	€ 62 500
Two		P3 = Small	€ 27 500	€ 100 000	€ 12 500
More		P4 = Very small	€ 27 500	€ 140 000	€ 27 500

Example 2: More accurate estimation of soil parameters through local investigation

Imagine the following situation. A local dike reinforcement is designed according to the available substratum model and the estimates of the soil properties. Calculations relating to the stability of the inner slope indicate that a considerable support berm is required. In consequence a house lying behind the dike will have to be demolished and the cost of compensation, demolition and the construction of the berm amount to € 250 000.

The stability analyses were implemented with characteristic shear strength parameters that are derived from regional test collections. By definition these estimates of shear strength are on the conservative side. From a recalculation of stability, in which the shear strength of the resistance-offering moment of the most important soil layer has been raised by 10%, it appears that a narrower berm will suffice, avoiding the demolition of the structures on the dike. In this case, local reinforcement will cost € 50 000, a potential saving of € 200 000.

The following text weighs up the pros and cons of trying to make savings by carrying out local soil investigations. Local investigation of soil characteristics is defined as investigation in the direct environs of the location, for which soil-mechanical analysis has to be performed; this type of location is often indicated with the aid of the measurements taken every hectometre along the length of the dike. The assumption in local investigation is that the area (in the longitudinal direction of the dike) is so small that regional spatial distribution (see Annex 1 Characteristic estimates of shear strength) is not significant. According to current ideas, this concerns areas of some 100 to 150 metres along the length of the dike. If the field of investigation is much longer, regional distribution will play a role in the test results, and the average of a sample of results is not necessarily representative for a slip plane at the point of the location for which the soil-mechanical analysis has to be performed.

In addition to the costs of local investigation, the question of how great the probability is that, as a result of local investigation (sampling and laboratory tests), the characteristic shear strength increases by at least 10% with respect to the characteristic shear strength given in the test collection. The latter is a lower limit of 5% of what may be found at a random location within the region of the test collection. The probability that a higher average shear strength is found at a specific location is therefore 95%. However, in local investigations, account will also have to be taken of distribution in the test results: calculations must be made using the characteristic local average shear strength. For a small number of tests, there is the considerable probability that this will give results below or only barely above the characteristic shear strength given in the regional test collection.

On the basis of the regional model for the test collection, probabilities can be calculated for the possibility that an increase of more than 10% is found in local investigation for the characteristic shear strength. This probability depends on the number of tests in the local investigation, and on the relative distribution (= standard deviation divided by the overall average) of the test collection. These probabilities are given in the Table 6.4.3.

Table 6.4.3 Probability of >10% increase in the characteristic shear strength through local investigation

$V_{c\ rpv}$*	4 tests	8 tests	12 tests
0.20	0.15	0.45	0.55
0.30	0.25	0.55	0.65

* $V_{c\ rpv}$ = Relative distribution in regional test collection

It should be noted that for a relatively small sample size (4 tests) the probability of finding higher characteristic shear strength is relatively small, and that for greater numbers the probability increases. However there always remains a considerable probability that there will be no improvement with respect to the regional test collection.

It is assumed that the cost of the local soil investigation consists of fixed costs, € 9 000 and variable costs: for 4 tests € 8 000, for 8 tests € 16 000 and for 12 tests € 24 000. These

figures are used in the following analysis which was made with respect to the value of local investigation with various numbers of random samples (assuming a relative distribution in the test collection of 0.20)

- Sample size 4
Probability of € 200 000 saving is 15 %
Probability of € 17 000 loss is 85 %
Net result expectation: $0.15 * € 200\,000 - € 17\,000 = € 13\,000$
- Sample size 8
Probability of € 200 000 saving is 45 %
Probability of € 25 000 loss is 55 %
Net result expectation: $0.45 * € 200\,000 - € 25\,000 = € 65\,000$
- Sample size 12
Probability of € 20 000 saving = 55 %
Probability of € 33 000 loss = 45 %
Net result expectation: $0.55 * € 200\,000 - € 33\,000 = € 77\,000$

On the basis of this analysis, it would seem rational to choose a 12-sample test in the local investigation. However, the profit, or return per invested euro, is greatest for 8 tests.

Table 6.4.4 Expected return for local soil investigation

Number of tests	Probability saving € 200 000	of loss investigation costs	of Net anticipated return	Return per invested euro
4	15%	85%	€ 13 000	0.76
8	45%	55%	€ 65 000	2.60
12	55%	45%	€ 77 000	2.33

For this reason, 8 tests are chosen for this example. Despite the expected return, it is still necessary to take into account a relatively high probability that the investigation will not give the desired result. Of course that is always the case with investigations. With the chosen size of the random sample, the relationship between expected return and the costs invested in investigation are at a maximum. The conclusion from the two examples is that the choice of the optimal nature and the scope of additional research depend on the individual situation. It is always necessary to decide between the possible return and its probability, and the costs of performing the investigation and the probability that it will not provide any results. This will give rise to different decisions per situation.

6.5 Soil Improvement

6.5.1 Introduction

This section gives supplementary information about optimization based on the strength and soil mechanical aspects. A distinction is made between improving a weak substratum (§ 6.5.2) and a permeable substratum (§ 6.5.3).

6.5.2 Weak substratum

With a substratum that is composed of a thick compressible series a considerable over height is needed in the soil profile to compensate the anticipated settling. Extra space is often is also necessary with regard to the gentle inner slopes or wide berms needed to give the dike sufficient stability.

In an optimization process it is necessary to investigate whether it is interesting to reduce the required over-height and/or the total space required by the dike by entirely or partly removing the weak layers from the location of the future dike. In principle, the weak layers are excavated (dredged) from the location and replaced by sand. It is only possible to make a few general comments about the depth of this excavation. Very weak top layers consist of unripened silt and in any case are removed. These layers are also often so contaminated that from an environmental point of view their removal is compulsory. Such layers have little or no bearing capacity. Sometimes pumping away is an option, but certainly for wider landfill this is an uncontrolled process with a big chance of silt enclosures. The removal of peat layers that lie within the weak layers, for example in the first 5 m, is usually also worthwhile. Although these layers have a considerable bearing capacity (pushing out does not readily occur) these are precisely the layers that necessitate a considerable over-height and make extra space necessary. Soil improvement to a greater depth, for example the removal of all weak layers down to the depth of the sand, will less often appear to be rewarding. Preferably the width must be such that the dike body can distribute its load via the soil improvements to the remaining weak layers.

Special attention must be paid to the possibility of using a top seal with clay on a sand-filled trench; this is related to underflow and possible contact with a water bearing series. With a sand-filled trench in principle an entry point for the outer water is created at the outer toe of the structure. This may cause difficulties for both dikes with clay cores (danger of piping) and dikes with sand (a high phreatic line). Moreover it is often necessary to place a sealed revetment on the top of the cut-off trench. With a deep cut-off trench it is necessary to ensure that no permanent link is created between the cut-off trench/cunet filling and the deep water-bearing sand layer.

For the execution of the work it is important that no heave occurs during the excavation of the subsoil. If within the framework of a dike improvement, a cut off trench is made (and excavation takes place adjacent to the dike that is being improved) in addition to the stability of the subsoil the stability of the existing dike must also be guaranteed.

In addition to being used for the benefit of the optimization of the design, the use of soil improvement can also be considered in order to optimize the execution of the work. For example if the duration of the operation will be too long as a result of the existence of weak layers, it may be certainly be advantageous to employ measure to improve the soil.

6.5.3 Permeable subsoil

Piping berms play an important part in determining the amount of room taken by a dike body. Wide piping berms on the inner side of the dike may be considered undesirable because of the space they need. By constructing an impermeable berm on the outer side (possibly embedded in the foreland) the entry point can be lowered and the same effect can be achieved as with an inner berm.

6.6 The position of the dike improvement

6.6.1 Introduction

When improving existing dike a strict technically economic solution, as described above, is often impossible. In fact an effort will be made to keep the old well-settled dike body in tact and to include it in its entirety in the new structure. Figure 6.6.1 shows examples of the ways reinforcement can be accomplished:

- On the inner side of the dike (§ 6.6.2)
- On top of the existing dike (§ 6.6.3)
- On the outer side of the dike (§ 6.6.4).

In § 6.6.5 a dike reinforcement near to Sliedrecht is discussed.

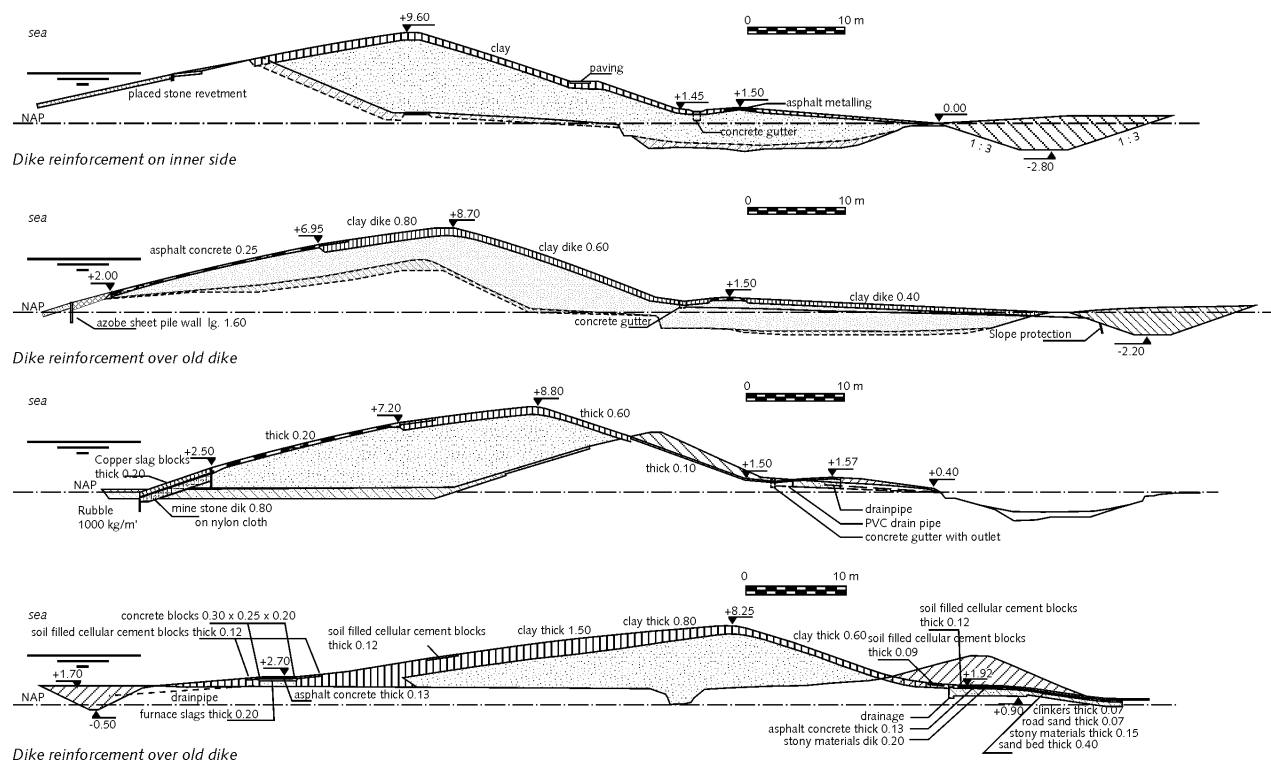


Figure 6.6.1 Various dike improvements

6.6.2 Reinforcement on the inner side of the dike

If the outer slope has a good slope structure, there will be both qualitative and financial reasons to maintain this and to choose to reinforce construct reinforcements on the inner slope. If the slope is not sufficiently stable when water levels are falling and it is still considered desirable to preserve it, the stability can be increase by constructing a support berm, possibly in combination with a reduction in the angle of slope above water. The execution of the work is also simpler, since the outer side of the dike remains largely in tact. At first sight, strengthening of the inner side seems preferable but the purchasing or compulsory purchasing of the land and buildings and the diversion of roads and/or watercourses may form a serious impediment. This impediment is even more serious if the LNC-aspects on the inner side of the dike in question have a high value. Another disadvantage is that when marine sand is used in the construction the salt water must be pumped back to the sea, but it is possible that some seepage of salt water into the polder will occur. Installing flushing pumps of sufficient capacity during the construction phase can counteract this drawback.

6.6.3. Symmetrical reinforcement

A symmetrical reinforcement (on both the inner and outer sides of the dike) is used only when conditions, (for example lack of space) make, this strictly necessary. The work is difficult and during the construction period the entire dike is disturbed. The water retaining function of the dike is more seriously affected than when the other types of reinforcement are used.

6.6.4 Reinforcement on the outer side of the dike

Reinforcement of the outer side of a dike is used when there is either no revetment or the revetment is not strong enough. A disadvantage is that the outer side of the dike is partially

dismantled during the operation. An advantage is that it is possible to work with free sand discharge, as a result of which there is less silt and the sand can be pumped directly into place. There is less chance of salt reaching the polder and it is not necessary to buy any agricultural land or built up land. With dikes that lie directly adjacent to open water high water levels may impede the work. When reinforcing the outer side a new dike body can be optimized from the point of view of the choice of crown height and the gradient and protection of the outer slope. Here LNC-aspects may play an important role.

From the above it can be seen that there is an interaction between the design and its execution. The design may be such that it leads to a particular method of execution that may be disadvantageous from the point of view of the continued reliability of the dike (strength and management) which can lead to modifications in the design. It is also possible that parts of the design may lead to expensive methods of execution or to a relatively big construction risk and these may be factors in the final form of the design. The two last mentioned factors generally weigh less heavily than the first.

6.6.5 Example of an optimization problem

The following example of dike reinforcement in Slidrecht illustrates an optimization problem. It was determined that the required height for the dike was not met in Slidrecht. It was also noted that in a large number of places the inward stability could not be guaranteed (insufficient macro-stability and uplift). In order to ensure that the target safety in the Ablasserwaard would be met, it would be necessary to reinforce the dike. The presence of the river and therefore of the dike is the reason for the existence of Slidrecht. The dike has a number of functions including carrying a traffic route, the location of residential and industrial premises, serving as a refuge for the local inhabitants in the event of high river levels and as flood defence for its hinterland. There is a remarkable diversity in the type, number of staff and site size of industries located along the dike at Slidrecht. The river is an important component of the primary waterway network for inland shipping between Rotterdam and the upstream hinterland. Outer dike reinforcements or changes in position of the dike are acceptable if the minimum river width is not affected. The optimization issue for the dike reinforcement in Slidrecht can be defined as follows:

- Maximise the interests of spatial planning, inhabitants and companies
- River width $\geq$ minimum
- Dike design = safe (according to the Delta Act)

It is unlikely that the problem has been defined so fully. It is a preliminary attempt at optimization. Boundary conditions such as management, maintenance and costs must still be included. At this stage, design scenarios are sought in which the feasibility of meeting the interests concerned is the key issue. These scenarios have associated price tags on the basis of which an initial solution is chosen, or in other words, a decision is made with respect to the LNC and other secondary functions for the further design process. The safety demand is fixed and as often also is the condition relating to the width of the river (= policy choice).

As long as the demands relating to safety are met, the applicable boundary conditions and basic assumptions may vary in order to reach the optimum design. In the optimization aspects relating to LNC values, allowing more overtopping, for example, may permit the incorporation of a lower crown for the dike. It may also be possible devise a better and possibly cheaper design when optimizing the dimensions of particular dike elements by reducing uncertainties in the layer structure. There are uncertainties in the structural design and in composition of the substratum (the substratum model) that can lead to oversizing in the design.

7 Revetments

7.1 Introduction

Over the centuries water defences, including revetments, have evolved in various ways, depending on the local hydrological and soil mechanical conditions and also the local requirements relating to other functions, such as those of river dikes which are often also used as transport routes. The evolution of the revetments will never cease and the developments in the decision making process relating to type of revetment and material are considered in § 7.2.

The philosophy around which the design of revetments evolved in consequence of the increasing insight into the interrelation between failure mechanisms. One element of this is the variation in the separation of the functions. Varying emphasis may be placed on the functional purposes of a revetment in relation to erosion and stability, the most extreme of these being:

- (a) the revetment may not fail during loading with a normal frequency. This is a strength stipulation relating to safety. Since the normal frequency is assumed, a certain contribution is attributed to the residual strength of the dike, this being the difference between the loading at normal frequency and that with 0.1 times the normal frequency (Ref. *Guidelines for sea and lake dikes*).
- (b) The dike itself may not fail as a result of the failure of the revetment. The probability of this must be at least a factor 10 smaller than the normal frequency. This means failure of the top layer revetment + dissipation of the residual strength of the top layer and underlying covering + dissipation of the residual strength of the dike body. The revetment of the dike is given a strength that is attuned as closely as possible to the lowest expected sum of the capitalized cost of construction and maintenance. This approach usually leads to a lighter revetment than approach (a) and is not a strength stipulation in relation to safety, but rather a maintenance stipulation.

The failure mechanism (b) can only be described to a limited degree and very much more research is needed to come to a generally acceptable mathematical model. Ultimately such models are needed in order to determine a realistic probability of flooding. Because this failure mechanism cannot yet be described, in the remainder of this chapter it will be assumed that the top layer of the revetment must be in state to withstand the design load. The function analysis relating to this is given in § 7.2.

In §7.3 attention is paid to the geotechnical aspects of revetments. The transitions between two types of revetment and the edges of revetments demand special attention. These special transitional constructions are covered in Chapter 8. The selection of the most suitable materials for specific cases is considered in *Guidelines sea and lake dikes Basic Report*. The information relating to this will be updated in the as yet unpublished technical reports on asphalt constructions and revetments.

7.2 Functional analysis

The revetments can contribute to the different functions of the dike in various ways:

- As a necessary protection for the underlying dike body and/or reduction for the wave run up with a view to maintaining the stipulated strength deriving from the legal safety norm.
- By limiting maintenance requirements: the damage occurring must be less than the damage that would be permitted from an economic point of view.
- The provision of a contribution to the water tightness of the dike
- An aesthetic/natural contribution; directed towards the other functions of the dike.

Often there is a combination of these contributions

There are many types of revetment to choose from (Ref. *Guidelines for sea and lake dikes: Basic report*). This will be explained in as yet unpublished reports on asphalt constructions and revetments. Depending on the type used, the composition of a revetment on a sand body consists of a minimum of one layer (top layer) to a maximum of four layers (a top layer, a filter on a geotextile and a clay layer). In normative conditions a water-retaining earth structure must possess sufficient height and stability and be sufficiently watertight, while the body of the dike may be either dynamic or static equilibrium.

Dynamic equilibrium

Dynamic equilibrium is possible in earth structures constructed from sand that are loaded by waves (dunes along the coast). The profile adapts itself to the hydraulic conditions (water level and waves); with severe storms erosion reduces the slope of the profile, while in calmer conditions the wind and waves carry the sand upwards and restore a steeper profile. Thus there is a continuous process of lateral transport of material, so the mass of the structure does not change unless there is a question of a gradient in the longitudinal transport (longshore drift); in this case the profile can extend or be reduced. The great width of the dune ensures that the water retaining function is satisfied. In some cases a threatened reduction of the transverse profile of a dune is reduced by a dune foot protection and by the planting of marram grass, while sand-trapping screens concentrate the growth of vegetation. Dynamic equilibrium in a sand dike is only possible if there are continuously changing water levels (tides). With coarser material (gravel) dynamic stability is also possible with stagnant water.

Sizing rules for dynamic equilibrium for an earth structure consisting entirely of sand are given in the *Guidelines for sandy coasts*. Gravel dikes do not exist, although gravel can be used as a top layer if this is considered desirable to satisfy other functions, for example as a beach close to the waterline for use by surfers (Ref. *Guidelines for sea and lake dikes: basic report*).

Static equilibrium

In most cases a dike body that is to be constructed from materials that cannot resist the attack of waves and currents and where no dynamic equilibrium is possible or can be permitted must be protected against erosion. This usually happens through the application of revetments directly onto the body of the dike. Only for non-primary dikes do situations arise in which attacks by waves and currents can be held away from the dike by a wide wet berm with a protected back (Ref. *Bank protection materials*).

7.3 Geotechnical aspect

When the choice falls on a number of layers the following geotechnical aspects play a part:

- The permeability of the top layer; this determines how the hydraulic loading is transferred onto the revetment itself and on the underlying soil body;
- The minimum required thickness of the revetment with an eye to the geotechnical stability
- The thickness and composition of the filter
- Whether a clay layer is needed and if so how thick it must be.

The permeability of the top layer

The interaction of soil, water and revetment under the influence of the hydraulic loading is shown in Figure 7.3.1 in the form of three characteristic components (Ref. *The interactions between soil, water and bed or slope protection*):

- (1) The translation of the external load (water level, wind and current) into load (pressure) components P_{ex} on the upper side of the revetment. The external loading is affected by

the depth of the foreland (limiting the wave height), gradient of the slope (type of breakers) and the roughness of the slope (wave run-up and overtopping).

- (2) The developments of the loading (pressure) components P_{int} in and under the revetment. This depends on the type of revetment (see Figure 7.3.2). Three basic types of revetment can be distinguished:

Very open

A top layer of loose dumped material or gabions placed on a filter (fascine) cloth or a granular filter underlain by clay for water tightness.

Very closed

A watertight top layer without a filter. This includes asphalt and concrete revetments and clay with a grass cover.

Slightly open

A permeable top layer consisting of placed elements on top of a filter on a watertight clay layer. The interaction of forces on the top layer is determined by the relation between the thickness and the permeability of the top layer and of the filter (Ref. *Handbook for the dimensioning of placed revetments and Dikes and revetments*).

- (3) The reaction of the elements of the top layer on the loading components. Every type of revetment reacts to the hydraulic loading in its own characteristic way. The resistance of the loosely dumped materials is determined by their own weight, the shape of the separate stones and the distribution in size. The gradient of the slope is also important. The resistance of an asphalt revetment derives from its resistance to erosion, its own weight, bending stiffness in flexure and possibly the suction of a weakly permeable under layer. A grass cover derives its resistance to erosion largely from the structure of its root system. Set stone revetments must rely on their own weight and the mutual packing pressure and friction of the elements.

The position of the phreatic line in the dike body (important to the micro-stability) is determined by the permeability of the total revetment, in which the infiltration into the crown and inner slope owing to wave overtopping and precipitation play a role.

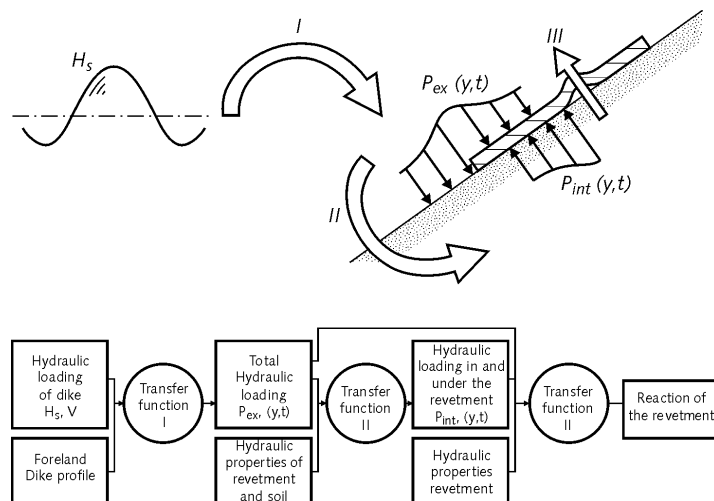


Figure 7.3.1 Components transferring the hydraulic load

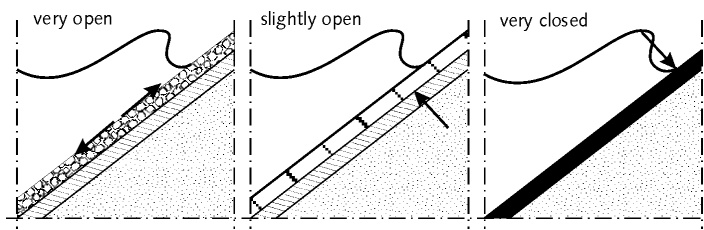


Figure 7.3.2 Basic types of revetment

The thickness and composition of the filter

With placed stones a granular filter is often used to take up size differences during the placing, but it can also take up material loss from the core when the substratum is porous. The latter makes demands on the grading of the filter but has little effect on the thickness required. If the filter has to play a role in reducing the loading on the top layer the thickness and permeability of the filter layer must be attuned to this. The total sand and water transport thus determines whether a geotextile or a granular filter is required or perhaps a combination of these. Next the result must be assessed on the minimum stipulated thickness for the filter that follows from the required geotechnical stability of the entire revetment. Filters for the various basic types of revetment are described in *Dikes and revetments*.

Clay layer

A clay layer fulfils various functions and each of these makes different demands on the quality of the clay (see *Technical report on demands of clay for dikes*). As an under layer a clay layer can contribute to the resistance to erosion of the residual strength of the revetment, (Guidelines on sea and lake dikes Basic report). With a permeable top layer the clay on the outer side must ensure that the earth structure is watertight. The aim is to make dikes as watertight as possible in order to restrict the water pressure and hydraulic head of the water in the dike and also the amount of water that is expelled via the inner slope. The composition of the clay on the outer slope must be such that few penetrating openings form (low sand content) and with a very small sensitivity to tear formation (low lutum content). For clay on the inner slope greater permeability is no disadvantage and usually even desirable. If the top layer consists of a grass cover the uppermost part of the clay layer must be sufficiently lime rich and not too impermeable. These factors also affect the thickness of the clay layer (Ref. Guidelines for sea and lake dikes, Basic report). With a watertight top layer the clay layer can be omitted.

The minimum required thickness of a revetment in relation to geotechnical stability

In the current *Guidelines on safety monitoring* a section is devoted to the soil mechanical stability of a revetment structure consisting of placed stone on a dike body. There are calculation rules that describe the resistance to shearing of the revetment construction under wave attack. For this mechanism it is assumed that the failure of the substratum along a shallow slide plane in the sandy core of the dike; the revetment structure itself does not fail. New insight in relation to the mechanism of resistance against shearing will possibly be included in the Technical Report on Placed Stone, which is to be published in 2002.

8 Objects that do not retain water

8.1 Introduction

In and in the vicinity of dikes, there are numerous objects that do not make a contribution to the water-retaining function, but that nevertheless can exert an influence on it. These include buildings, vegetation, cables, pipelines, roads, and so forth. These are known as non-water-retaining objects, or foreign objects. The influence of such objects on safety, management and maintenance has to be determined and, if necessary, compensating measures will have to be taken. A permit from the manager of the dike, which may include a number of conditions, is required for the presence of many objects.

A distinction is made between the following situations:

- a Existing objects, in combination with dike improvement
- b Entirely new objects, possibly in combination with a dike improvement
- c Existing objects in the context of the 5-yearly test for safety

Re a.

In the context of the work performed by the Boertien Committee, a number of ideas were developed for assessing the influence of existing objects in and in the vicinity of dikes. For some aspects, demands were set that may be somewhat less stringent than those applied to a design without objects. These are set down in the *Structural Design Manual* and are summarized in this chapter.

Re b.

When placing new objects, no concessions are made with respect to safety. This means that the dike, plus the object, must meet all the demands. There are separate norms for pipelines (ref. *NEN 3650, 3651 and 3652* and in the *Guidelines for water retaining structures and special structures*) which the design must meet. There are no such norms for other objects, and assessments of the influence of these objects have to be made on a case-by-case basis, as well as of how to compensate for these influences if necessary. If it is difficult to adapt the object and, like a residential area, it has a long life span, it is wise to consider whether supplementary regulations are worthwhile. The most obvious solution is to take into account a possible future dike improvement. In all cases, a permit will be required for new objects so there is always an opportunity to assess new objects and, where necessary, to formulate supplementary stipulations.

Re c.

In the context of the 5-yearly safety test, it is not always necessary to adhere to the demands of the design guidelines. The guidelines for the test are included in the *Guidelines on safety monitoring*. These are similar to the guidelines for existing objects given in the *Structural design Manual*.

In this Technical Report, attention focuses on the design of soil structures, in which account must be taken of existing non-water-retaining objects or new ones. This chapter therefore only covers aspects (a) and (b). References are made to the *Guidelines on safety monitoring* when the guidelines are applicable. Management plays a special role with regard to all non-water-retaining objects and this is covered separately in § 8.4.

In addition to the influence on the water defence, of objects that are not components of it, the dike also influences the objects in question, in particular during and after construction. Implementation issues are covered in Chapter 10. Section 8.5 briefly covers the issues that may be of importance in the design.

8.2 Existing objects

8.2.1 Introduction

When assessing existing objects the concepts 'assessment profile' and 'erosion profile' play a relatively important role. For this reason these concepts are explained first, but for more detailed information reference should be made to the *Structural Design Manual*.

8.2.2 Assessment profile

Annex 4 of the *Structural Design Manual* contains a more detailed description of the assessment profile concept a brief summary of which is given below. The assessment profile is used exclusively to assess existing objects located on the inside of the dike and relates to the assessment of erosion. The assessment profile is a theoretical minimum profile of particular dimension that must fit within the actual profile. The assessment profile is assumed to exist for a certain amount of time, during which there is the opportunity to implement any emergency measures in order to safeguard safety within a high water period. When applying the assessment profile, management is implicitly involved, in the sense that safeguarding and emergency precautions may be required.

The assessment profile is defined as follows:

- The outer slope that is physically present; this is the slope that is actually present, including any other structures
- A crown that is 2 or 3 metres wide and conforming with the height associated with an overtopping criterion of 0.1 l/s/m.
- An inner slope with a gradient of 1:2 for clay dikes and 1:4 for sand dikes.
- The inner berm that is required to ensure stability and counteract the effects of piping.

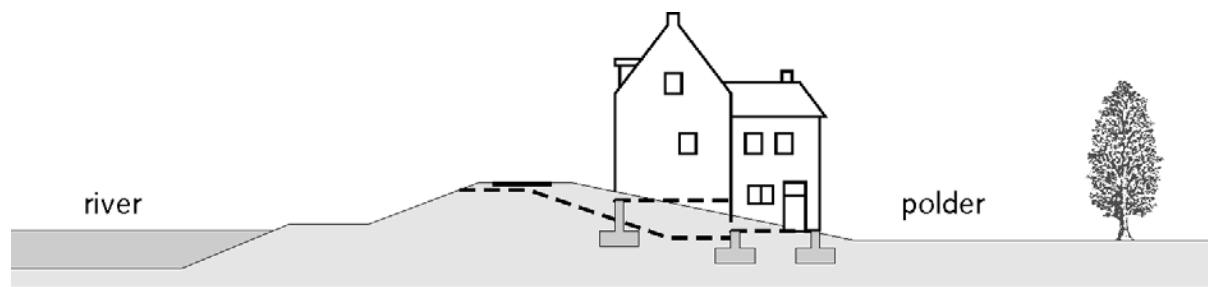


Figure 8.2.1 Example assessment profile

Water-retaining structures may be part of the assessment profile. In that case, the above-mentioned definition for the local situation will have to be adapted to the design profile of the structure. The following figure provides an example of an assessment profile with an erosion screen.

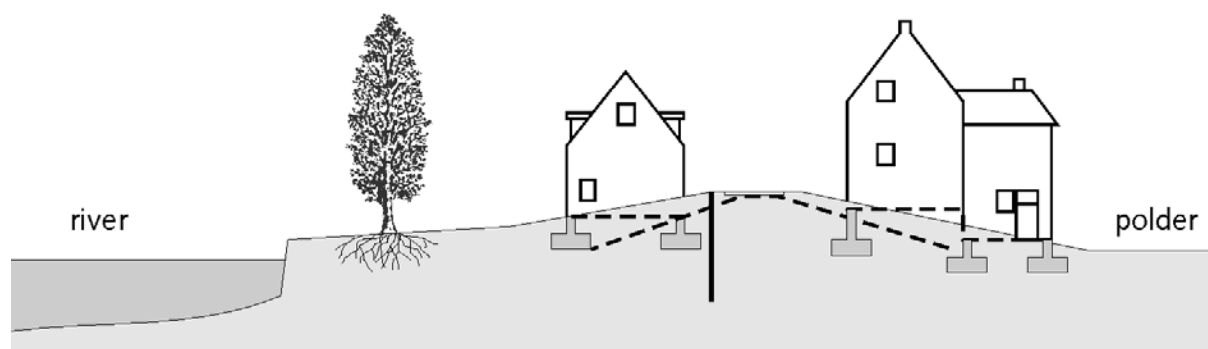


Figure 8.2.2 Example of an assessment profile with erosion screen

Use of the assessment profile is only permitted if the profile present, including the influence of objects located in the dike meets the stability and piping criteria.

8.2.3 Erosion profile

Annex 4 of the *Structural Design Manual* contains a more detailed description of the erosion profile concept, which is briefly summarized below. The assessment profile is used to assess objects located on the outside of the dike. The erosion profile is the profile that it is assumed will remain after considerable erosion on the outside of the dike. In principle, the erosion profile is defined as follows:

- A stable inner slope with gradient $> 1 : 3$ (this does not necessarily have to be the actual inner slope of a dike, but must lie within the assessment zone so that its existence is guaranteed)
- A crown that is 3 metre wide and has a height of $\text{NHW} + 0.3$ metres
- A theoretical outer slope with $1 : 20$ gradient up to a level of $\text{NHW} - 1$ metre
- A theoretical slope with gradient of $1 : 12.5$ that intersects the bank line at half the height of the base depth at NHW .

In principle, the erosion profile is applicable to all type of dike. With sea and lake dikes, outer dike buildings or vegetation are rare, so in practice the concept of the erosion profile is seldom applied to these types of dike does in practice. There are a number of conditions for the application of this erosion profile (see also Figure 8.2.3 for an outline).

- The existing foreland must be sufficiently wide and at least $\text{NHW} + 0.3$ metres high.
- The erosion profile must fall entirely within the existing profile.

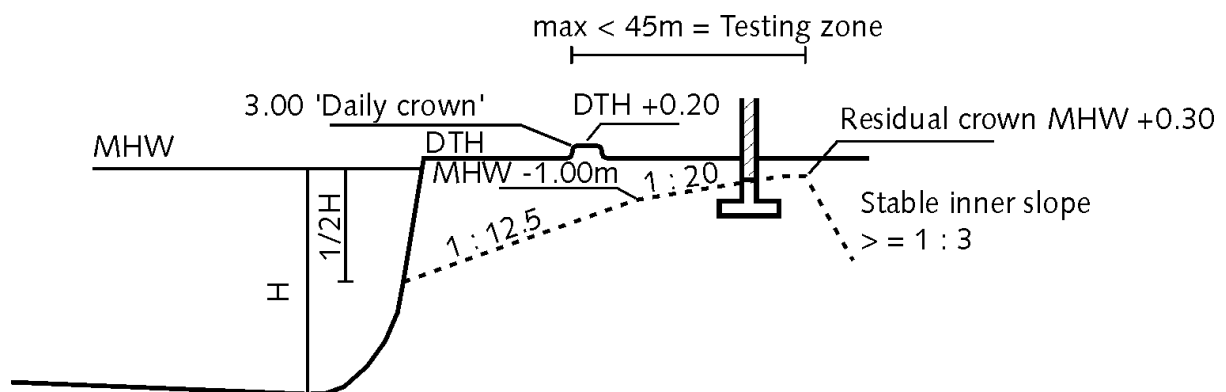


Figure 8.2.3 Principle of the erosion profile

The gradients for the erosion profile are derived from the equilibrium profile for dune erosion. This is believed to provide a reasonable and safe approach for sea and lake dikes with strong wave attack. For dikes in the river area, it can be expected that erosion by wave attack will be less, but another mechanism occurs here, erosion as a result of flowing water. In general, the erosion profile, as defined above, will provide a very safe basic assumption for river dikes. If there is any doubt, for example in an outer curve with relatively high current speeds and erosion-sensitive material such as sand, test calculations have to be made. If the erosion profile does not meet the standards set down in the *Structural Design Manual*, it is recommended that the speeds of the current under design conditions, the geometry and the soil structure should be taken into account when the erosion profile is being determined. If necessary, the erosion profile should also be adjusted. Specialists should be consulted in the event of doubt.

8.2.4 Categorization

A distinction is made between the following categories of objects, according to the *Guidelines on safety monitoring*:

- Vegetation
- Buildings
- Pipelines and cables
- Other structures (roads, land abutments, tunnels, ducts, quays and jetties).

These are outlines considered successively here but for further information; reference should be made to the literature mentioned in the bibliography and the TAW publications.

Vegetation

When a dike is being improved vegetation may be already growing on it. The decision as to what vegetation is valuable and what can be removed without objection is not covered here, although this may influence the dike design, and even the course of the dike. The dike reinforcement may influence the health of the existing vegetation while the presence of vegetation may influence the water-retaining capacity of the dike. The influence of trees and shrubs on the water-retaining capacity of a dike relates to the following aspects:

- Uprooting by wind
- Loading
- Soil properties and behaviour
- Phreatic line
- Erosion

The following major steps can be distinguished in the assessment:

1. It is necessary to ascertain whether the vegetation has negative influences on the stability with respect to:
 - Macro-stability: stability including vegetation (wind loading, own weight) and the stability after uprooting (uprooting pit)
 - Piping: see the *Technical Report on Sand-bearing Springs* for a summary of possible influences and interim design criteria.
 - Erosion: whether assessment is required depends on the location of the vegetation and the overtopping capacity. If the root zone is fully outside the assessment profile, or if there is an erosion profile on the outside that is not reached by the roots, the tree or trees can be kept without intervention.
 - Micro-stability: if the vegetation is on the inner slope, micro-stability may be influenced if uprooting should occur.
2. If it can be shown that the design profile, including the vegetation, meets the demands, the tree can be retained.
3. If it cannot be shown that the design profile meets the demands, measures may be taken to remove the vegetation to a position outside the assessment profile. Such measures may include the installation of a stability screen or an erosion screen in order to take over the water-retaining function of the vegetation covered inner or outer slope or the berm.

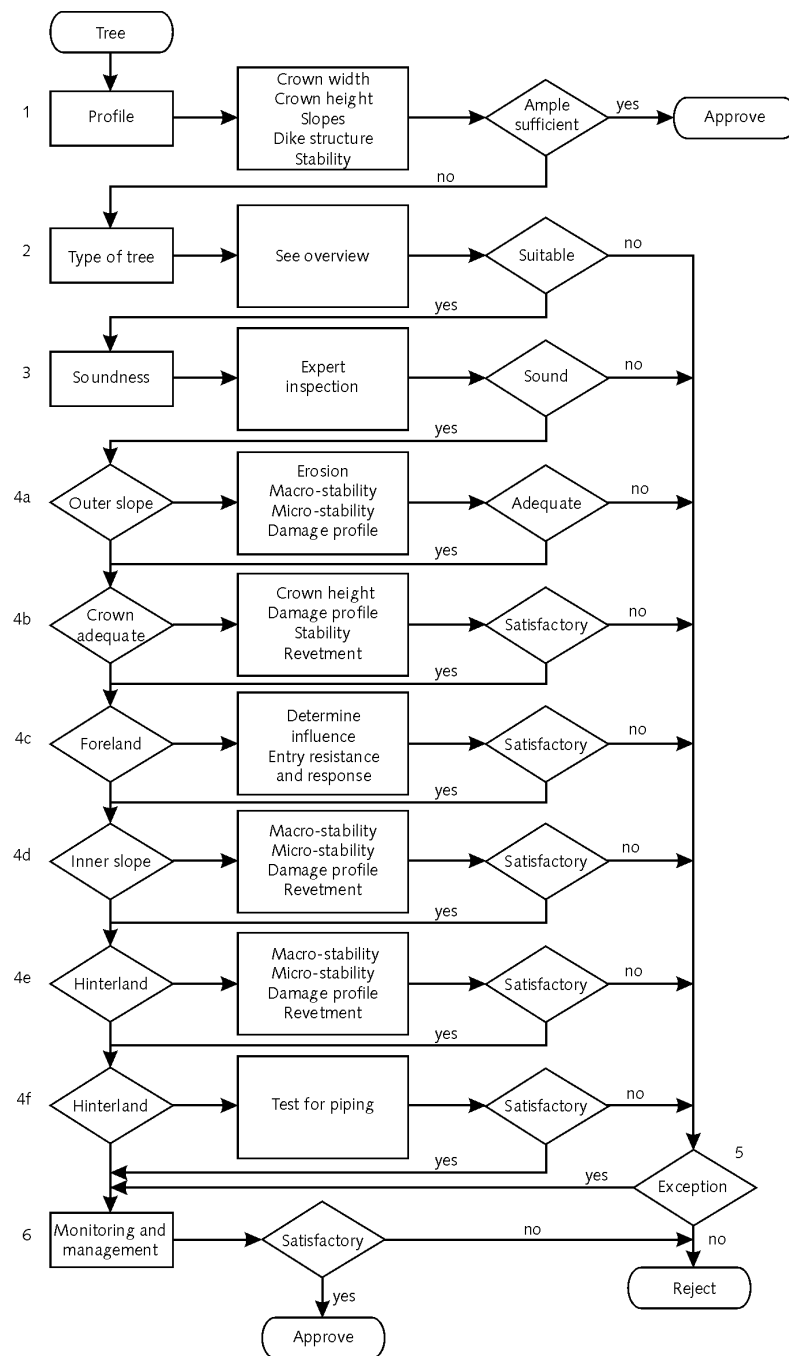


Figure 8.2.4 Flow diagram for the assessment of trees on water defences

Buildings

The influence of buildings on the water-retaining capacity is expressed in the following aspects:

- Loading
- Geometry
- Soil behaviour
- Water pressure development
- Erosion

Windmills are considered to be special types of building because they transmit of dynamic loads to the substratum.

The general assessment of buildings proceeds as follows:

1. It is necessary to ascertain whether the buildings influence stability negatively with respect to the following aspects:
 - Macro-stability: the stability of the dike including buildings (wind loading, eigen weight and vibrations)
 - Extra settling: turning windmills cause vibrations, while the wind load is transferred to the dike body via the foundations. Soil mechanical investigation is necessary to establish whether this vibration will cause extra settlement (for example by compression of deeper lying sand layers) and/or endanger the stability of the dike.
 - Piping: seepage occurs if the buildings, including basements and pile foundations, intersect covering layers. No piles with a reinforced foot may be used in buildings on dikes.
 - Erosion: depending on the location of the buildings and the overtopping capacity, assessment of erosion will be required. If the buildings are located entirely outside the assessment profile, or if there is an erosion profile on the outside, and the building is located outside this erosion profile, the building may remain without intervention.
 - Micro-stability: if the buildings are on the inner slope, they may intersect a covering clay layer and therefore give rise to leaks along the walls.
2. If it can be shown that the design profile, including buildings, meets the demands, the buildings may remain. In this case possibly parts of the buildings may be defined as 'special structures' (parts of the water defence).
3. If steps 1 and 2 do not provide a solution, the structure may be moved outside the assessment profile by introducing another structure.

Demands on special structures are included in the *Guidelines on Water-retaining Hydraulic Structures and special structures*).

With respect to buildings, it is rarely only the building itself that is of importance; other issues also play important roles. These include:

- Gardens and property delineation
- Cables and pipelines
- The situation with respect to maintenance, management, inspection and the associated ownership issues.

All these aspects will have to be assessed separately and with respect to their mutual influence. In (Ref. § 5.6.5 of the *Guidelines on sea and lake dikes, The Windmill and water retaining structure Act* and the concept *Guideline on wind and Water pressure development turbine foundations*).

□

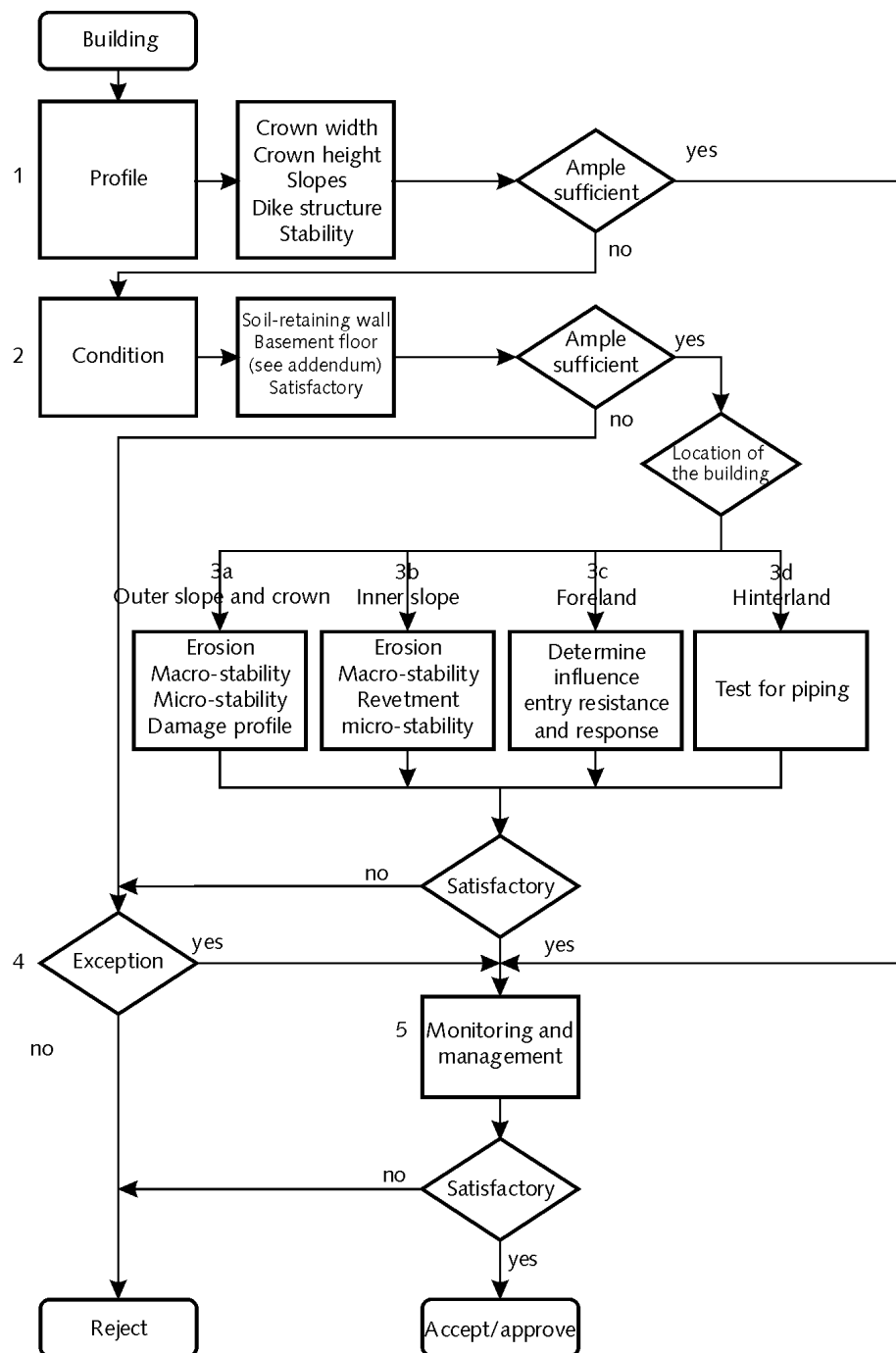


Figure 8.2.5 Assessment of buildings on water defences

Pipelines and cables

The influence of pipelines on the water-retaining capacity of a dike is seen in the following aspects:

- Springs, water pressure
- Soil behaviour as a result of digging activities

For pipelines the consequences of local erosion processes that will occur if a pipeline breaks must also be taken into consideration.

For low pressure pipelines (pressure < 10 bar) with a diameter of no more than 0.3 metres (Ref. *Structural Design Manual*), and for cables (Ref. *Guidelines on safety monitoring*) the assessment proceeds as follows:

1. If the pipelines and/or cables are located outside the assessment profile, no intervention is necessary.
2. If the pipelines and/or cables are in a poor condition, or if they are not in use, they must be removed.
3. If it can be shown that the dike profile, including the pipelines and/or cables, meets the demands, they can remain in place.
4. Cables that are below or partly below NHW must be equipped with a seepage screen.
5. /Mantle tubes must be fitted onto the cable in such a way that they are watertight and if connected below NHW they must be equipped with a seepage screen.

There is no clear guide for pipelines with higher pressures or larger diameters, except that of the assessment according to the *Guidelines on safety monitoring*. It is proposed that this procedure should be followed and that a score 'good' must to be achieved. In the context of a dike design, a score of 'satisfactory' is not acceptable. In principle the assessment proceeds as follows:

1. If the pipeline is in a poor condition or does not meet the general demands relating to a pipeline in a water defence, the pipeline should be removed or adapted.
2. If the pipeline was designed according to the revised Pipeline Code that was applicable at the time of construction of the pipeline and no changes have been made to the soil profile and the settlements used in the calculation, the pipeline may remain in place without any intervention. The demands stipulated in the original design may not be exceeded.
3. If the demands as set down in step 2 are not met, a recalculation according to NEN 3650, 3651 and 3652 will have to be performed, with a number of changes on the basis of the *Guidelines on Safety Monitoring*. If the pipeline is satisfactory on the basis of this calculation, it may remain in place. If the pipeline does not meet the requirements, changes will have to be introduced.

In all cases, for both small and large pipelines, safety tests should be performed on hydraulic engineering-related provisions such as seepage screens, sealing structures and even the possible replacement of the water defence, in accordance with NEN 3651 and the *Guidelines on Hydraulic Structures and Special Structures*. The cables should preferably be located in the same soil as the surrounding dike material. If this is not the case, changing the cable bed may be considered, or the dike profile should be assessed, taking into account the different type of soil at the location of the cable.

Other structures

The most important issue with respect to roads is the foundation. Since these are generally made of a different material than the dike, there is a discontinuity between them. There is a danger that water may seep underneath, especially if the foundation is composed of permeable material. In order to prevent this, the sealing clay layer on the outer slope must run continuously up to the height of the foundation of the road. An alternative is to construct the foundation of non-permeable material, for example of material that after sealing/cementing becomes impermeable, or by using stabilized sand. However this gives rise to a risk of tear formation, so it is not the preferred method.

Other points to take into account include:

- Vegetation in the verges of the roads is vulnerable to swerving traffic
- Signage and other objects
- Penetration of precipitation in open reinforcements and the verges of roads
- The design of access and exit roads

It is desirable that there should be a good connection between the revetment and the inner and outer slopes, where necessary with a special transitional structure. This is also applies to road paving on the access and exit roads. The design of the soil structures of the access and exit roads demands special attention. On the one hand, this relates to maintaining stability, while on the other it concerns preventing the formation of areas where grass cover cannot be effectively maintained, particularly on the shoulders or in angular hollows. Access and exit roads outside the dike are subject to greater current and/or wave loading and are therefore more sensitive to erosion. In addition, maintenance of the grass cover on bends and slopes adjacent to access and exit roads is less easy than on straight stretches, as a result of which, weak spots develop in the grass cover. Traffic loading should be included in the stability calculations as a static load. Vibrations caused by traffic can lead to water overpressure if the clay in the dike is saturated.

Tunnels

For tunnels, there are two ways to meet the requirements as water-retaining function:

1. Relatively small ring dikes]
2. Sealing measures

In the first case, the relatively small ring dikes must be designed entirely as soil structures. In the second case, the tunnel is a water-retaining hydraulic structure, and should be designed according to the principles laid down in the *Guidelines on Water-retaining Hydraulic Structures and Special Structures*.

Points to be taken into account with regard to ducts, quay structures and jetties include:

- The connection of the structures to the dike in the context of erosion
- The permeability of the dike or the outer slope if this is intersected by structural components
- Additional loading as a result of traffic, shipping, and loading stemming from the structures themselves

8.3 New objects

No concessions are made with respect to safety for new non-water-retaining objects that are to be introduced. This means that the probability of failure of the water defence as a result of the presence of the object must be very small. If as a result of other considerations there is a desire to introduce new objects, it is often necessary to take additional measures to compensate for the negative impact on safety. These in turn can have a negative impact on use of space, costs, landscaping issues and maintenance. In such cases, a decision will have to be made on the basis of the positive effects of the compensatory measures and the negative aspects from a LNC, financial, management and maintenance perspective.

This section covers those aspects that play a role when setting the conditions for new non-water-retaining objects, in addition to the information provided in § 8.2. For new objects a permit or test is always required in the context of the permit policy. This is covered further in § 8.4.

Vegetation

The basic premise for planting new trees and shrubs is that the safety of the dike continues to meet the demands. For this reason the trees or shrubs will usually have to be planted outside the design (or ledger) profile and, if possible, outside the zone of influence. One measure that can be taken to limit the negative influence of trees on the water defence is the application of anti-root sheeting, which is placed on the crown and/or the inner slope. The soil in which the trees or shrubs are to be grown is placed above this sheeting. The disadvantage of this method is that the vegetation cannot root so deeply and is therefore more prone to uprooting.

The influence of trees and shrubs on an inner berm depends strongly on the function of the berm and on the thickness of the clay layer. If covering layer of clay is relatively thick the trees or shrubs, will have little influence, even if uprooted. Depending on the situation, a soil settlement allowance, or settlement allowance in combination with an anti-root sheet may be sufficient to compensate for the negative influence of the vegetation. If the berm is relatively large in comparison to an uprooting hole, with an associated lack of danger of uplift following uprooting, additional measures may be omitted entirely. In considering this fully-grown trees should be taken as model and not their size at the moment they are planted.

Trees and shrubs are rarely permitted on an outer slope, since, on the one hand, a their shadows inhibit the development of grass cover, and on the other hand, a very erosion-sensitive point is caused in the event of uprooting. Trees and shrubs on the foreland can influence the water permeability of the clay layer. Little is known about this as yet. Careful design criteria are given in the *Technical Report Sand-bearing Springs*).

Buildings and windmills

In principle, features like new building and gardens, yards should be kept outside the design profile. For residential buildings, this is often a reason to introduce a settlement allowance, so that the residents can enjoy having a modest garden. Of course all other issues relating to safety must be assessed. Pile foundations, which are frequently used, must be assessed in relation to seepage and spring formation. Piles with a reinforced foot, or other types of piles that may not effectively seal with the soil are not permitted unless special measures are taken. Piles that displace soil generally give rise to less seepage but it is always necessary to ascertain no undesirable effects are possible. An exception to the need to avoid cutting through the design profile can be made for windmills and the solid parts of a foundation on steel. In such cases it is still necessary to consider the possibility of seepage and spring formation along the foundation.

Considering the relatively long life span of buildings, certainly where more than one building is concerned, it is best to use a higher design water level and to determine the design profile on the basis of applicable failure mechanisms. It must remain possible to introduce further dike reinforcements. The design for the foundation of the new buildings must also take into account the influence (extra bending moment as a result of horizontal deformations in pile foundations, and differential settling for piles on steel) of the future dike reinforcement.

Considering the above-mentioned, it is preferable that the erection of new buildings should not be permitted on the inner slope. It should also be noted that the crown height is determined in the context of the resistance to erosion of the inner slope. With an overtopping capacity greater than 0.1 l/s/m, either the height of the design crown will have to be changed, or the resistance to erosion of the inner slope will have to be guaranteed in a different way.

In the context of 'Space for the River', it is often necessary to consider whether the erection of new buildings in or close to a dike will impede the possible extension of the winter bed of a river or in the future.

Pipelines and cables

Pipelines within the zone of influence of a water defence should be designed according to norms NEN 3650, 3651 and 3652, also taking into account the additional stipulations according to the *Guidelines on Water-retaining Hydraulic Structures and Special Structures*.

For new cables the minimum requirement must be that they are located outside the assessment profile, preferably above NHW. Cables that intersect below NHW must be protected by a seepage screen. The cables may not be laid in a separate cable bed. In principle, cable mantles are undesirable and if their use cannot be avoided, special attention should be devoted to their design.

Larger cables, that is those other than domestic connections, should be laid outside the zone of influence of the dike. This is because the impact of laying and maintaining these cables is greater, and also because adapting these cables is a costly process.

Other structures

In principle, in this category no distinction is made between new and existing structures. If new structures are introduced, however, it will be easier to explicitly introduce the required safety aspects.

8.4 Management

□

Management plays an important role in non-water-retaining structures. Despite careful design, this type of object often brings an additional risk. Moreover, normal maintenance is not always possible in the vicinity of these objects, and other activities may concentrate here that do not occur elsewhere. In such cases careful inspection, supervision and, if necessary, additional maintenance is required to keep the water-retaining function in optimal condition.

□

Management involves the following aspects:

- Prior to construction or adaptation, the situation is assessed from the maintenance and management perspective: the object, its connection to the soil structure and layout and the finish of the building are designed to allow optimal maintenance and management. Additional space may be reserved for maintenance activities.
- When an object is built or adapted, if necessary a permit is provided including any conditions relating to issues such as:
 - Access for maintenance, inspection and emergency provisions;
 - Excavations;
 - Adaptations when changes are made in setting the norm for the water defence;
 - Conditions set with respect to the object or with respect to maintenance.
- Characteristic aspects are recorded in the ledger and in the management register;
- The situation is assessed in the five-yearly safety testing.

If optimal management is not possible, or it causes an disproportionate increase in the maintenance task, this may be reason enough to exclude the object from the water defence, because in such a case safety cannot be guaranteed in the long term. A continuous line of objects which must all individually meet the demands with respect to design and management can cause an unmanageable situation. There may also be interaction between objects with respect to soil properties, water pressure, and other factors. It is therefore recommended that not only are the individual objects assessed, but also the situation on a broader scale, from both the technical and management perspectives.

The primary concern in the management method employed, including the assessment of new, foreign objects, can be largely recorded in a management plan. Further detailing occurs in policy memorandums, such as permit policy, construction policy or planting policy. This provides a manageable convenient situation, for both for the manager of the water defence and for third parties wishing to construct non-water-retaining objects.

8.5 Design aspects

□

It is important to recognize the effects of the objects present in dike improvement plans at an early stage, so that they can be taken into account in the preliminary design. This section provides an outline of the possible implications, which makes no claim to be exhaustive; an individual assessment will be required for each situation.

In certain cases, it is useful to prevent the negative effects by taking special measures. This may require changing the design, or additional measures such installing a screen to prevent damage resulting from settlement.

Vegetation

Raising the ground level around a tree is only possible for a limited number of species. A field assessment by a specialist is required for each case. A change in ground water level as a result of soil supplementation, drainage, or the installation of seepage screens and so forth will affect trees. Certain trees can be moved to a more suitable location. For certain characteristic vegetation, it may be necessary to introduce screens to separate various functions in order to keep the vegetation.

Buildings and windmills

Soil deformation is important for buildings and other structures that are stiff in comparison to the soil. Soil deformation includes settlement flow and horizontal displacement of the soil as a result of an increase in height. This may give rise to deformation in the buildings or parts of them and therefore cause damage. Horizontal displacements of the soil can give rise to increased loading on foundations or basement walls. It is necessary to assess whether the buildings can resist this additional loading. The structure and condition of a building play an important role in the assessment of potential damage. When the ground level is raised, the drainage of surface water may change. Facilities for drainage around the building are therefore often required in the vicinity of the building.

Pipelines and cables

For pipelines, settlement and an increase in loading from above play important roles. Consultation of design calculations for the pipeline, supplemented by settlement measurements after the pipeline has been introduced will be required to assess whether the pipeline can resist the extra loading or deformation. For cables settlement is the primary issue. In a number of cases during the placement of the cables, settlement must be accommodated of by making a loop in the cable. This is particularly necessary where the cable is connected to or passes through a relatively stiff object. In other cases, it is possible to simply assume only limited difference in settlement for a certain length.

Roads

Roads place extra loads on a dike, as a result of their weight and the weight of the necessary reinforcement and of traffic loading. It is necessary to take into account that these loads may cause both vertical and horizontal deformation. Over time, as a result of extra settlement, the dike may no longer reach the stipulated crown height. Horizontal deformations are accompanied by risks to the macro-stability and the cohesion of the adjacent revetment of the dike. Cracks in the longitudinal direction of the road bring risks of extra infiltration caused by overtopping. An additional disadvantage of cracks in the road structure is that they often reappear after repairs have been carried out. Therefore when making the design it is necessary to pay extra attention to the horizontal stability of the road structure. When a road is being laid on a new dike of entirely or partly on an extension of the width of a dike it is recommended that the construction of the road should be done only after the consolidation of the earth structure is largely finished.

9 Special Structures

9.1 Introduction

Special water-retaining structures are usually structures that are either not built of soil or only partly built of soil. These structures are used to assume the water-retaining function, or part thereof, of an earth structure and to maintain or make possible the accommodation of other LNC values on or near the water defence. The primary reason why special structures can be valuable is the fact that they take up less space than traditional earth structures. However there may also be other reasons why a special structure is chosen, such as time saving during construction.

This chapter provides a summary of the possible applications and foremost points of attention and is a summary of what is included on this subject in the *Structural Design Manual*. Extended coverage of special structures would be out of place in this report, so references to various publications giving further information are included.

In principle the safety demands set with respect to special structures are the same as those set for water-retaining hydraulic structures. These are covered in the *Guidelines on Hydraulic Structures and special structures*. In many cases, a structure may influence several failure mechanisms. In the design, the consequences of applying a special structure should be taken into account for all relevant failure mechanisms. This chapter does not cover the design in detail and for these also reference should be made other publications.

9.2 Subdivision according to function in the water defence

On the basis of the statements of the Boertien Commission, the following subdivision is applicable for special structures:

- Replacement of existing dike material
- Choice of a different material when widening the dike
- Improvement of the existing dike material
- Improvement of the strength properties of the substrate
- Erosion protection
- Influencing of the ground water potential
- Structural elements
- Moveable water defences

An explanation is given per category, including a number of options

Replacement of existing dike material

The material in the existing dike can be replaced by a material with better strength characteristics or with different permeability in order to improve the stability of the dike or to reduce the influence on the surroundings of an increase in height. A general advantage of replacing existing material is that there is little or no increase in the space used. A disadvantage is that it is necessary to move relatively large volumes of soil and that temporary structures are often required to sufficiently guarantee the water-retaining function during construction.

Examples include:

- Changing the soil type
 - *Objective:* To replace the existing material by material with properties that are more suited to the current situation. This may include replacing sand by clay, thereby increasing the water-retaining capacity;
 - There are no special points of attention and the design of soil structures is covered in this Technical Report.
- Replacement of the core by light material
 - *Objective:* To increase the stability or reduce the construction time of dikes on substrata that are not able to bear heavy loads

- Materials that may be used include
 - Sand of wind blown origin
 - Expanded clay balls
 - Foam concrete
 - Polystyrene
 - Information on these materials and on the design can be found in CUR publication 162 (*Ref. Construction with soil and Design and construction of earth structures both on and into highly compressible subsoil of low bearing capacity*)
- Reinforcement with geo-textiles / geo-grids
 - Objective: To increase the stability of dikes on substrata that are unable to carry great loads, or to permit the use of steeper slopes
 - General information on the materials and on the design is included in CUR publication 186 *Geo-synthetic materials and river dike improvement*. For specific information on the materials and products, please refer to the manufacturers
- Sand/thread composition
 - Objective: To make it possible to build steeper slopes by improving the strength characteristics of the soil.
 - General information on the materials and on the design is included in CUR publication 186 *Geo-synthetic materials and river dike improvement*). For specific information on the materials and products, please refer to the manufacturers
- Sand-cement stabilization
 - Objective: To increase the stability and watertightness of a sand dike
 - Sand-cement stabilization is only possible for sand with a sufficiently high permeability. Local investigation will be required to assess the possibilities

Choose another material when widening the dike

When widening or increasing the height of a dike, it is possible to choose material other than sand. The materials to be applied are mainly the same as those mentioned under 'replace existing dike material'. There is also the option of:

- Using heavy material in a berm
- Objective : To prevent heave of a sealing layer and/or increase the stability of the slope
- No special design methods are applicable. Depending on the material chosen, any irregularities with respect to design will have to be recognized

Improve existing dike material

Under certain circumstances the properties of an existing dike consisting of sand or clay can be improved, as a result of which the additional improvement works can be of more limited scope.

Examples include:

- Sand-cement stabilization: This is covered under 'replacing existing dike material'.
- Electro-osmosis
 - Objective: To improve the strength and stiffness properties of clay to improve stability.
 - Electro-osmosis can only be applied in saturated clay without too many interruptions. A specialist in this field should be consulted on design and construction issues.
- Gel injections
 - Objective: To improve the waterproof properties of a sand body
 - Gel injections are only possible in sufficiently permeable soil. The gel is often not durable. A specialist in this field should be consulted for design and construction issues.
- Compacting the soil
 - Objective: To improve the strength characteristics and/or waterproof properties
 - Depending on the type of soil, compaction can be done by using vibrating rollers or by falling weights. The impact of the treatment is expected to be relatively superficial. These methods are applied generally in certain countries. There is no experience with the application of this type of dike improvement in the Netherlands
- Sand filled trench)

- Objective: To replace weak soil by soil with better strength characteristics so that the stability increases and the construction duration decreases.
- The application of a sand filled trench is often a good method in the construction of new dike stretches. The method of assessment is the same as that for other water-retaining soil structures.

Improving the strength properties of the substrate

Improving the strength and/or stiffness characteristics of the substratum may also provide advantages in construction and when certain techniques are used also in the user phase. If these techniques are used in inner dike sealing layers, attention must be paid to seepage and piping during the preparation of the design. The methods mentioned here are covered in greater detail in CUR publication 162, Construction with soil).

Examples include:

- *Vertical drainage*
 - Objective: To decrease the consolidation time and thereby the construction time
 - There is a great deal of experience of using this method.
- Vacuum consolidation
 - Objective: To decrease the consolidation time and increase stability.
 - This method cannot be used under all circumstances. Specialist input is advisable here.
- Electro-osmosis: This is covered under 'improve existing dike material'.
- Lime/cement piles
 - Objective: To improve the bearing capacity of clay by applying lime/cement in drill holes.
 - This method is not applicable for all types of clay. There is no experience with this method.
- Gravel piles
 - Objective: To improve the bearing capacity of clay by applying gravel in drill holes.
 - This method is not applicable for all types of clay. There is a certain amount of experience of using this method

Influence the ground water flow

Ground water flow plays an important role in piping and micro and macro-stability. By intervening directly in the ground water level or potential, it is possible to make great improvements with respect to these aspects. A distinction is made between the ground water levels in the dike itself and the potential in the water-bearing layers under and behind the dike.

The ground water level in the dike can be influenced by:

- Making the dike body less permeable by the application of a less permeable material, or by placing a screen in the dike.
- Introducing a drainage structure, as a result of which the ground water level falls in the vicinity of the inner toe.

The potential in the sandy substratum can be influenced by:

- Introducing a filter in the aquifer and reducing the potential in the filter by means of drainage
- Introducing a relief ditch
- Introducing a seepage screen

In filter structures and relief ditches, often the removal of water, with or without pumping, will be required to ensure satisfactory results from this method and to prevent a water burden.

A seepage screen in the aquifer can have various effects, depending on the dimensions and location:

- A seepage screen that seals a large part of the aquifer will reduce the potential, and thereby increase the stability and decrease the probability of heave in a clay layer. It will

also result in an increase in length of the seepage line, reducing the probability of piping. This type of seepage screen will also influence the geo-hydrology under average water level conditions, with the possible consequence that the conveyance of water or precipitation decreases, or seepage to nature areas decreases. Assessing the effect of this method is part of the design process.

- Seepage screens that seal only part of the aquifer have a limited effect on the potential, although the seepage line does increase. This reduces the probability of piping.
- A seepage screen placed on the inner side of the dike prevents the horizontal washing out of sand. There is then no piping but heave.

There will generally be an increase in potential on the river side of a seepage screen. Depending on the position of the screen in the dike profile, this may reduce stability. Examples of seepage screens in an aquifer include sheet piles, cement-bentonite walls, or walls formed by means of injection.

The last example is the seepage embankment. This is an inner dike embankment that serves to catch seepage water. Since the water level within the seepage embankment is higher than ground level, the probability of heave decreases. This may also increase the stability of the dike. The hydraulic gradient in the dike decreases, decreasing the rate of soil washing out from the inner slope. A disadvantage is that it is no longer possible to inspect the inner toe. A seepage embankment is seldom chosen as a dike improvement, but if a seepage embankment is present, its influence should be taken into account.

With respect to the dimensions of all measures in which the ground water flow is influenced, the calculation naturally plays a major role. This is described in greater detail in § 4.5 Geohydrological aspects and the *Technical Report on Sand-bearing Springs*

Protection against erosion

Traditionally, earth dikes are protected against erosion by a grass cover. Hard revetment structures are applied on stretches that experience relatively intensive erosion loads. Alternatives to a hard revetment structure include a number of revetment structures that may be useful while waiting for the grass cover to develop. In most cases, erosion-inhibiting properties will continue to exist for some time after the development of the grass cover. Examples include:

- Reinforced grass cover: Placing a mat close to the surface increases the cohesion of the soil. There are various artificial and natural cloths available.
- Honeycomb structures or cellular blocks. These prevent too much erosion before the grass cover develops.

CUR publication 186 'Geo-synthetic fabrics and River Dike Improvements' and CUR publication 168 'Nature-friendly banks' cover these and other types of similar applications.

Structural elements

Structural elements are those elements that owing to their strength can assume a part of the function of a water-retaining dike. Examples include:

- Sheet pile/retaining wall. Generally a concrete wall or steel sheet pile wall that retains soil and water and thereby assumes the function of a slope and/or stability bank
- Reinforced soil structures: A vertical wall that is anchored to the soil body by means of horizontal anchor elements and can assume the function of the slope
- Gabions: Steel nets or nets made of artificial materials, filled with stone, allowing the construction of a relatively steep and erosion-resistant slope. Their application as riverbank protection is very widespread. The nets are relatively vulnerable to damage.
- Cofferdam/deep wall: In principle these have the same function as a sheet pile/retaining wall.
- Ground nailing: Plates are applied to the slope and anchored in the shallow substratum. Tightening the anchors gives rise to tension in the soil. The disadvantage of this structure is that it is difficult to keep a vegetated slope on it and that the tension must be checked regularly.

The design of the sheet pile structure is covered in CUR publication 166 'Sheet pile structures'. There are no specific books available for the other structures.

Moveable water defences

Moveable defences are only placed under high water conditions. Examples include:

- Elements brought from elsewhere, including stop logs, and their modern variations
- Elements available on location that can be put in place by pushing or turning
- Floating structures; water defences that are positioned by the force of the water.

In addition to structural aspects, the closure procedure plays an important role in the assessment for safety of a moveable water defence. The *Guide on Hydraulic Structures and special structures* covers the design and the assessment of the closure procedure.

9.3 Selection

The need for a special structure, possibly as alternative to an earth structure, must be assessed on a case-by-case basis. This includes the issue of which special structure is most suitable. This section covers a number of aspects that are important in that decision, a distinction being made between the water-retaining function of the dike, and other functions.

Water-retaining function

Demands with respect to safety must always be met. This means that the demands must be met for all the relevant failure mechanisms and that an investigation must be performed to determine whether there are any secondary consequences. If the properties of the structure are not fully known in the circumstances in which it is to be applied, this will have to be taken into account.

The following issues play a role in safety:

- The degree to which the structure or the material is vulnerable to damage, for example, by vandalism
- Durability in relation to the desired life span of the structure and the possibility of its replacement at the end of its lifecycle
- The ease with which it is possible to test the structural integrity or the operation of the structure
- Construction issues
- Consequences for management and maintenance

It is also useful to check whether the structure can be adapted to changing demands, for example those relating to design water level.

Other selection criteria

Special structures are generally used to maintain or facilitate other functions of a dike or values on or near a dike. A first demand is therefore that this demand is met. Further selection occurs by determining whether it is possible to introduce an alternative in a specific situation. A not unimportant criterion is costs. For this, a total picture of the costs of construction and the costs of management and maintenance must be made for the life span of the special structure. Certain methods can have a negative impact on nature or landscape. This impact will also have to be taken into account in the decision-making process.

Table 9.3.1 (from the *Structural Design Manual*) gives a summary of materials and methods, and provides a relative assessment value for a number of selection criteria. Safety is not included in the table, as any demands with respect to safety must always be met.

Table 3. 9.1 Selection criteria
□

Material/Technique □	Durability□							
	□	Implementation□						
		□	Cost indication□					
			□	Reparability□				
				□	Experience□			
					□	Maintenance□		
						□	Extension options□	
							□	Design□
Adapt existing dike material / adapted material in expansion	□	□	□	□	□	□	□	□
Clay core	□□□	□□□	□□	□□□	□□□	□□	□□	□□
Light material	□	□	□	□	□	□	□	□
Sand of wind blown origin /dune sand	□□□	□□□	□□	□□□	□□□	□□	□□	□□
Expanded grains of clay	+	□□□	□□□	□□	□□	□□	□□	□□
Foam cement	□□	□□□	□□□	□□	□□	□□	□□	□□
Geo-textiles/grids	□□	□□□	□□□	□□□	□□□	□□	□□□	□□□
Sand-thread composition	□□	□□	□□□	□□	□□	□□	□□	□□
Sand-cement stabilization	□□	□□□	□□□	□□	□□□□	□□	□□	□□
Weighted berm	□□□	□□	□□□	□□□	□□	□□	□□	□□
Electro-osmosis	□□□	□□	□□□	□□□	□□	□□	□□□	□□
Gel injection	□□	□□	□□	□□	□□	□□	□□	□□
Improve substratum	□	□	□	□	□	□	□	□
Vertical drainage	□□	□□	□□	□□□	□□□	□□	□□	□□
Vacuum consolidation	□□	□□	□□□	□□	□□□	□□	□□	□□
Lime and cement/lime poles	□□	□□	□□□	□□	□□	□□	□□	□□
Gravel poles	□□□	□□	□□□	□□	□□	□□	□□	□□
Influences on Groundwater potential	□	□	□	□	□	□	□	□
Filter structure	□□	□□	□□	□□	□□□	□□	□□	□□
Relief ditches	□□	□□	□□	□□□	□□□	□□	□□	□□
Seepage screens	□□□	□□	□□□	□□	□□□	□□	□□	□□
Seepage embankments	□□□	□□	□□	□□□	□□□	□□	□□	□□
Screen in the dike	□□	□□□	□□□	□□□	□□	□□	□□	□□
Protection against erosion	□	□	□	□	□	□	□	□
Reinforced grass cover	□□	□□	□□	□□	□□□	□□	□□	□□
Honeycomb structure	□□	□□	□□	□□	□□	□□	□□	□□
Structural elements	□	□	□	□	□	□	□	□
Sheet pile/retaining wall	□□□	□□□	□□□	□□	□□□	□□	□□□	□□
Reinforced soil	□□	□□□	□□□	□□□□	□□	□□	□□□	□□
Gabions	□□	□□	□□□	□□	□□□	□□	□□	□□
Cofferdam/[deep wall/diepwand]	□□□	□□□	□□□	□□□	□□	□□	□□□	□□
Ground nailing	□□	□□□	□□□	□□	□□	□□	□□	□□
Moveable water defences	□	□	□	□	□	□	□	□
Transport from other locations	□□	□□□	□□□	□□	□□□	□□□	□□□	□□
Turns/slides	□□	□□□	□□□	□□	□□	□□□	□□□	□□
Floating	□□	□□□	□□□	□□	□□	□□□	□□□	□□
An explanation of the symbols is given on the next page□								

□

.

Explanation of the Table9.3.1

Durability□ □ □ □	□□□ Durable□ □□ Durability not guaranteed□ □□ Temporary in transitional period□ □□ Unsatisfactory□
Construction□ □ □ □ □	□□□ Simple□ □□ Possible without too much disturbance□ □□ No unreasonable disturbance□ □□ Construction increases disturbance□ □□□ Considerable disturbance□
Cost indication□ □ □	□□□ Far more expensive□ □□ More expensive□ □□ As expensive□
Reparability□ □ □ □	□□□ Simple to repair□ □□ Reparable with effort□ □□ Tough to repair□ □□□ Irreparable, needs full replacement□
Experience□ □ □ □ □	□□□ Good experience in dike construction□ □□ Good experience outside dike structure□ □□ No experience□ □□ Not applicable without care□ □□□ Bad experience□
Maintenance□ □ □ □ □	□□□ No maintenance□ □□ Little maintenance□ □□ No additional maintenance□ □□ Additional maintenance□ □□□ Much extra maintenance□
Option to extend□ □ □	□□□ Easy to extend□ □□ Extendable□ □□ Not extendable□
Design□ □ □	□□ Standard design methods□ □□ Specialist□ □□ Experience in implementation and/or later checking required□

□

10 Construction

10.1 General

This chapter covers construction issues relating to the construction of the earth structures covered, as a supplement to the information provided in the *Fundamentals of water retaining structures*. This chapter provides information that may be important in developing a design, and is not intended as a manual for the construction process. Points covered include:

- Environment
- Agreement between design and construction
- Applying and compacting the soil
- Stability during construction
- Safety of the dike during construction
- Settlement allowance in finishing
- Quality control

□

10.2 Environment

Implementing a dike improvement can give rise to negative consequences in the environment that in the worst case can lead to considerable damage to buildings. In the planning stage, an estimate must therefore be made with respect to the probability that damage will occur, caused for example by vibrations, settlement or horizontal soil deformation. This aspect is covered in greater detail in § 5.2 and § 8. 5. Vibrations may be caused by transport, compacting soil, the introduction of dam planks, and so forth. The structure and the condition of the buildings greatly influence their sensitivity to such influence. Measures may be included in the plan/specifications to reduce such influences. These may be physical measures, such as a sheet pile screen to prevent settlement under buildings, or organizational measures. In some cases, an agreement for compensation is made prior to the initiation of activities.

If the local infrastructure was not built to carry the transport required, additional measures should be taken and/or permission should be obtained from the manager of the road or waterway. Restrictions on road traffic or shipping as a result of transport or construction should be published well in time. Timely discussions on accessibility may also be necessary with residents, users of adjacent sites and auxiliary services.

Vibrations that may give rise to damage to buildings occur during the transport, application and compaction of soil. This demands care during implementation, to limit vibrations as far as possible. This may include avoiding placing material storage sites near buildings, the use of lighter equipment near buildings, the use of the correct machinery for installing sheet piles, and similar measure. It is also recommended that a record of the condition of existing buildings should be made before construction work starts so that damage claims can be assessed. Other measures include:

- Placing strain bolts on buildings
- Performing vibration measurements
- Monitoring
- Informing residents
- Making agreements with insurers as to insurability.

Measurements from the strain bolts and the vibration measurements will enable the cause of damage to be more easily determined. This is useful information when assessing damage claims. The first measurements of the strain bolts should be made shortly before construction starts. This provides a good reference. The contractor is responsible for a number of the issues mentioned. Of course agreements on responsibilities should have been made before activities start.

10.3 Agreement between design and construction

The staff to be involved in supervising the construction must be involved in making the detailed design and in the preparation of the specifications. During implementation, the project leader for the construction work maintains (preferably frequent) contact with the designers but certainly if the design requires changes relating to construction issues or if, during implementation, issues arise that were not recognized when the detailed design was being drafted.

10.4 Applying and compacting soil

In the construction phase of dike improvement projects, earth moving is an important parameter. Earth moving can be defined as: all activities in the scope of excavation, supplying, transporting and placing of soil. This section covers placing and compacting soil. These aspects play an important role in obtaining the required soil-mechanical properties. For a description of excavation and transport, reference should be made to the various handbooks in that field. All these issues must be stipulated in the conditions of contract. A number of issues are covered in the 'Standard RAW regulations.

Dry earth-moving methods

When this method is used, the material, whether clay or sand, should be transported as close as possible to the work site by lorries and dump trucks. Before work can start on applying soil, the existing ground level should be worked over to ensure that a good bond made with the soil that is to be introduced. Vegetation, such as grass, should be removed. Slopes that are under water and ditches should be cleaned. This involves the removal of vegetation and also any layers of silt. In addition the surface of the ground can be roughened by working it over with a rotary cultivator. When increasing the height of an existing slope, notches can be introduced to prevent the formation of preferential slip planes. The original topsoil may be required for future vegetation, in which case topsoil is stored separately, and replaced at a later stage.

If it has the desired moisture content, it is relatively easy to work with sand. The initial density of the soil is obtained by spreading the material in thin layers and using the transport vehicles to distribute the soil evenly gives the initial density of the soil. In some cases, it may be necessary to compact the soil further by using a vibration plate or roller to achieve the stipulated degree of compaction. Material may not be applied in layers that are thicker than 0.4 metres.

Clay cannot be sufficiently compacted when under water and must therefore always be applied in dry conditions. The Technical Report Clay for Dikes provides advice on processing and compacting clay, in both the core and in the revetment. A number of key issues are summarized below.

In order to obtain optimum compaction, the clay must be applied in layers no more than 0.40 metres thick. Each layer must be compacted individually, for example by using a vehicle with caterpillar treads. Compaction using a machine on air tyres, or using the bin on front of a digging machine rarely, if ever gives the required compaction level and therefore these methods are not recommended. To obtain effective compaction, it is essential that the clay meet the demands with respect to workability and compaction. These relate to the moisture content in particular. No work may be executed with frozen clay, as it is impossible to obtain the required density when clay is in this condition. After compaction, the dry density must be at least 97% of the Proctor density (one-point test) with the water content present.

Over compaction can give rise to a loss of strength in the clay while the penetration of the roots of vegetation, which is important in the top layer proceeds slowly and with difficulty. For this reason, the clay that is transported over the work area should be evenly dispersed by the vehicles. The clay must be handled in such a way that no puddles are formed on it by precipitation. The clay layers should be drained as they are introduced.

The application of material to a slope from the crown downward should be avoided, as it is difficult or impossible to achieve the required compaction. It is preferable to start at the toe of the dike, so that compaction occurs in horizontal layers.

Hydraulic processing

When constructing large sand bodies, or when applying sand under water, hydraulic handling of the sand is generally an attractive method. This method is sometimes avoided owing to hindrance and possible detrimental consequences for the environment. Usually the chosen method is to pump the material into a fill area above water, between the existing dike and a hydraulic fill dam. Spreading the material via a hydraulic pipeline above water usually gives high levels of compaction. Sometimes for deeper banks, an underwater depot is used. If no fill embankment is provided, the sand will flow off under relatively shallow slopes. This working method gives rise to loose layers of sand that are prone to weakening. Depositing sand under water is therefore not recommended. It is preferable to first deposit a base layer of (coarse) material up to above the low water level, and to compact the sand as effectively as possible in the above-water deposit. When hydraulic pumping is used in a fill area, from the point of view of soil mechanics this differs from a dry working method in a number of ways. Although the rate at which the construction proceeds is usually far higher, the presence of water during the fill phase adds complication. As there is little information available on this in the literature a brief description follows.

The stability of the soil structure, including any auxiliary structures such as a hydraulic fill dam, must be guaranteed at all times. This may have consequences for the use of space or materials, which must be recognized at an early stage. Various types of equipment and methods are used to improve the stability in the construction phase. These may include the application of sheeting to cover a hydraulic fill dam; thereby preventing the hydraulic fill dam from being entirely softened. It is also important that the excess water is transported away as soon as possible. The water level in the depot and any water overpressure in the underlying layers greatly influence stability.

The phasing of the work is generally performed on the basis of prior calculation. This is used to determine the construction method, the time required for the increase in height and the capacity of the equipment. If desired, the calculations can take into account measures such as additional drainage to shorten the time required. It is always necessary to monitor the work during construction to ensure that the stipulations that have been used in the calculations are adhered to.

10.5 Stability during construction

General

If there are any thick weak layers, water overpressure resulting from increases in height must be taken into account. This is important in determining the stability of the dike during construction and for assessing the feasibility of the design. If an increase in height cannot be achieved in a single operation without a loss of stability, additional measures will be required. Options for these include:

- Increasing the height in stages, with a period of consolidation between the stages;
- The introduction of vertical drains in order to improve the consolidation process;
- Soil improvement
- Improvement of the stability with geo-textiles/geo-grids;
- Increasing the height by using light material;
- In exceptional cases, changing the design, for example by introducing larger berms or less steep slopes.

The method of calculation is covered in Chapter 5.

The special structures, such as soil improvements, light fill materials and geo-fabrics are covered in Chapter 9. Information on the use of vertical drains can be found in

Constructing with Soil/Design and construction of earth structures both on and into highly compressible subsoil of low bearing capacity and Vertical Drainage). If vertical drainage has been introduced special expertise is required to measure and interpret the water pressure. An important fact resulting from the calculation of stability during implementation and the construction method is the duration of the construction. It is necessary to take into account that in winter only limited work is possible. In the final year of construction, time should be reserved for finishing.

In reinforcement on the outer side of sea dikes, particularly if sand is used, almost always one of the demands is that work is completed in one season. A defended dike body must be completed the storm season. In some cases it may be necessary to apply pre-loading before the actual work commences.

Construction and supervision

If the water overpressure is critical for stability during construction it must be measured during the construction. The calculations may contain various inaccuracies, for example as a result of uncertainty about the soil structure or soil properties, or as a result of water overpressure. Supervision during construction generally focuses on verifying that the actual water overpressure remains below the permitted water overpressure level. Effective supervision will allow the rate of the increase in height to be adjusted as required. This allows the height of the stages to be adapted and/or the waiting period between stages to be adapted. The number of locations at which the substratum may react otherwise than expected is far larger than the number of locations in which it can be reasonably expected that water pressure meters can be placed. Careful consideration of the selection of the points to be monitored is therefore essential. In addition to the above-mentioned aspects, the work activities should be taken into consideration when determining the location. On the one hand, hindrance to the activities by the presence of the meters must be prevented, while on the other hand, the meters or their cables may not be damaged.

Water pressure may remain higher than expected as a result of creep. This is also the case if plastic deformations have occurred. The maximum height of each stage may not be exceeded, even temporarily as a result of a temporary soil depot.

Stability monitoring in hydraulic filling is more complicated than when height is increased under dry conditions. With increases in height under dry conditions, the layers are less thick, and intervention may be required at any stage. When thick layers of hydraulic fill are used, the increase in loading near the water pressure meters may be so quick that timely intervention is not possible without very intensive supervision. Using drainage by well-pointing can make the process more manageable.

10.6 Safety of the dike during construction

In principle, the safety of the dike (or its water-retaining capacity) may not decrease during construction. This means that no activities may be performed in the winter period from 15 October to 15 March for river dikes and from 15 October to 15 April for sea and lake dikes. Activities that in no sense affect the safety may be permitted in consultation with the manager. Even during the summer months safety must not be endangered. Separate boundary conditions for have been drafted for sea and lake dikes with respect to water levels and wave heights (see on *Guidelines on sea and lake dikes*). It may be possible to deviate from these if additional measures are taken to guarantee safety. In such cases, for example, the contractor must be able to restore the former strength of the dike within a specified number of days. If during or after the winter activities the outer revetment of the dike is not yet present or ready temporary protection must be applied.

With respect to the safety relating to stability during a construction phase, a distinction must be made between shearing that affects the safety of the dike and shearing that results exclusively in damage to the work. In the first the protection of the vicinity is at issue and the safety criteria must be as closely as possible in line with the area protection norm for the hinterland. The second type of shearing related to an economic consideration. The costs of construction are weighed against the costs of repair in the event of shearing.

Investigation indicates that this economic optimum is reached in the event of relatively large instability probabilities, in the order of 10^{-2} to $3 \cdot 10^{-2}$. These probabilities are at levels higher than the permissible instability probabilities, which in turn are in line with the area frequency (10^{-5} to 10^{-6} per year).

When assessing the stability during a planned construction situation, it is necessary to determine to what extent the critical slip planes or slip circles affect the water-retaining capacity of the old dike. The rule is that slip planes that affect the crown, possibly after secondary shearing, must meet the strict stability criterion (see following stability factor criterion), taking into account the normative water level for the construction period. It is only possible to deviate from this if a risk analysis shows that safety levels remain sufficient. For slip planes that only damage the work in progress, a lower criterion may apply.

Hydraulic fill or dry introduction by vehicle

For the stability assessment during the construction phase, it is essential that it is possible to safely measure the acting water over pressure in the (old and new) dike body and substrate. The degree of insecurity with respect to water pressure development depends strongly on the way the new dike material is applied: hydraulic fill or dry application by vehicle.

If the material is carried in by vehicle, the generation of water overpressure in the substrate is relatively easy to predict. It is more difficult to determine the degree of dissipation of water (over) pressure as a function of time after applying a layer to raise the height of the structure. For this reason, considerable safety margins are used in the design with respect to the consolidation coefficient expected during construction. Depending on whether the water pressures are monitored, a higher or lower partial material factor is used for the consolidation coefficient. Lower material factors may be used only if, on the basis of observed water pressure during construction, appropriate measures are taken if unexpected unfavourable conditions occur with respect to stability.

In hydraulic filling, the development of pore pressure in the substratum and the dike body are considerably less favourable and uncertain, since:

- The rate of construction is far higher than with dry application by vehicles
- The deposited material is saturated, so that water penetrates the old dike body and hydraulic fill dam from the deposited material
- The working method is less flexible than it is when vehicles are used, thereby reducing the chance of effectively intervening in the event of unexpected pore pressure developments.

□

Boundary conditions with respect to outer water level

If the outer water level plays a role in the stability analysis, safe estimates must be made. If high water levels play a role in stability, extreme water levels have to be taken into account. For activities in the summer period, this may be based on the high water statistics for the summer period, with probability of exceedence equal to the applicable area frequency. Some information on this is included in the *Guidelines on Sea and Lake Dikes*. More information and information on water levels can be obtained from the Dutch Directorate General for Public Works and Water Management RIKZ or RIZA.

Safety

The situation during the execution of dike reinforcement activities is often more unsafe with respect to soil-mechanical instability than when the work has been completed. This situation results from water overpressures developed during the fill phases, which dissipate only very slowly in the substratum. The probability of instability is greatly influenced by the rate at which the increases in height are applied, and is easier to estimate as the security relating to the development of the process of dissipation of water overpressure increases.

From the perspective of economic optimization, setting the permissible probability of insecurity during construction is determined by both the construction costs and anticipated expected damage. Low work rates result in high costs, while the cost of repairing damage increases as the probability of instability increases.

In addition, during construction boundary conditions relating to the risk of inundation must be taken into account. If the outer water level plays a role in the stability analysis, a safe

assumption will have to be made for this (see § 11.6 and *Guidelines on Sea and Lake Dikes*, § 6.5). Provisionally, a water level will have to be assumed that can be retained by the dike enclosure ring at the time of improvement. For practical reasons, a water level that is 0.50 metres lower than the lowest existing crown height in the dike enclosure ring is recommended at the start of the improvement, taking into account the associated river gradient. For sea and lake dikes, water levels approximately equal to the crown height can be used, minus the freeboard required according to the design.

The issue of permissible probability of instability and the associated preconditions is still under investigation.

Water over pressure as a result of increases in height

As a result of former increases in height, there may still be initial water overpressure, which can be indicated by water pressure meters. This reduces the strength of the dike. Increasing the height for dike construction or dike reinforcement reduces the shear strength in the clay/peat layers in, under and next to the dike. The increase in loading is primarily expressed as water overpressure, as a result of which the effective stress, and therefore the shear strength decreases. As a result of the consolidation process, the water overpressure decreases in time, resulting in an increase in effective pressure, or shear strength. In the Lower River area, the initial shear strength is often insufficient to bear the entire addition. In this case, phased construction is desirable, so that use can be made of the gradual increase in the effective pressures in the substratum, and thereby that of the shear strength.

When determining the construction rate, which depends on the various construction stages, the level of water overpressure and the consolidation speed must be calculated. If, on the basis of the geometry and soil characteristics, it can be expected that the shear deformations will remain limited, the increase in pore pressure can be predicted on the basis of the elasticity theory. The development of consolidation can be globally calculated using Terzaghi's one-dimensional consolidation theory. If, with low stability factor, the expected shear deformations are large, the water overpressure is higher than can be predicted on the basis of the elasticity theory. This is the result of elasticity and contraction. The magnitude of the water overpressure can in this case be predicted using an element method based on an elasto-plastic soil model, such as the Soft Soil Creep model.

Two-dimensional consolidation

If the consolidation behaviour of the soil is largely determined by horizontal deformation and/or horizontal run-off of the over-pressurized ground water, two-dimensional representation is required. This is the case in anisotropic peat layers (see § 4.3.6) in which the horizontal permeability resulting from the deformation process, may be a multiple of the vertical value. A two-dimensional consolidation calculation may also be desirable if the width of the height increase is relatively small in comparison to the thickness of the compressible layers. In this instance, pressure dispersion occurs in the substratum, as do two-dimensional flows in the pore water. By using a finite elements method, it is possible to calculate the two-dimensional consolidation behaviour. This makes use of the basic comparisons developed by Biot.

As a result of creep phenomena, the predicted consolidation rate may be disappointing, while the water overpressure remains high. These effects can be calculated by using FEM on the basis of the Soft Soil Creep Model. If the water overpressure calculated is critical to stability during implementation of the work, it must be measured during implementation. It is recommended that electrical or Bourdon water pressure meters be used for this. The actual water pressures may deviate strongly from those predicted; uncertainty about the layer structure, the calculation model and the degree of accuracy to which water permeability can be determined are the main causes of this. It may be desirable to adjust the calculation results during implementation to prevent construction taking too long, or to avoid the need to take undesirable risks.

Squeezing

Under the influence of loading on the ground level, weak layers directly under or just under the ground level may be pushed off. This phenomenon is known as squeezing. This can be important during the implementation of dike reinforcement activities. Squeezing is the result of the development of local water overpressure in a relatively short period of time in the weak layer, as a result of which the shear pressure exceeds the available shear strength. Methods of calculating calculate squeezing are given in §. 9.6 of CUR 162 *Constructing with soil*.

10.7 Settlement allowance in finishing

The height of the crown of the dike, and of any banks, must be at least equal to the design height multiplied by the settlement that may occur within the planning period. In general, settlement calculations will be performed in order to estimate settlement. The results of these calculations can be adapted on the basis of regular measurements of settlement gauges. Extrapolation of the measurements gives a prognosis for the settlement but, if applicable, account must be taken of the fact that the measurements were made while the increase in height was not yet fully completed. Although of primary importance, the correct height of the crown is not the only boundary condition for the final profile. After all, it is clear that the profile to be completed must be satisfactory in both the longitudinal and transverse directions. A complicating factor is that the objective of placing sand in the core, with clay as covering layer is to introduce the settlement allowance in the cheaper sand, and not in the clay. This means that a good estimate of the settlement must be available when profiling the sand. If the estimate is not good, either some of the sand will have to be removed or, if there is a deficit in the settlement allowance, more clay must be added. Here, a good balance between the supervisors and soil-mechanics advisor is essential.

If hydraulic fill methods are used, it is also true that with regard to the settlement prognosis it is not always possible to make accurate predictions. As a result of the high filling rate, the settlement process will not be usually fully underway when the prognosis is made. In addition, the configuration of the transverse profile directly after pumping in the fill is not the same as the final transverse profile. For the prognosis, it is important that there is a clear picture of the water level in the depot. For this reason the walls of the settlement gauges perforated on the bottom so that they can function as open reference tubes.

Predictions based on measurements are only possible if the measurement series is sufficiently large. This means that a prediction cannot be made shortly after the last increase in height has been carried out. Usually it is advisable that the finishing of the dike should not be completed immediately after the application of the last increase in height. This is because the residual settlement is still high at this stage, and occurs quickly, so it would possibly result in damage to a hard finishing layer such as placed stone or asphalt. Therefore a season sometimes passes before the final layer is applied and in such cases a temporary finish will have to be applied.

If vertical drainage is used, a complication may occur in the settlement prediction as a result of the difference in consolidation behaviour. It is certainly advisable to obtain a professional interpretation of settlement measurements and pore pressure measurements in this case. In practice, the calculation method used for construction materials will also influence the increase in height that is achieved. This is related to the economic motives of the contractor.

10.8 Quality control

Soil properties

The soil used must be assessed with respect to composition and level of compaction. This will reveal whether the soil does in fact possess the properties required according for the design. The degree of attention that must be devoted to the composition and compaction of the soil and the influences of these properties on the various failure mechanisms depends on the

soil properties required by the design. For the demands placed on clay, reference may be made to the *Technical Report on demands on clay for dikes*.

Soil testing is described in the *Standard RAW stipulations*. The final assessment takes place on the basis of samples taken once the soil has been worked. The composition of limited number of samples of the material is determined in a laboratory, in accordance with the *Standard RAW stipulations*. Compaction is also assessed on the basis of the Standard RAW stipulations. In addition to the assessments stipulated by RAW it is advisable to use experienced members of staff to assess the soil composition and compaction of the soil throughout the entire duration of the work. The degree of compaction can be estimated from time to time with the aid of a penetrometer or manually operated equipment. If there is any doubt about the compaction of clay supplementary test trenches can be excavated. These will reveal whether there are still any spaces in the clay.

Documentation

When the work has been completed it is advisable to collect together all the relevant information. This should include:

- Assessment reports on the material used including soil;
- Information about changes in the design, including any supplementary design calculations;
- The results of settlement measurements and the revised settlement prognosis;
- The results of any pore pressure measurements, compared to the predicted rates of consolidation;
- Revised measurements or the final measurements;
- Unusual occurrences during the construction period;
- Unusual occurrences relating to management and maintenance, for example because special constructions or non water retaining objects are present in the dike section or because certain assumptions in the design of which inspection in practice indicated must continue to be valid;
- This information, together with the design report and the terms of contract comprises all the information that the managers of the water retaining structure need for the management of the dike. Specific maintenance regulations, for example for a grass cover or special structures, can be added to this.

11 Case study

□

11.1 Introduction and project description

The previous chapters described the general design of water retaining soil structures from a geo-technical perspective. In this chapter a number of the issues covered will be considered on the basis of an example. This is primarily intended to illustrate a dike improvement project, so a number of integral aspects such as the feasibility of translating results to other dike stretches and the choice of a representative transverse profile for a full stretch are not covered.

The example in question is a section of the dike along the Zwarte Water, north of Zwolle, the Netherlands. At the end of the nineteen-nineties, it was necessary to assess this water defence and, if necessary, to adapt it to meet the norms described in the Water Defence Act and worked out in the applicable guidelines and technical reports. In order to minimise the potential changes it was decided to introduce an inflatable surge barrier near the N 50 road bridge over the Zwarte Water at Ramspol. When inflated this 'bellows barrier prevents rising waters backing up from the IJsselmeer from flowing in the direction of the Zwarte Water and therefore leads to lower normative water levels in the area located behind the it. It was predicted that this measure would have a favourable impact on the safety of the water defences.

The construction of an inflatable barrage changed the character of the river system behind it. The elimination of the influence of the IJsselmeer meant that the river system could assume the characteristics of the upper river courses. Initially, the condition of the water defence along the Zwarte Water was audited in accordance with the applicable Guidelines and Technical Reports. The results of this soil-mechanical investigation were disappointing in that the changes on the basis of this investigation would be proportionally greater than was anticipated, considering the additional construction of the bellows barrage. As a result of this, and thanks to the availability of the 'actual strength' method, an investigation was performed to discover whether, within the design conditions and primary assumptions set by authorised body, there were any other possible ways to optimise the results of the soil-mechanical principal investigation. The final investigation performed on the basis of the 'actual strength' method led to a considerable reduction in the necessary improvement works and design dimensions.

The process described above is highlighted on the basis of a standard profile. The standard profile is located at the Driehoek along the Zwarte Water (see Figure 11.1.1) and is normative for that dike stretch.

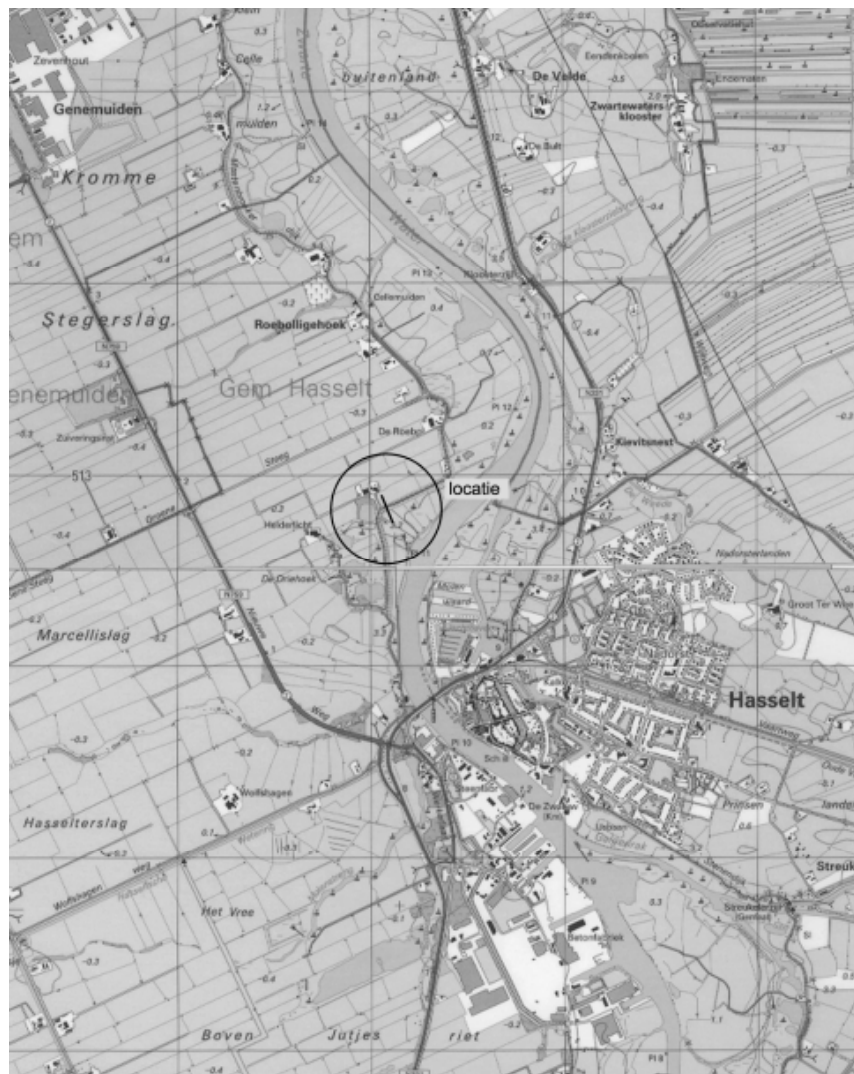


Figure 11.1.1 Location of the model profile the Driehoek along the Zwarte Water.

11.2 Data soil-mechanical investigation

11.2.1 Introduction

This section provides the basic data for the project: soil structure, soil surveys, loading and other information.

11.2.2 Soil survey, geology and soil structure

To determine the structure of the substratum along the Zwarte Water for the soil-mechanical investigation, fieldwork was performed and the geology of the area was investigated. The fieldwork consisted of:

- Electromagnetic measurements
- Penetration tests
- Deep machine drilling samples
- Shallow manual drilling samples

The geology of the location in question along the Zwarte Water is largely determined by sediments deposited in the Pleistocene and Holocene periods under the influence of rivers, sea and wind.

The Holocene is characterised by a rise in sea level, with the development of peat in lower areas and of sea clay deposits. The lower location of the land along the Zwarte Water led to the development of a moist environment that allowed the formation of peat. The area adjacent to the Zwarte Water was exposed to frequent inundation during which sea clay was deposited. The local Holocene series consisting of clay and peat may be interspersed

with river dune or sandy river sediments. The Pleistocene sand is located under the Holocene series. The top layer of the Pleistocene sand, with an average thickness of 5 metres, is part of the Twente formation and has a relatively small average grain size of 0.2 millimetres. The coarser sands are located under the Twente formation, and are part of the Kreftenheye formation. This layer is some 50 metres thick. The average grain size of this sand is determined at 0.5 millimetres.

The soil structure was determined on the basis of the site survey and geological considerations, and geo-technical longitudinal profiles were set for the inner toe, crown and outer toe of the water defence. Along the Zwarte Water, there is a virtually continuous Holocene series consisting of peat and clay. The peat consists primarily of 'Hollandveen' and is 2 to 3 metres thick. Locally, the peat is covered by a thin layer of clay of the Duinkerke formation that is between 0.2 and 0.6 metres thick. The covering clay layer of the Duinkerke series is missing at the point of the standard profile, and the entire Holocene series is 2.3 metres thick, while its thickness in the hinterland is 2.5 metres. As a result of the weight of the dike, the thickness of the peat package under the dike decreased to 0.8 metres. The dike itself consists entirely of clay that has been placed on it. The crown is located at NAP + 3 metres. The Pleistocene sand located under the Holocene series forms the massive uncompressible base.

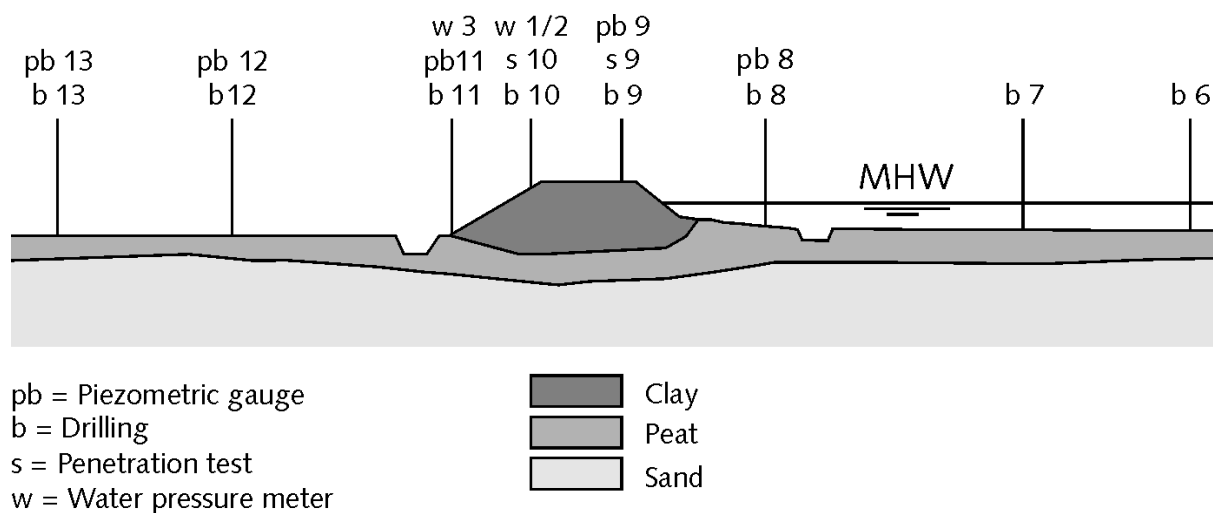


Figure 11.2.1 Profile at the Driehoek

11.2.3 Laboratory investigations and parameter determination

To determine the characteristics of the soil types at the standard location, undisturbed samples were taken and laboratory tests were performed. Regional test collections for strength parameters were set for the compressible soil types using statistical methods. These were for the following types of soil:

- Unloaded *Hollandveen* (adjacent to the dike, HVN)
- Loaded *Hollandveen* (under the dike, HVO)
- Dike material (OB).

These test collections were used to set the characteristic values of the strength parameters by means of statistical manipulation. The characteristic values of the strength parameters are corrected by a material factor in order to take into account differences between testing methods and uncertainties in the description of the material behaviour. The corrected characteristic values of the strength parameters (calculation values) are shown in Figure 11.2.2

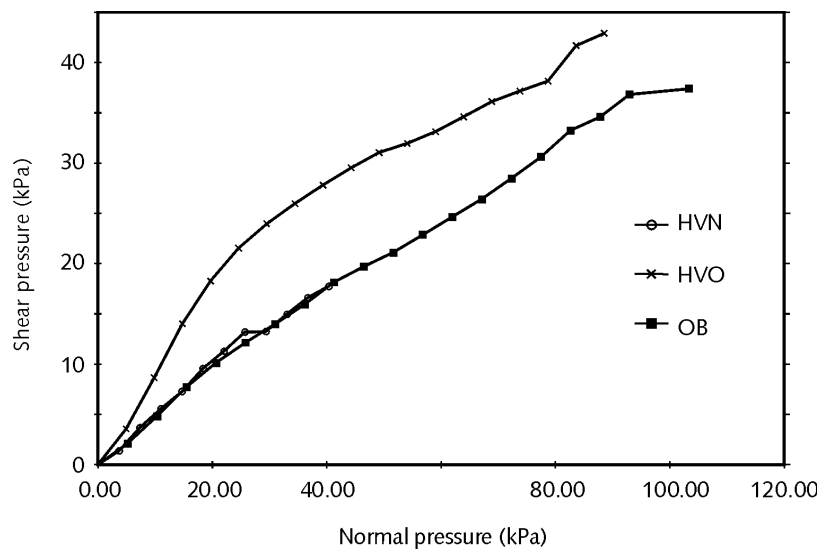


Figure 11.2.2 Calculation values for strength parameters according to the soil-mechanical

The incompressible soil types were not tested in the laboratory. An effective angle of internal friction of 30 degrees and an effective cohesion of 0 kN/m² were assumed for the Pleistocene sand in accordance with NEN 6740.

11.2.4 Other information

The maximum loading of the soil body occurs under normative conditions. The normative high water level (NHW) is NAP + 1.5 metre for the Driehoek. The polder level is NAP - 1.3 metres. The dike crown is at NAP + 3 metres. There is a metalled road on the dike, such that traffic loading must also be taken into account. NHW

The Safety Monitoring Guide gives the following processes as additional hydraulic loading:

1. Local gusting, storm squalls and storm oscillations
Not applicable in the Upper River Area
2. Wave overtopping
Application according to guidelines in the *Safety Monitoring Guidelines*
3. Waves in outports
Not applicable in the standard area
4. Currents
Not applicable in the standard area
5. Shipping-induced waves
Not applicable under NHW conditions
6. Wave preconditions under lower than normative water levels
NHW conditions are considered normative
7. Seepage currents
Part of the test for the occurrence of piping

Other loading is described in the *Safety Monitoring Guidelines* as influences or processes that may threaten the functionality of the water defence. The following processes and influences are included under other loading:

- 1 Ice
Not applicable in the standard area
- 2 Traffic
To be included in calculations as set out in the Technical Report on Soil Structures
- 3 Wind

- No direct influence on the water defence. Wind loading is only applicable for high structures and trees.
- 4. Collisions and floating objects
Only applicable for hydraulic structures that can be closed
- 5. Earthquakes
Not applicable in the standard area

11.3 Tests and Design Consequences Soil-mechanical Investigation

11.3.1 Introduction

Safety testing of the standard profile at the Driehoek was carried out in accordance with the Safety Monitoring Guide. The tests cover the following parts:

- Crown height (HT)
- Piping (STP)
- Inward macro-stability (STM)
- Outward macro-stability (AF, STV)
- Micro-stability (STU)

11.3.2 Crown (crest) height (HT)

The score of the test for crown height is determined by the absolute crown height, the accessibility of the crown, the amount of overloading, retention and erosion of material and the stability of the crown and the inner slope. The absolute crown height is determined by:

- Crown height reduction as a result of settlement (HT)
- Wave run-up (STP)
- Local gusting and the influence of storm squalls (STM)
- Additions for high water increases (STU)

Step 1:

Considering the soil structure and the length of time since the last increase in height of the dike, the expected settlement is 0.05 metres. The current crown height is NAP + 3.0 metres and the NHW is + NAP + 1.5 metres. The settlement allowance therefore meets the minimum demand of 0.5 metres.

Step 2:

Local gusting and the influence of storm squalls are not considered significant enough to be included in the assessment. Calculation of the wave run-up height according to the applicable Guidelines and Technical Reports results in a height of 0.26 metres, so that the crown height meets the rules of the applicable Guides and Technical Reports.

Effective fetch (F)	350 [m]
Water depth foreland d)	2.0 [m]
Design value wind speed (u)	9.0 [m/s]
Cotangent angle of outer slope	2.5 [-]

Score HT Good

11.3.3 Piping (STP)

The occurrence of piping under normative conditions is determined by:

- The length of the seepage flow;
- The thickness, permeability and erosion sensitivity of the water-bearing layer;
- The presence and thickness of a weakly permeable top layer;
- The difference in potential over the length of the seepage flow.

The piping phenomenon will only occur if the seepage flow in the Pleistocene series reaches the ground level through the Holocene series. It is assumed that water flow entering and exiting the foreland and hinterland respectively may occur if the Holocene is less than 1.5 metres thick. This assumes a minimum thickness of 1 metre and a safety margin of 0.5 metres that takes into account variations in layer thickness and the presence of disturbed soil. In the event of greater thickness of the covering strata, neither the entering nor the exiting water flows be significant for the occurrence of piping.

The occurrence of out-flowing water on the inner side depends on the possibility of the occurrence of heave in the Holocene covering layers. The test for heave is made by comparing the hydraulic head of the water bearing strata under the ditch on the inner side of the dike with the weight of the overlying Holocene series. The hydraulic head of the aquifer is calculated according to the guidelines given in the *Guidelines for the design of river dikes Part 1* and is based on the following:

Permeability capacity(kD)	5000	[m ² /dl]
Permeability of covering layer (k)	0.1	[m/d]
Length of foreland (L1)	350	[m]
Length of dike body (L2)	25	[m]
Length of hinterland (L3)	250	[m]

The hydraulic head at the 2.5 m wide 1.0 m deep inner dike ditch is 0.11 [m] above NAP. For the assessment of the occurrence of heave according to NEN 6370 a safety factor of $\gamma = 1.1$ is used.

$$1.2 \cdot 10.5 + 0.3 \cdot 10 / (1.1 \cdot (0.11 + 2.80) \cdot 10) = 0.5$$

The reaction between the water pressure and the weight of the covering layer is 0.5, so heave can occur and the water retaining structure must be tested for the occurrence of sand carrying springs

Step 1

The seepage line length L between the point of entry and the exit point of the seepage flow is 26.8 metres. The difference in potential over the dike is 2.8 metres and the thickness of the Holocene strata D at the point of the ditch at the inner toe of the dike is 1.2 metres. The profile does not meet the safe dimensions for soil pressure and seepage line length.

Step 2:

The dike was not designed recently and the basis for the design is unknown.

Step 5:

A large number of soil samples were taken for which grain distributions were determined, so it was possible to analyse the design demands.

Step 6:

Applying the Bligh method (equation 5.6.5) with a coefficient C_{creep} determined from one of the grain distributions gives the following result:

$$L_b = C_{\text{creep}} \Delta H = 17 \cdot 2.8 = 47.6 \text{ metres}$$

The length of the seepage line length is therefore 19 metres too short.

Step 8

The seepage line length is L greater than 10 H, so the Sellmeijer method can be used. According to the Sellmeijer equation, the critical gradient should be greater than the critical gradient under NHW conditions. The normative gradient under NHW conditions follows from

the difference between the normative river level and the polder level (pl) multiplied by a safety factor of $\gamma = 1.2$ which covers the uncertainties in the model and the parameters that are used (Equation 5.6.4).

$$\Delta H_{\text{strength}} > \gamma \Delta H \text{ with } \Delta H = (\text{NHW} - \text{pl})$$

The grain distributions are used to determine characteristic values for the parameters for the application of the Sellmeijer method.

The following characteristic values were determined:

- $d_{70, \text{rep}} = 0.228 \text{ mm}$
- $d_{60, \text{rep}} = 0.205 \text{ mm}$
- $d_{10, \text{rep}} = 0.085 \text{ mm}$.

The Sellmeijer method indicates that the demand is not met. The seepage line length required according to the Sellmeijer method is 33.6 metres which is 5 metres less than is required.

Step 9

Considering the high permeability of the water-bearing strata and the relatively long duration of the NHW period, non-stationary calculations are not an option, so score 'unsatisfactory' has to be given.

Score STP: Unsatisfactory

It must be noted that a theoretical score of unsatisfactory does not necessarily mean that in practice the dike is actually unsatisfactory. The residents who have lived along the dike for many years can unerringly indicate springs and instabilities. Whether or not piping is a problem can be more easily assess from this information than on the basis of a theoretical calculation. The results of slip plane calculations can also be tested against information from residents or managers. In other words, possible unstable parts of a dike (deformations, partial shearing) can be recognised in the field: if not the results of the calculations must open to doubt.

11.3.4 Inward macro-stability (STM)

The test for inward macro-stability is based on a situation T with a normative high water level.

Step 1

The dike consists of clay and the substratum consists of a series of Hollandveen (peat) on the Pleistocene sand. The geometry is described by the following parameters.

D	=	1.9	[m] average thickness of compressible strata under the dike
N	=	2.0	[-] cotangent of slope of the inner slope
H	=	3.6	[m] height of the dike with respect to the the ground level of the hinterland
o.b.	=	1.0`	[m] over-width of crown
A	=	>25.0	[m] length of the hinterland with a level ground surface

According to the *Guidelines on safety monitoring* the following factors are derived:

F_k	1.1 (-)
F_a	0.5 (-)

The test gives the following result

$$\text{o.b.} \geq F_k \cdot H \quad \text{and} \quad A \geq F_a \cdot H$$

$$1.0 \geq 1.1 * 3.6 \text{ and } 25 \geq 0.5 * 3.6 \quad \text{Not satisfactory}$$

$$o.b. \geq F_k * H \quad \text{and} \quad A \geq F_a * H$$

$$(B_{cr} - o.b.) + (h_{cr} - h_d) * (n + 1) \geq k$$

$$(4.0 - 4.0) + (2.95 - 2.0) * 3 \geq 3 \quad \text{Not satisfactory}$$

Step 2

The dike was not designed recently and the basis for the design is unknown.

Step 4

During the soil investigation, sufficient investigations were performed to permit a test according to the Guides and Technical Reports. Further investigation in the form of stability calculations according to the Bishop method is possible. According to the applicable Guides and Technical Reports, a minimum damage factor of 1.1 should be used when calculating inward macro-stability. A score of 'good' for inward macro-stability can therefore be given for safety factors of 1.1 or higher.

When calculating safety factors, the hydrological boundary conditions must be determined. The effect of the formation of the phreatic line in the dike body and the effect of uplift in the hinterland is important to the stability of the dike. In considering the inward macro-stability of the dike two shearing mechanisms can be distinguished: the regular slip along circular planes and a press bar mechanism that occurs under uplift conditions. The possibility of uplift occurring in the hinterland and the initiation of press bar (compression) mechanisms is determined by calculating of pore pressure in the aquifer. The measurement is carried out according to the guidelines given in Guidelines for the design of river dikes Part 1 and has been described in § 11.3.3 Piping (STP). Testing the stability of the dike body when the press bar staff mechanism occurs can be carried out by an adaptation of the standard Bishop method that is used for circular slip planes. The uplift zone that is without resistance can be simulated by not giving any strength to the Holocene series in the uplift zone.

The phreatic line in the dike body during NHW conditions is dependent on the type of soil of which the dike is composed. In weakly permeable clay if there is sufficient precipitation an upward curvature of the phreatic line occurs. The height of the curvature depends on the permeability of the clay. The clay in the dike is not homogeneous in composition so the most reliable way to measure the water pressure in the dike body is to determine the degree of curvature in the phreatic line. The results of this measurement are given in Figure 11.3.1.

On the basis of the results of the measurements, the following assumption was used for the stability calculations: in a dike with clay core, the phreatic line is 1 metre under the dike crown. On the outer slope side, the phreatic line runs linearly to NHW height. On the inner slope side, the phreatic line runs linearly to the dike's toe.

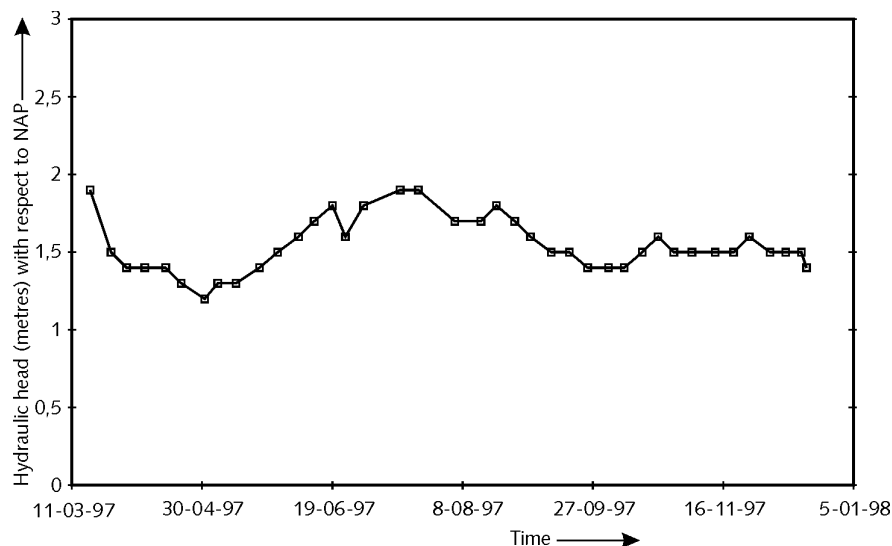


Figure 11.3.1 Water pressure measurements in the model profile

In the test, in addition to the hydraulic boundary conditions traffic loading of 13 kPa was assumed.

Step 6

The stability factor gives a safety factor 0.45 (uplift situation). The initiation point of the slip circle in Figure 11.3.2 is in the inner slope so it is necessary to ascertain whether this shearing can be considered normative. If the shearing does not affect the minimum required profile to prevent inundation a differentiation of the damage factor can be applied. The minimum required profile to prevent inundation is determined according to the *Construction design manual*. The following parameters are used:

NCH	=	1.9	[m] normative crown height
k	=	3.0	[m] minimum required crown height
n	=	2.0	[-] minimum required cotangent angle of inner slope
H ₁	=	3.6	[m] difference in height between crown and hinterland

Zone 1 = 3 + 3.6 = 6.6 m.

The initiation point of the slip circle occurs in zone 1 so the differentiation of the damage factor is impossible.

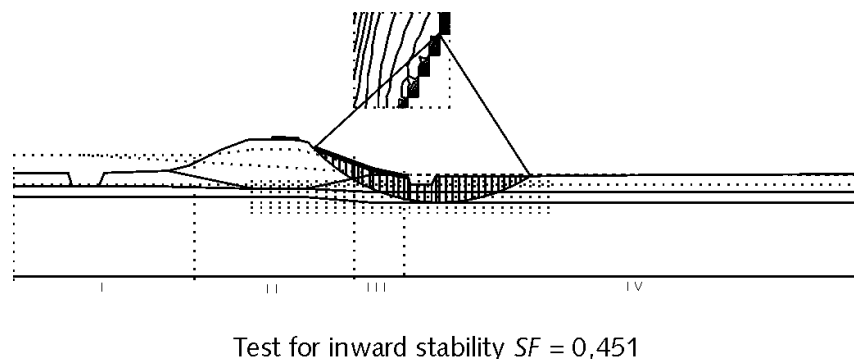


Figure 11.3.2 Test for inward macro-stability stability

Step 11:

Proof of the good behaviour of the water defence from a historical loading situation proves to be unreliable, so that a score of 'unsatisfactory' has to be awarded.

Score STM Unsatisfactory.

11.3.5 Outward macro-stability (AF, LB, STV)

The outward stability is assessed on the basis of the occurrence of settlement flow and shearing, and the assessment of stability in the event of a rapid drop in the outer water level.

Step 1.

The assessment for the occurrence of shearing is the first step in the test for outward stability. There are no known areas prone to weakening in the foreland. Tests for shearing (AF) should therefore be performed.

AF 1 The dumping criterion is met

AF 3 The channel depth is less than 9 metres

Score AF Good

Step 2

The second step relates to the assessment of the situation during a rapid fall in outer water level (LB). The dike consists of clay and the substratum consists of Hollandveen strata over the Pleistocene sand. The geometry is described by the following parameters

V 350 [m] length of foreland
N 2.5 [-] cotangent of angle of inner slope
D₁ 1.9 [m] average thickness of compressible strata under the dike
H₁. 3.5 [m] difference in height between crown and foreland
h 2.0 [m] difference in height between NHW and the foreland
D₂ 50 [m] thickness of the layer that does not consist of clay with humus

$$V \geq 1.5 * H_1.$$

$$350 \geq 1.5 * 3.5 \quad \text{Satisfactory}$$

$$n \geq 3$$

$$2.5 \geq 3 \quad \text{Fails}$$

$$h + D_1 < 5$$

$$2 + 1.9 < 5 \quad \text{Satisfies}$$

$$D_2 \geq DH_1 + D_1$$

$$50 \geq 3.5 + 1.9 \quad \text{Satisfactory}$$

The outward stability with low water on the outer side cannot be accepted on the basis of geometric determinations. Further investigation of the form of the stability calculation according to the Bishop method is necessary.

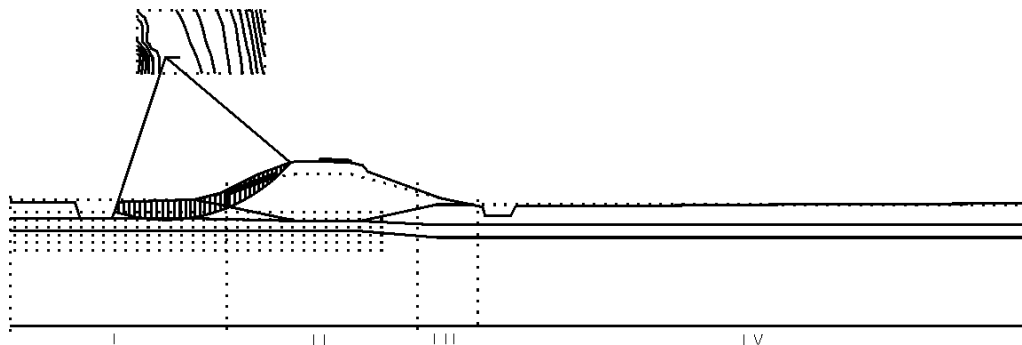
Step 3

According to the Guidelines on River dikes Part 2 Lower rivers, a stability factor of 1.0 should be used in calculations of the outward stability. This means that the calculated safety factor should be greater than 1.0 to attain a score of 'good'.

The test for outward stability is based on the following assumptions.

- In a dike with clay core, the phreatic level remains unchanged after the drop in river water level (see inward stability). The outer slope of the dike emits water during the time of the decline in water level, so the phreatic line falls to 40 centimetres below the ground level of the outer slope. Under the dike crown the height of the phreatic line remains unchanged;
- In the absence of foreland above 0 metres + NAP, the phreatic line in the foreland drops to 0 metres + NAP;
- In the presence of foreland above 0 metres + NAP, the phreatic line falls to the ground level in the foreland.

In addition to the hydrological boundary conditions, in the calculations traffic loading of 13 kPa was also assumed for the test of outward stability.



Test for outward stability $SF = 0,608$

Figure 11.3.3 Test for outward macro-stability

The results of the stability calculation give a safety factor of 0.61, so a score of 'unsatisfactory' has to be assigned.

Score STV Unsatisfactory

As far as back as can be remembered there has never been shearing of the outer slope after high water. Evidently things behave differently in practice than they do in theory. On the basis of experience, the advisor can determine whether a calculated slip off will affect the water retaining capacity of is undesirable from a maintenance point of view. If, by chance, a high outer water level does occur, the advisor will be able to give optimal advice that can be satisfactorily explained to the parties involved and takes into account the advice of the Boertien Commission.

11.3.6 Micro-stability (STU)

The test for the occurrence of micro-stability is based on an equilibrium condition in the top layer of the inner slope of a dike under normative conditions. Instability in the top layer of the inner slope under normative conditions cannot occur under the following conditions:

- An almost normative loading has occurred and no loss of micro-stability was observed;
- There is a well-functioning drainage structure in the inner toe of the dike, so a rise in the phreatic line in the dike body is limited;
- The dike has an impermeable clay core, so a rise in the phreatic line under NHW conditions is limited;
- The core of the dike consists of sand and the inner slope has a gradient of 1: 5 or less, so that the outflow gradient is too low to displace soil material.

Step 1

The dike has a clay core, so that a score of good can be awarded.

Score STU Good

11.3.7 Design consequences and conclusions

In the foregoing assessment of the water defence, it was discovered that the dike exhibits shortcomings with respect to the occurrence of piping and inward and outward macro-stability.

The shortcoming in the form of a seepage line that is too short to prevent piping can be solved by filling in the ditch on the inner toe of the dike. In this way, the potential exit point is removed with a sufficiently thick layer thickness in the hinterland of more than 1.5 metres.

Constructing an inner dike support berm can solve the shortcoming with respect to inward macro-stability. The dimensions of the support berm are determined by means of a stability calculation in which the minimum demand for the safety factor of 1.1 is met.

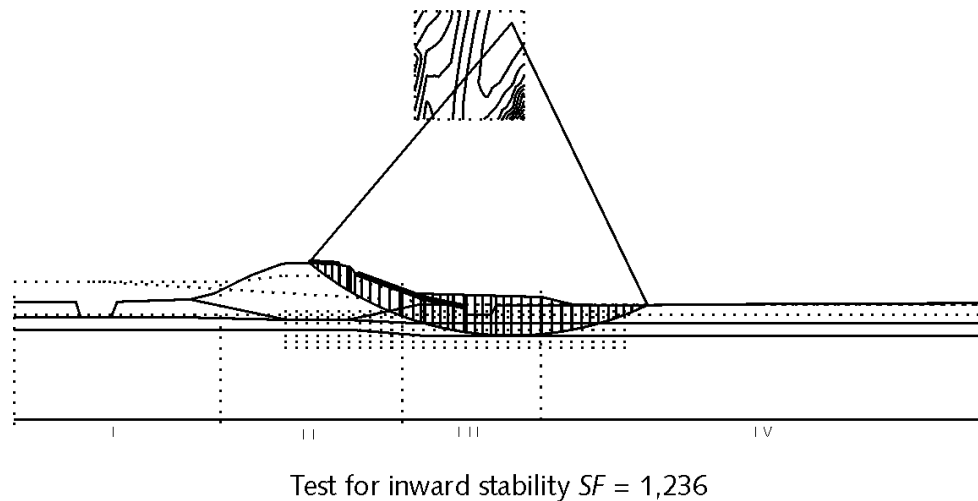


Figure 11.3.4 Dimensions of the inner support berm. Length = 10.5 m thicken = 0.9

The shortcoming with respect to outward macro-stability can also be solved by constructing a support berm.

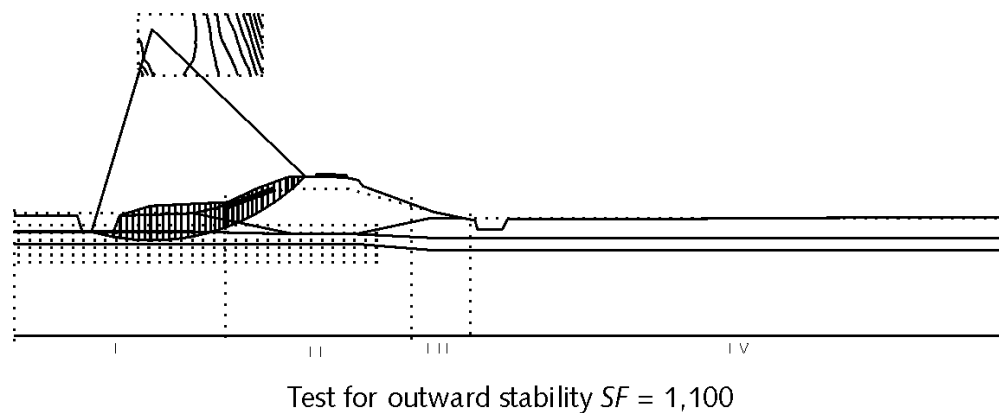


Figure 11.3.5 Dimensions of the outer support berm. Length = 6.5 m thickness = 1.0

11.4 Testing actual strength

Because the Ramspol inflatable surge barrier was installed it was possible to reduce the normative high water level. In the past water level occur that were higher than the present NHW. The information acquired then, together with information of resident and from the high water levels of 1993 and 1995, can be useful in possible plans for dike strengthening.

The testing has shown that the results of the soil-mechanical investigation could not form a directly basis for further development of the plans. The results were disappointing to such a degree that the changes on the basis of this testing would turn out to be on a larger scale than expected previously on the basis of the construction of the inflatable surge barrier. For technical details, reference should be made to the *TAW Research Report actual/existing strength*, or to the *Technical Report actual/ existing strength*.

Within the framework of the testing and the design of a prototype profile, two methods were used to gain a better estimate of the current strength of the water defences. The historical investigation method gave a good indication the strength of the water defence, which was greater than had previously been assumed. The actual strength as compared to the strength determined from the soil-mechanical investigation is in part supported of the local investigation method.

Annex 1

Characteristic estimates of the shear strength

1.1 Introduction

A geo-technical analysis is based on characteristic estimates of the average values of the soil characteristics within the soil layers at the site of the location for which the analysis is conducted. A characteristic estimate is understood to mean an estimate for which there is only a small probability of deficiency (in the case of strength characteristics) or exceedance (in the case of loads) by the actual value. In the Dutch provisions and guidelines, including the Geo-technical Standard (NEN 6740) or the TAW guidelines, a 5% probability of deficiency or exceedance is generally assumed.

The term 'actual value of soil characteristics present' requires some explanation. When measuring the soil characteristics, the value that is representative of the volume of the soil that is used in the measurement is determined. For a triaxial test, for example, that is a soil sample that does not exceed a few hundred cm³, while for on site measurements, such as soundings, the stability zones in the soil are never more than a few decimetres. Geo-technical analyses are concerned with slip planes (in the case of equilibrium analyses) or soil volumes used (in the case of deformation analyses) with measurements of the order magnitude of tens of metres or cubic metres respectively. The dimensions of the soil samples in laboratory or in on site tests are therefore small in comparison to the dimensions of slip planes or volumes concerned in the actual deformation problems. The measured values of soil characteristics are accordingly indicated as 'point values'. For example, within slip planes and other deformations, point values of the soil characteristics may vary. For the equilibrium of deformation analyses the averages of the soil characteristics within the slip planes or soil volumes used are important. The 'actual value present' in the above paragraph means the average presence of the soil characteristic within the slip plane or volume. Variances in point values are more or less averaged out.

In Dutch geo-technical practice, this idea has led to the procedure of working with the average of the test series (within a soil layer) when determining soil characteristics on the basis of laboratory or on site tests. In fact, these are characteristic estimates of the average values for the soil layer within the study area (that is the geographical area from which the soil samples are taken or in which the on site measurements are made. It is necessary to consider to what degree averages within such a study area are representative of a slip plane or volume in the case of a deformation problem

This may be assumed as long as the study area more or less corresponds to the area in which the slip plane or the soil volume is situated. In such a case it can be termed a local study. When the study area is much larger, as is the case for regional test collections for example, it is necessary to take into account that the averages over slip planes or soil volumes within the study area may deviate substantially from the overall average of the whole study area. In this case the average of the test collection is not representative of the average along the slip plane. When determining the characteristic values for geo-technical analyses on the basis of regional test collections the distribution of averages along a slip plane within the study area must be taken into account. As a result, the method deviates from that used to determine characteristic values from local soil studies.

In the Dutch norms and guides, in accordance with the Eurocodes, a semi-probabilistic set-up is used for the mathematical analyses. The characteristic values for loads are multiplied by partial safety factors, the load factors, while the characteristic values for strength characteristics are divided by partial safety factors (the material factors). In this way, we find the design or calculation values used as input in the analysis. For the underlying philosophy of this method reference may be made to § 5.8, which examines the method of determining characteristic values from a random sample of observations. The survey is focussed on shear strength parameters, but in principle the method is also applicable to other soil-mechanical parameters.

The distribution of test results on a series of soil samples is generally attributed to spatial variables of soil characteristics (within one specific soil layer). Although the possibility cannot be excluded that any non-reproducibility of soil-mechanical test is partly caused by the

spread in test results, it is assumed below that the contribution of spatial variables is dominant. For a good understanding of the method it is desirable that there should be insight into the underlying (generic) conceptual model for spatial distribution; this model is handled first. Subsequently techniques for determining characteristic estimates in the case of local soil studies and, in the case of the use of regional test collections are discussed.

1.2 Conceptual stochastic model for spatial spreading

The model is described for one shear strength parameter, for example undrained shear strength C_u , within a soil layer. It is assumed that, within the soil layer under consideration there is no provable or explicable trend, such as an increase in the depth. Although on average the value of C_u is approximately the same, different C_u values will be measured at different places within a layer due to spatial variances. It is assumed that C_u continuously fluctuates unpredictably from place to place within the layer around the overall average value, but that the character of the fluctuations (that is the size of the deviation with respect to the overall average and the 'velocity' of fluctuations) is approximately the same overall within the layer.

The variance pattern of C_u can then be modelled as a stationary stochastic field. That means that the actual value at every point within the layer is understood as a stochastic function, and that the value realised at a random point appears to be derived from a probability distribution function. The base characteristics of the probability distribution function are the probability distribution type, for which in this case the normal distribution is taken, the expectation value μ_{cu} (representing the 'overall average') and the standard deviation s_{cu} (reflecting the size of the fluctuations with respect to the average). 'Stationary' means that when the two random positions x_1 and x_2 within the layer are examined, the expectation values of $c_u(x_1)$ and $c_u(x_2)$ are equal ($E[c_u(x_1)] = E[c_u(x_2)] = \mu_{cu}$), that the standard deviations are equal ($\sigma[c_u(x_1)] = \sigma[c_u(x_2)] = \sigma_{cu}$) and that the correlation between $c_u(x_1)$ and $c_u(x_2)$ is only a function of the distance components (horizontal, vertical) between the two points. In general, in the case of small distances the correlation is almost equal to 1.0, in which case there is continuity in the variance pattern, and with the increase in the distance components, the correlation is reduced to zero. The function that reflects the reduction of the correlation as a function of the distance components is called the autocorrelation function. In the literature various forms of autocorrelation function are indicated. The type that the considerations in this section are based on is

[EQUATION] (1.2.1)

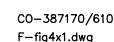
Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken.
(1.2.1)

Here ρ is the correlation between two points, as a function of δz , the horizontal distance between those points, and δy the vertical distance. The parameters D_z and D_y are called the horizontal and vertical correlation scale or correlation length respectively. The smaller the correlation scale, the more quickly the correlation decreases. This manifests itself as quicker fluctuations in the variance pattern (see Figure 1.2.1). It is generally assumed that in naturally deposited soil layers vertical fluctuations occur much more rapidly than those in horizontal directions do. Fluctuations in vertical directions represent variations in deposition conditions in time. If the parameter α is equal to 1.0 is chosen, the autocorrelation function changes to a simple e-force often used in the literature:

[EQUATION] (1.2.2)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken.
(1.2.2)

As a result a conceivable variance pattern for undrained shear strengths in a soft clay layer is given; with quick fluctuations in the vertical direction and relatively slow fluctuations in the horizontal direction. The conceptual model is as follows: when the shear strength is measured along a certain vertical, a spread in the point values will be observed in relation to the average over the vertical. If another vertical is chosen then, setting aside statistical random test effects, the same spread around the average of that vertical will be found. If all point measurements along a large number of verticals are added together, then the distribution in relation to the average of all observations is greater than the distribution in a single vertical. The parameter α is the relation of the variance of the spread within a single vertical and the variance of the spread of all measurements in relation to the average over the entire measured area. In other words, besides the distribution within a single vertical there is also a distribution of the averages over a vertical. In other words, there are verticals that on average are stronger than the average of the whole soil layer and there are verticals that are weaker than the average of the whole soil layer. If $\alpha=1$, then, besides the statistical random test effects, there is no difference between the verticals and the spread would be just as great along one vertical as the spread in the whole soil layer. This proposition is only acceptable when the entire measured area, that is the area in which the shear strength measured along the verticals, is not too large. For regional test collections the measured area often extends over many kilometres and in such cases the proposition that relatively strong and weak spots are not consistently found is less acceptable.



1.3 Averaging out spreading over slip planes

The parameters needed in geo-technical analyses are generally representative estimates of values for averages of soil characteristics over a slip plane or other volume. Owing to the spreading of point values and the fact that random tests to estimate parameters for geo-technical analyses are infinite by definition, the estimates for averages over slip plane are also uncertain variables. In other words, the average of a soil characteristic over a slip plane is also a stochastic function. However the distribution of that stochastic function is smaller than the distribution of point values of the soil characteristic, because averaging out occurs. This is shown as follows:

[EQUATION] (1.3.1)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.1)

where $\sigma^2(C_{u,G})$ is the variance of the average shear strength over a slip plane G and $\sigma^2 C_u$ the point variance and $\Gamma^2(G)$ the variance reduction factor. In the averaging out, two contributors play a role, namely averaging out in vertical direction and averaging out in horizontal direction. As slip plane dimensions in vertical direction (5 to 10 metres) are large in relation to the vertical correlation scale D_v , which is of the order of a few decimetres, the averaging out in vertical direction is large. Dimensions of a slip plane in horizontal direction are generally not large in relation to the horizontal correlation scale D_z , which is of the order of 50 to 100 metres or more. Accordingly, averaging out is limited in horizontal direction. An indication of the degree of averaging out, that is the size of the variance reduction factor is obtained by considering a 'rectangular vertical slip plane' with dimensions h in vertical direction and b in horizontal direction. The average shear strength over this slip plane is:

[EQUATION] (1.3.2)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.2)

The expectation value of this is:

[EQUATION] (1.3.3)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.3)

and the variance:

[EQUATION] (1.3.4)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.4)

Using the autocorrelation function (1.2.1) the latter can be written as:

[EQUATION] (1.3.5)

where:

[EQUATION] (1.3.6)

and

[EQUATION] (1.3.7)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.5)

where:

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.6)

and:

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.3.7)

In Figures 1.3.1 and 1.3.2 the development of the variance reduction factors are given in accordance with (§ 1.3.6) and (§ 1.3.9). A practical indication range for b/D is 0.5 to 1.5; Figure 1.3.1 shows that the variance reduction due to averaging out in horizontal direction in that case is relatively limited.

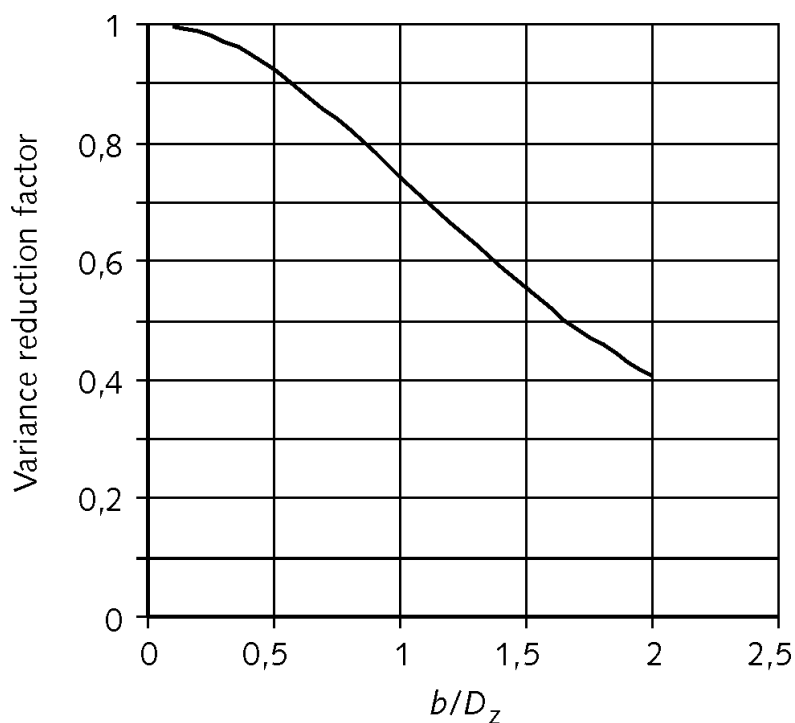


Figure 1.3.1 Variance reduction factor for averaging out in the horizontal direction

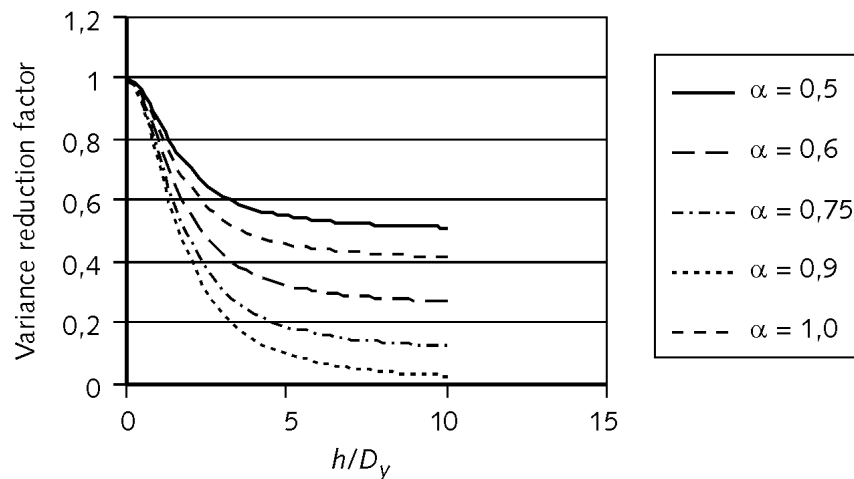


Figure 1.32 Variance reduction factor for averaging out in the vertical direction

Figure 1.3.2 shows that the normally large relation between slip plan dimensions in vertical direction (at least a few metres) and the vertical correlation scale (present assumption: a few decimetres) the variance reduction factor is often around the limit value, that is $(1-\alpha)$. That means that within a slip plane vertical fluctuations of shear strength are mainly averaged. Roughly speaking, this implies that an estimate of the average value of the shear strength in the soil layer at the site of the slip plane is a good indicator of the average shear strength within the slip plane.

Here a support appears to be found for the custom in Dutch geo-technology that was mentioned in the introduction to assume characteristic estimates for low averages for geo-technical analyses. In principle, this concerns low averages at the site of the more specific slip plane location for which the geo-technical analysis is conducted. The average shear strength of the soil layer can vary from slip plane location to slip plane location. The variance in this distribution is approximately equal to $(1-\alpha)$ times the point variance σ_{cu}^2 . In other words, the variance reduction factor $\Gamma^2(G)$ is roughly equal to $(1-\alpha)$.

1.4 Characteristic shear strength in local soil studies

In local soil studies the measured area, that is the area in which the on site tests are conducted or soil samples are taken for the laboratory tests, is limited to an area within which regional variances play (almost) no role. Given the above, the dimensions of such an area must be great with respect to the horizontal correlation scale D_z . This, for example, is not greater than 100 to 150 metres in the longitudinal direction of the dike and the same magnitude in the direction perpendicular to the dike. In this case the average shear strength found in the random test of observations gives a reasonable indication of the average shear strength present within the a slip plane in this measured area. The characteristic estimate of the average of the soil layer, assuming a normal population spread of shear strengths is

$$\text{[EQUATION]} \quad (1.4.1)$$

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.4.1)

where $c_{u,av}$ is the mathematical average of the random test, s_{cu} the standard deviation of the random test and $t_{N-1}^{0.95}$ the Student t-factor that corresponds to the 5% characteristic lower limit value and the number of degrees of freedom $N-1$, where N is the number of (independent) observations in the random test in the local soil study. In Table 1.4.1 values are given for the Student t-factor, as a function of the random test size N .

Table 1.4.1 Values of the student t-factor

N	3	4	5	6	8	10	15	20	∞
$T_{N-1}^{0.951}$	2.920	2.353	2.132	2.015	1.895	1.833	1.761	1.729	1.645

When the random test size increases the random test variance converges the population variance σ_{cu}^2 , and the Student t-factor increases to the 95 percentile value of the standard normal probability distribution, that is 1.645. In the case of a relatively large spread in the random test it is wise to assume a log-normally distributed population. In this way negative values of the shear strength are excluded, which is physically more realistic. This is done by first taking the logarithm of the random test observation and then the average and standard deviations.

In this way the characteristic lower limit is determined in accordance with (1.4.1) and the exponent of the result is determined by

[EQUATION] (1.4.2)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.4.2)

where $(\ln c_u)_{av}$, en $s_{\ln c_u}$ are the mathematical average and the standard deviation respectively of the logarithms of the random test results. It is noted that the greater the number of observations in the random test, the smaller the difference between the average of the random test and the characteristic lower limit. In the extreme case, that is the case of an infinite number of observations, this difference becomes zero. This difference reflects only statistical uncertainty about the expectation value of the average shear strength available at the site of the slip plane, which reduces as the number of observations in the random test increases. In determining the characteristic estimate, the only role played by the spatial variance of the shear strength plays i than that, in combination with the limited size of the random test, it creates uncertainty about the actual average shear strength within the slip plane. It should be noted that the spatial variance does play a role when 'point values' in the geo-technical analysis are concerned. As a result, in the equations for the characteristic estimates, besides the statistical uncertainty, the spatial distribution must also be allowed for. The equations are then:

[EQUATION] (1.4.3)

and

[EQUATION] (1.4.4)

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.4.3)

and

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.4.4)

An example of a geo-technical analysis in which the characteristic estimate of point values must be taken into consideration is a heave analysis. This concerns the thickness of the covering clay/peat layer at the critical location, and not the average thickness in an area. When a series of observations of the thickness, for instance from soundings, is available, the characteristic lower limit estimate of the thickness is determined with by using equations (1.4.3) or (1.4.4).

1.5 Characteristic shear strength in regional studies

In the case of regional soil studies the measured area in which the on site tests are conducted or soil samples for laboratory tests are taken (within the same soil layer) is of regional size. The database with test results is also called the regional test collection. The regional test collections came into being during the large-scale dike reinforcement works along the major rivers, particularly in the west of the country. The idea was that the results of soil studies, conducted for the design of dike reinforcement projects at various locations, would be useful for deriving initial indications of soil characteristics for new projects in the region. By collating the results of local studies in the various projects test collections are generated, making the random test size much greater than is the case with local soil studies and so the statistical uncertainty is smaller (Ref. GeoDelft 1997). The latter is true, in so far as it concerns the statistical uncertainty of the overall average of the soil characteristic in the entire region. Besides this uncertainty, the spatial variability of 'local averages' with respect to the overall average also plays a role. The variance here is, as derived from § 1.2 equal to $\Gamma^2_{\sigma_{cu}^2} = -\alpha) \sigma_{cu}^2$ in the reduction of the random test average (of the regional test collections) to realise a characteristic estimate of the average at a location. The equations, analogous to (4.10) and (4.11) are accordingly:

$$\text{(EQUATION)} \quad (1.5.1)$$

and:

$$\text{(EQUATION)} \quad (1.5.2) \quad \text{Query (1.4.3) in the Dutch text but in not the same as (1.4.3)}$$

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.5.1)

and

Fout! Objecten kunnen niet worden gemaakt door veldcodes te bewerken. (1.5.2)

Here the index $_{prv}$ indicates that they are averages and standard deviations of the regional test collections.

Until now, little has been said about the size of parameter α of the stochastic field, and so about the size of Γ^2 . Visual inspections of existing test collections of shear strength parameters (based on cell tests) for diverse layers of the substratum in the Alblasserwaard, indicate a range of values for α , varying from 0.5 and 1.0. In the *TAW Guide for the Design of River Dikes (Part 2, Tidal Rivers and Transitional Zones, 1989)* an value of $\alpha = 0.75$ ($\Gamma^2 = 0.25$) is assumed.

In the scope of the later TAW study (Ref. Calle and Van der Meer, 1997) an attempt was made to determine the values of α more closely by using formal statistical monitoring of test collection material. The disappointing result was that no unequivocal statistically underpinned judgements could be made on the size of parameter α on the basis of the available test collections. None of the formal hypotheses $\alpha = 1$, $\alpha = 0.75$ and $\alpha = 0.5$ can be excluded on the basis of the observation material available in the test collections. Parameter α is therefore an uncertain variable. In geo-technical engineering practice this means that a choice must be made, in which it is important that it must in any case lead to a reasonably safe design methodology. At the moment, the choice made in *Guide on River Dikes* would appear to be a wise one.

1.6 Pseudo-characteristic values

The above sections addressed in detail the estimate of a 5% characteristic lower level for local average shear strengths in a soil layer. In geo-technical slip plane calculations several

soil layers will generally play a role. In accordance with the valid norms and guidelines, account should be taken of a characteristic 5% lower limit for the average shear strength within a slipplane. When the slip plane only cuts through one soil plane the 5% characteristic lower limit is for the entire slip plane. If the slip plane cuts two or more (independently sampled) soil layers, the choice of the 5% characteristic lower limit for all soil layers involved provides an estimate of the average shear strength along the slip plane that has a probability of deficiency that is less than 5%. This method is accordingly somewhat conservative. This conservatism can be eliminated by assuming lower limit estimates for the individual soil layers that combine to make a 5% lower limit estimate for the average shear strength along the total slip plane. In general this leads to lower limit estimates for the individual soil layers that have a probability of deficiency that is less than 5%. These estimates are called pseudo-characteristic values.

The core of the method, which is described in (Ref. Calle, 1996] is that the relative contributions along a potential slip plane to the total shear resistance of the individual soil layer cross sections are considered. Based on the expectation values and standard deviations (in which both contributions of spatial variables and contributions of statistical random test uncertainty) of the average shear strengths in the individual soil layers, lower limit values for each of the soil layers are calculated. These are the pseudo-characteristic values, which combine to realise the shear resistance for the entire slip plane that only has a 5% probability of being exceeded. The basis for this calculation is a simplified probabilistic 'first order second moment' analysis. Naturally, the pseudo-characteristic values for the various soil layers must be calculated for each potential slip plane (for example, by making a Bishop analyse), as the relative contributions of different slip planes to the total shear strength of the slip plane are different. The calculation procedure is therefore practically useable only when it is built into the computer program for the slip plane analyses. The methodology is implemented in the GD computer program MSTAB for stability analysis according to Bishop's method. Practical experience with the methodology indicates that that this leads to approx. 10% higher calculated safety factors.'

References

Calle E.O.F. 1996. *Karakteristieke waarden voor schuifsterkte t.b.v. stabiliteitsanalyses*. (Characteristic values for shear strength in relation to stability analyses). Delft Soil-Mechanics report CO-350000/43, July

Calle E.O.F. and M.T. van der Meer, 1997 *Probabilisme in de Geotechniek, Onderdeel Ruimtelijke Variabiliteit, Fase A.II*. (Probability in Geotechnology, Spatial variability, Phase A.II.) Delft Soil-mechanics report CO-361410.95, December 1997.

Statistisch verwerking of cel- en triaxiaalproeven, (Statistical processing of cell and triaxial tests) Rapport GeoDelft SE-50676, February 1997.

Annex 2 ADP method

One of the internationally recognised ways to account for anisotropy is the ADP method (Active, Direct, Passive and Passive). The ADP method is an undrained stability analysis (OSA analysis) to determine the strength of soil structures and is often used in Scandinavia and the English-speaking world. This method is based on undrained collapse of the soil; that is that the change in load is fairly rapid and there is no time for the soil to adapt to the increased or decreased load. Increasing the height of soil structures Raising in one or more building phases can be allowed for.

The ADP method takes the anisotropy of the soil into account. To permit allowance to be made in a position to allow for the anisotropy, active, horizontal and passive collapse zones are distinguished. In Figure 2.1 the A (active), D (direct simple shear,; horizontal) and P (passive) zones are indicated for an ideal plastic material.

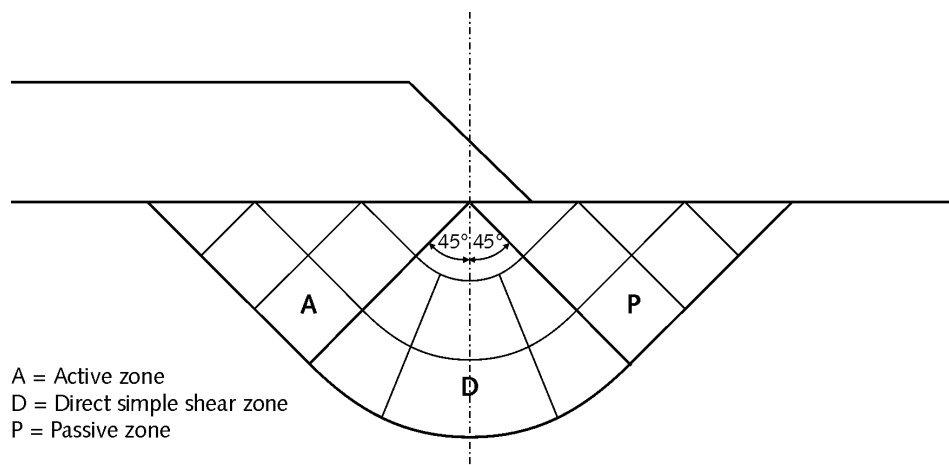


Figure 2.1 Undrained failure of an ideal plastic material

The method makes use of anisotropy consolidated undrained triaxial tests with shearing directions in compression and in extension (test types ATC and ATE respectively), as well as consolidated undrained pure shearing tests (DSS). The relation between the kind of laboratory test and the location in the slip plane is indicated in Figure 2.21.

Triaxial compression test (ATC)

This test simulates active collapse of the soil, by which the shear strength increases in the same direction as the shear stresses already present in the K_c condition. The natural structure remains intact and collapse occurs in the case of a relatively limited axial strain. After the plasticity limit is reached the strength is reduced. In English-language publications this is called 'strain softening' in English-language publications.

Triaxial extension test (ATE)

In this test, in which a passive collapse is simulated, the shear stresses are contrary to those initially present. During the test the shear stresses decrease to zero and then increase in the opposite direction. During the process there is substantial distortion of the natural soil structure and full plasticity is reached after large axial strains, after which 'strain softening' is no longer relevant.

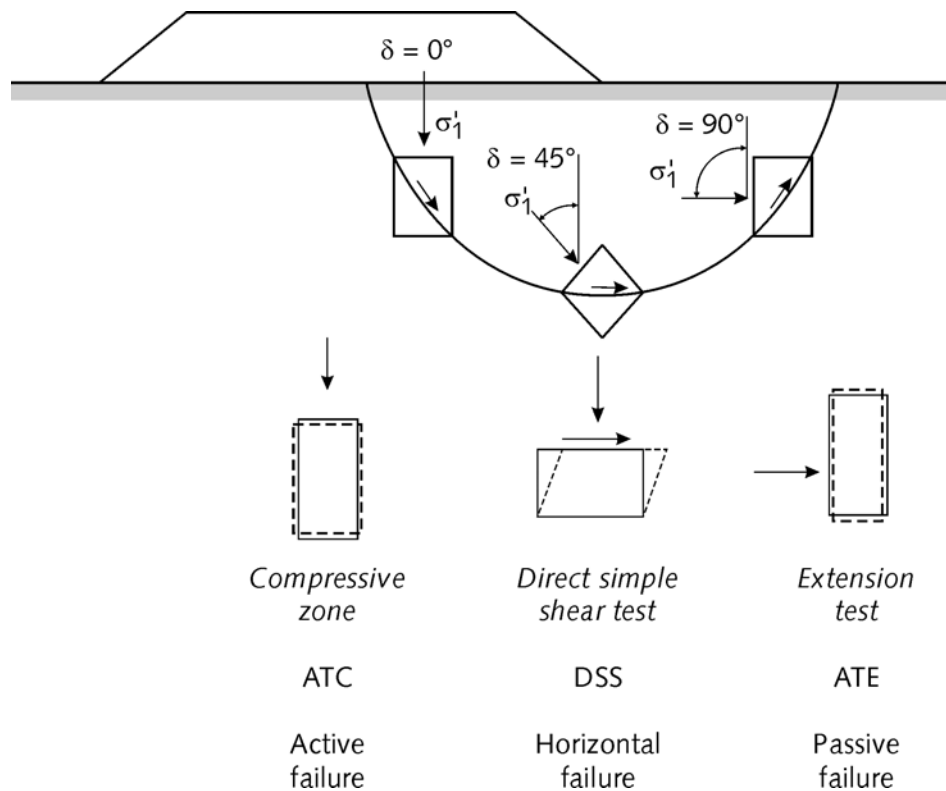


Figure 2.2 Slip cylinder with relevant laboratory tests

Direct Simple Shear test (DSS)

The DSS test, see section 4.4.8 for a description of this test, is used to determine the strength along a horizontal, or almost horizontal, part of the slip plane (see § 4.4.13 for a description of this test). In the test the main pressure rotates to 45° , which means that the disruption of the natural soil structure lies between the deformation/distortion of the ATE and ATC tests before full plasticity is achieved.

The undrained shear strength is determined as a function of the incline of the slip plane to the horizontal. In Figure 2.3, the anisotropy of the undrained shear strength is represented schematically.

The anisotropy is defined as follows:

$$\text{UNNUMBERED EQUATION} \quad (2.1)$$

$$K_a = \frac{f_{undr;ATC}}{f_{undr;DSS}} \quad K_p = \frac{f_{undr;ATE}}{f_{undr;DSS}}$$

When the height of a structure is raised At a raising as a result of several building phases forecasting and measuring the water overpressure continues to be limited to the forecasting and/or monitoring of the consolidation percentage. The undrained shear strength is calculated as follows:

$$\text{EQUATION} \quad (2.2)$$

$$f_{undr;bouwfase(n)} = f_{undr;bouwfase(n-1)} + \Delta\sigma_{v;n} \lambda \frac{f_{undr}}{\sigma'_{vo}}$$

Where:

$\Delta\sigma_{v;n}$ vertical total rise in pressure building phase n
 λ consolidation percentage of building phase (n-1) 1)
 $\sigma'_{v;o}$ initial vertical effective pressure.

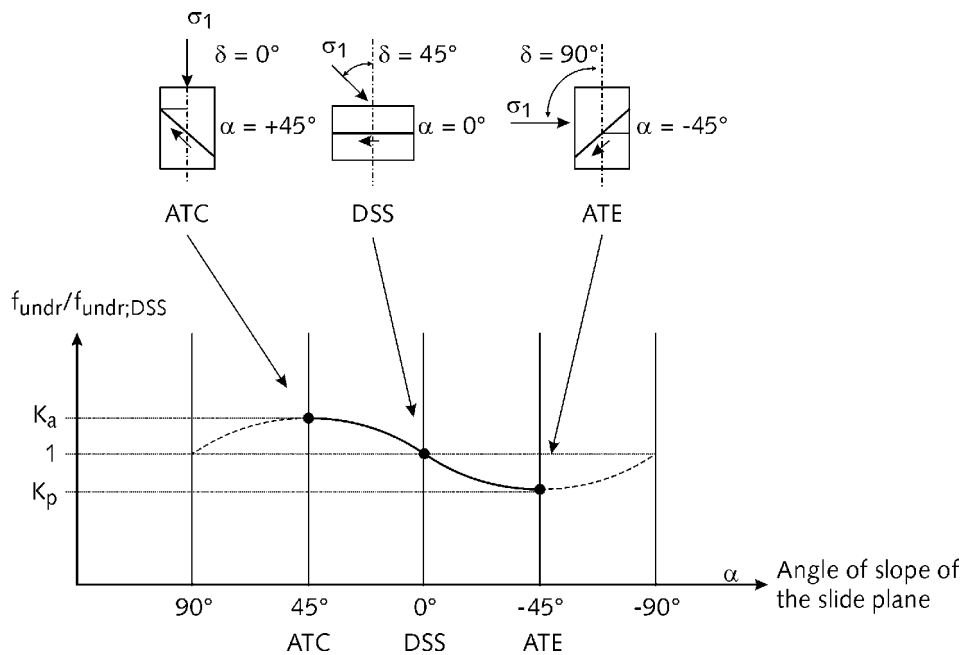


Figure 2.3 Anisotropy of the undrained shear strength

When determining $\Delta\sigma_{v;n}$ Preferably, it is preferable that the pressure distribution of the elevation raising is calculated. in the determination of $\sim O''v;n$. To limit the research costs, it is recommended that inexpensive methods are used cheap measurements to determine the basic shear strength profiles are recommended. Using these profiles the anisotropy factors K_a . and K_p are extrapolated in the laboratory, as well as the absolute size of the shear strength $f_{undr,DSS}$, so that a reasonably continuous ideampression can be formed of the substratume in a vertical or in a horizontal section is obtained ditch.

The basic shear strength profile can be found as follows:

- **On site vanefin tests (FVT) or Lab vanefin tests (LVT).**

The undrained shear strength $f_{undr;FVT/LVT}$ can be measured at depth intervals of approx. 0.5m. By applying a correction factor to this undrained shear strength the average undrained strength along a circular slip plane is approached. The correction factor μ_{FVT} [FVT of Aas] et al [(Ref. 1).] can be use for this. The correction factor also takes account of the differences between strain rate in between the test and in the actual shearing.

EQUATION (1.3)

in which:

EQUATION (1.4)

$$\mu_{FVT} = \frac{f_{undr}^*}{f_{undr;FVT}}$$

in which :

$$f_{undr}^* = \frac{f_{undr;ATC} + f_{undr;DSS} + f_{undr;ATE}}{3}$$

- **Penetration tests (CPMT, PT)**

For the determination of the undrained shear strength the cone pressiometer (CPMT) offers good prospects/opportunities. For the time being the correction factor μ_{CPMT} will have to be determined for each project for the time being, because no generally applicable relationship is not yet available. The measurement of the cosine resistance alone is too imprecise for the determination of the undrained strength, even for a class 1 penetration test.

The reduction in the undrained strength after the peak strength can also be taken into account. The ADP method does this takes account of this effect by determining the corresponding values of the active, horizontal and passive strengths, at the peak value shear strength ($\tau_{av,peak}$), determining the corresponding values of the active, horizontal and passive strengths. This procedure is also called Ladd's compatibility method [Ref. 2]. The procedure is represented schematically in Figure 2.4. Here a fixed slip plane is assumed/supposed. A high/large degree of 'strain softening' has an effect on the calculated stability factor and can lead to the sudden collapse of the soil structure without warning.

The standard ADP method gives a safety factor that is too high/safe and accordingly less suited to situations in which the soil structure has been present for a long time and in which the increase in load is limited in size. Under these circumstances the soil structure is much stronger than it would be the case of a standard application of the method. This In the English-language publications this effect is termed/described with the terms '*pre-shearing and evolving anisotropy*'. To determine the actual strength pressure path for this situation tests are need for this situation that are compatible with consolidated to the pressure present on site. For a relatively large increase in load and/or important cases/large interest the application of the ADP method is also worthwhile in this situation. If the increase in load is limited in size however, and of relatively minor significance the interests are relatively small an effective stability analysis (ESA analyse) is recommended.

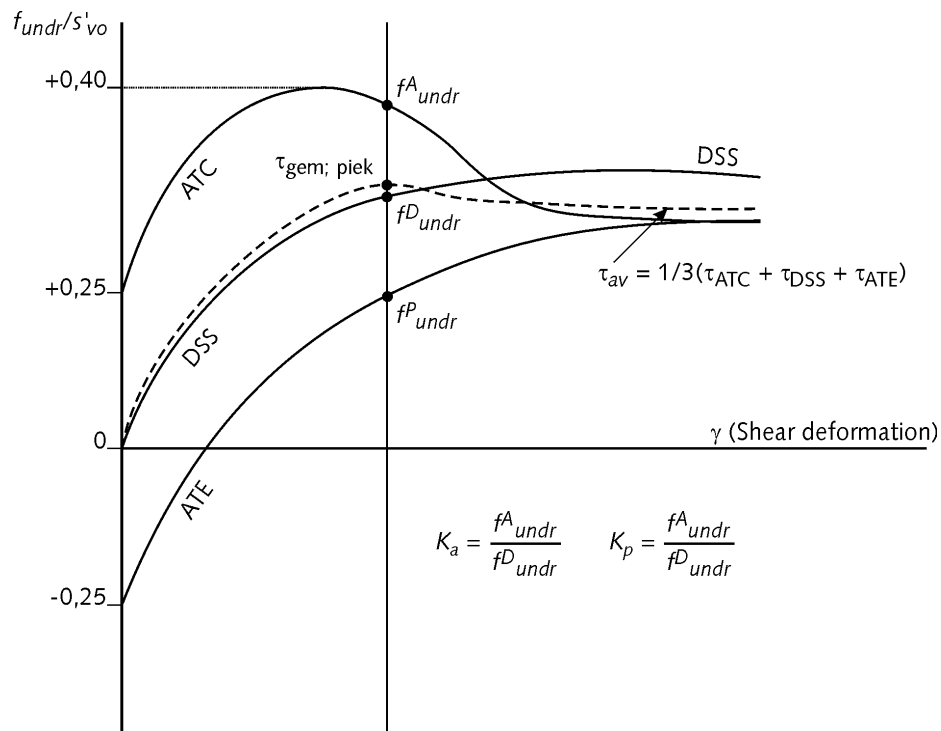


Figure 2.4 Determining undrained shear strength f^A_{undr} , f^B_{undr} and f^C_{undr} bearing in mind the compatibility of the strain and based on a fixed slip plane

References

- [1] Aas et al. (1986), Use of in situ tests for foundation design on clay. Invited lecture ASCE speciality conference 'In situ 86' Blacksburg, VA
- [2] Ladd, C.C. (1986), Stability evaluation during staged construction. The twenty second Tetzaghi Lecture
- [3] Pachén, M.A. en A.F. van Tol, Ongedraineerde Sterkte Analyse met de ADP-methode, Ingenieursbureau Gemeentewerken Rotterdam

Annex 3 Drilling

A short explanation of the drilling methods that are used in the Netherlands with its generally weak soils.

Hand drilling with an auger or gouge drill

This is a dry drilling method and can only be used in coherent soil that is not too weak, in which the drilling hole remains open and does not require lining. The depth that can be achieved depends on the soil conditions and varies between 5 and 15 metres. This method cannot be used in sand under groundwater level. The quality of this type of drilling is generally good. Because the soil is extracted in its natural state, it is relatively easy to assess its type and quality. This drilling method, in combination with a tapering pod gauge drill, is therefore perfectly suited for taking undisturbed samples.

Pulse drilling

This method of wet drilling, in which the drill hole is protected by tubular lining, is often used. It is particularly suitable for weak soil and sand. However the soil extracted with the drill has been stirred and mixed with water, to such a degree that only a general indication of its composition can be obtained. The quantitative properties of the soil cannot be measured by using this method. For soil-mechanical purposes, therefore, this type of drilling should always be combined with the taking of undisturbed samples. These are required for a more precise assessment of the soil profile, and for the determination of the mechanical properties of the soil layers. In practice, this has led to a special type of pulse drilling with continuous sampling; this is called Ackermann drilling (Figure 3.1). In this method, the pulses serve only to clean out the drilling hole each time a 0.3 metre sample has been taken and to prepare it to take the next sample.

When pulse drilling for the soil analysis, extremely accurate operation is required. If this process is performed badly, the soil under the base of the pulsed drilling hole may be disturbed over a depth of at least 0.1 to 0.2 metres. This may be caused by the upward flow of water resulting from water levels in the drill shaft being too low, or as a result of the pulse being extracted too quickly in which case it may function as a type of vacuum device in the drill hole.

□

Continuous drilling (Ackermann drilling)

For this method the Ackermann extraction device is used to continuously extract undisturbed samples of some 0.30 to 0.40 metres in length. These samples can be laid out, described and photographed in the laboratory, providing a continuous drilling profile. Samples can be selected from representative layers for testing in the laboratory. The method is reliable but as a result of the methodology can generate a certain amount of disruption. It is less suitable for sandy soil since the sample container remains entirely or partially filled only if catchers are used which at the same time may disturb the sample. If a continuous, virtually undisturbed sample is required, the more expensive Begemann drilling method should be applied.

Continuous drilling (Begemann drilling)

With this technique, it is possible to extract a continuous, undisturbed sample of soil down to a depth of some 20 metres in the Pleistocene sand. It is possible that the sleeve may tear as a result of the presence of an intermediate layer. In fact, this method uses penetration extraction as oppose to drilling. The continuous penetration device, with its extension tubes is usually pressed into the soil by a probe device. The continuous drilling method can be used with a diameter of 29 mm to 66 mm.

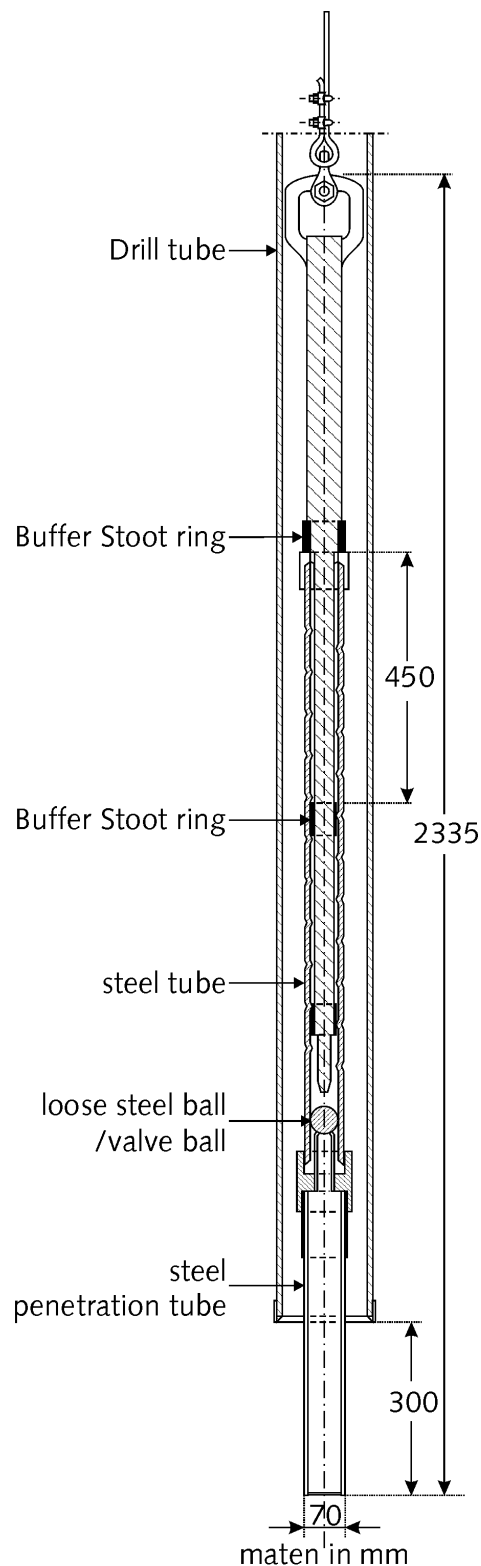


Figure 3.1 Ackermann drilling

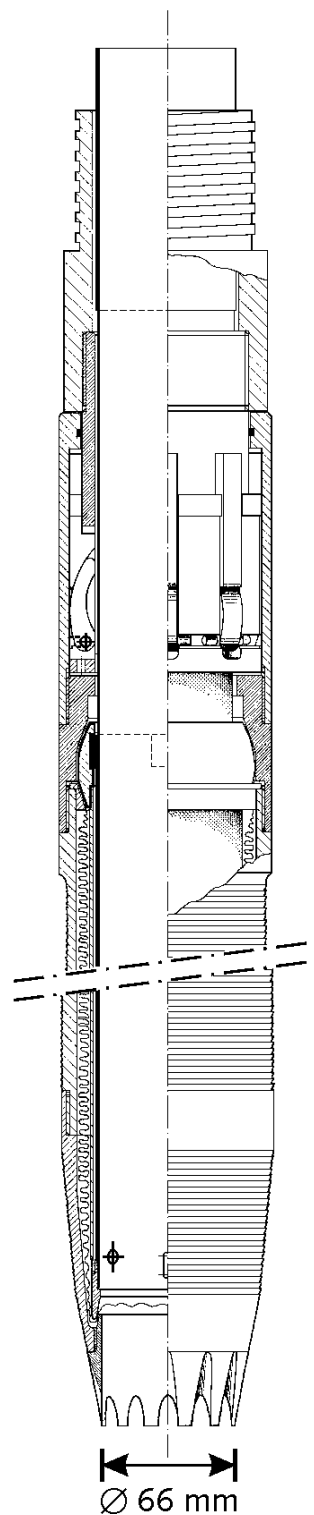


Figure 3.2 Continuous penetration apparatus

The small diameter is used to obtain an accurate determination of the soil profile, and in simple investigation tests to determine weight per unit volume. In addition to determining the soil profile, the 66 mm diameter is suitable for performing laboratory tests. As a result of the special sampling method, the 66 mm samples are of a high quality.

Figure 3.2 gives a cross-section of the 66 mm diameter continuous sampling device. Figure 3.3 is a photograph of the result of such drill core. An important advantage of this method is that a detailed picture can be obtained of the structure and composition of the soil. On the basis of visual assessment, it is also possible to make a well-considered choice as to which undisturbed samples should be used for determining mechanical properties.

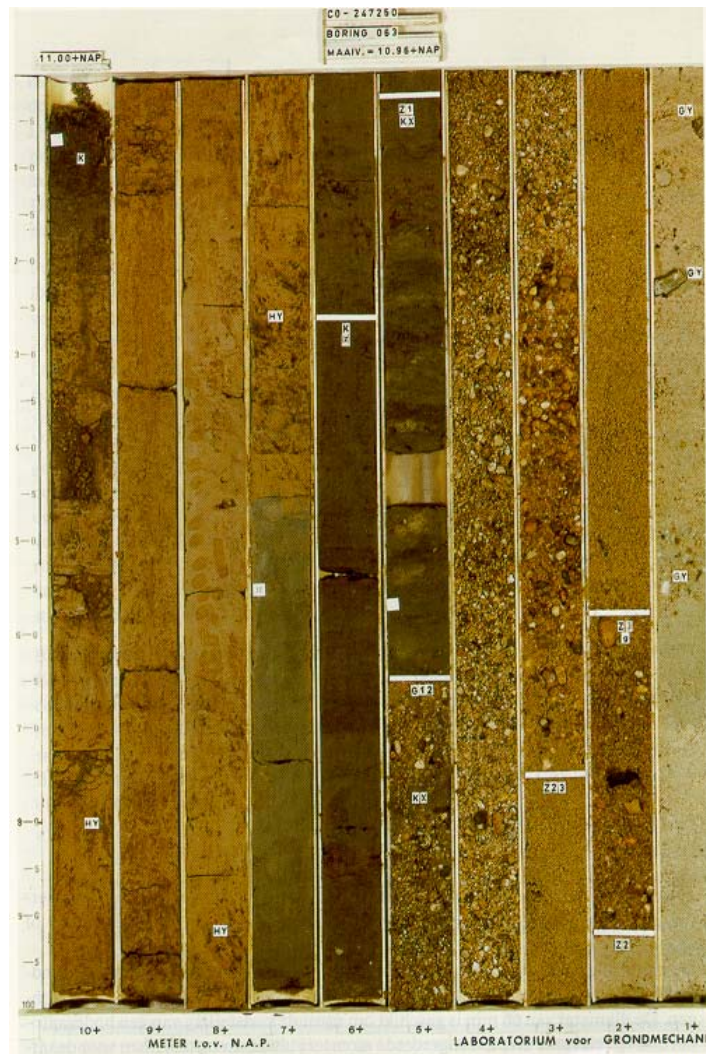


Figure 3.3 Core obtained by using continuous penetration apparatus

Annex 4 Vane tests

□

The vane consists of four thin, rectangular blades attached crosswise to the lower end of a thin steel rod (rotation rod). Figures 4.1 and 4.2 show the apparatus that is used for field measurements.

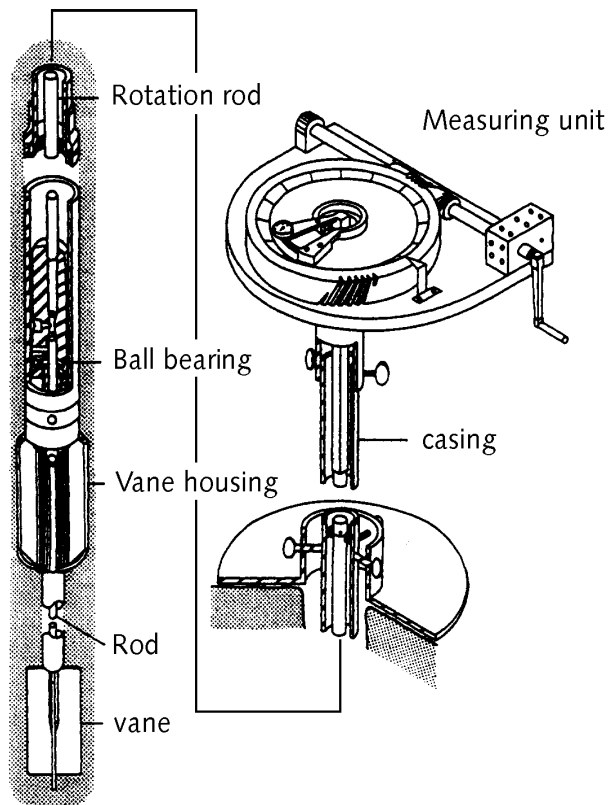


Figure 4.1 Geometry of the vane apparatus



Figure 4.2 Vane apparatus with casing

Usually the height (H) of the fin is equal to twice the diameter (D) of the sheathing cylinder. The size of the vane apparatus for field measurements varies between H = 160 mm with D = 80 mm for weak soil and H = 110 mm with D = 55 mm for firm soil. The speed of rotation of the vane is more or less standardised at 1.0°/s. For further information reference should be made to NEN 5106.

In the Netherlands the vane, which is often contained in a tubular sheath, is usually inserted into the soil with the aid of a probe. The vane, with the sleeve is lowered to the desired depth and then rotated. The sleeve ensures that the soil friction cannot affect the measurement of the rotation. The most modern measure both the turning moment and the angular displacement/rotation close to the fin, so that the friction of the rod is eliminated and an accurate measurement of the angular rotation is obtained. By alternately drilling deeper and performing a vane test a soil profile can be easily obtained that gives an indication of the undrained shear strength as a function of depth (Figure 4.3).

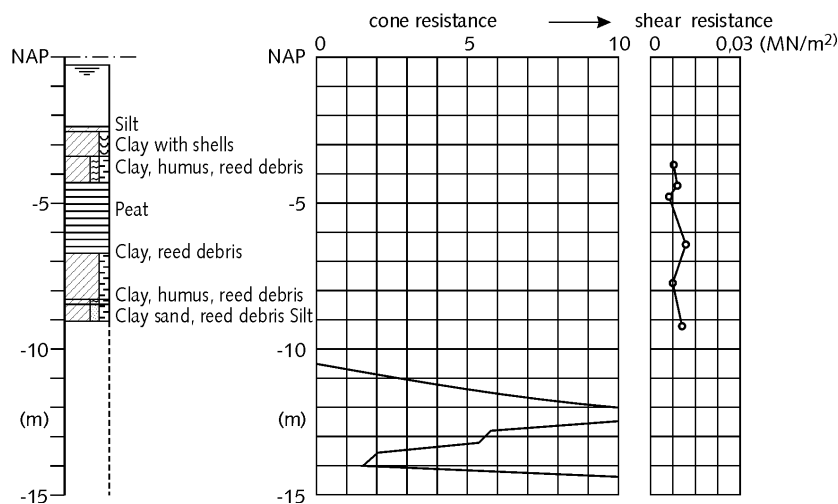


Figure 4.3 Development of undrained strength with depth

The undrained shear strength ($f_{undrained}$) is related to the maximum torsion moment (M) and the magnitude of the diameter (D) of the enclosing cylinder:

$$f_{undrained} = CM$$

$$f_{undr} = \frac{cM}{\pi D^2} \quad (4.1)$$

From the investigation into the determination of parameters (Ref. *Strain softening equations for pressure meter test analysis*) it appears that, in contrast to the more conservative constant $c = 0.86$ from NEN 5106, the undrained shear strength can be determined by using $c = 0.91$. From this research it also appears that it is preferable to measure the moment immediately above the fin blades, since the angular correction for the system for the ground level is in the order of magnitude of the angular displacement that is to be measured.

The above mentioned research has shown that for Dutch soils the vane apparatus is less suitable for determining the sliding modulus. The vane apparatus gives values for the sliding modulus that are a factor 3 to 5 lower than the values determined from by the CU triaxial test.

References:

NEN5106 Geotechniek. Bepaling van ongedraineerde schuifsterkte Terein-vinproef, NNI 1991 (Determination of undrained shear strength)

Strain softening equations for pressure meter test analysis, Fugro Ingenieursburo B.V. Report p-542 /02 December 1998

Annex 5 Field investigations using deformation parameter tests

Pressiometer

Using pressiometer tests, it is possible to measure the horizontal pressure-deformation behaviour of the strongly compressible layers in situ (Ref. *The Pressure meter*). With the various types of tests (such as the Ménard pressiometer, the self-drilling pressiometer and the Camkometer) and the aid of semi-empiric formulas, a horizontal elasticity module, and sometimes also the horizontal field (on-site) pressure can be determined. With some types of test, it is also possible to obtain indicative information on settlement behaviour. In comparison to other types of pressiometer, the self-drilling pressiometer provides qualitatively higher results while, on the basis of correlation, more parameters such as strength and consolidation parameters can be deduced. To date there is little experience with these tests in the Netherlands. In France and in other French-speaking nations, the pressiometer is used with great success.

Dilatometer

The dilatometer test is closely related to the pressiometer test. The dilatometer (see Figures 5.1 and 5.2) consists of a thin steel blade that is provided with a circular membrane on one side. This membrane can be expanded hydraulically. Measurements are taken at various depths, (usually at intervals of about 20 cm) and the pressure required to just move the membrane and to expand it membrane 1.1 mm is recorded. As in the pressiometer test, the horizontal pressures required to effect a particular horizontal movement in the soil are correlated to the horizontal elasticity modulus and the horizontal site pressure on the basis of semi-empiric formulas (Ref. *The flat dilatometer test and its applications*). In addition to these relations there are also correlations to determine strength and permeability parameters.



Figure 5.1 Marchetti dilatometer



Figure 5.2 Dilatometer blade

Cone pressure meter

The cone pressure meter (Figure 5.3) consists of a cylindrical membrane that expands somewhat cylindrically when inflated. The cone pressure meter is placed directly behind a

penetration cone. The penetration cone is pushed into the ground in the usual way; when the cone pressure meter is at the desired depth readings are taken by inflating the membrane. At this stage, a number of loaded/unloaded graphs are made (see Figure 5.4)



□□□□□□□□□□ □□□□□□□□□□□□□□□□

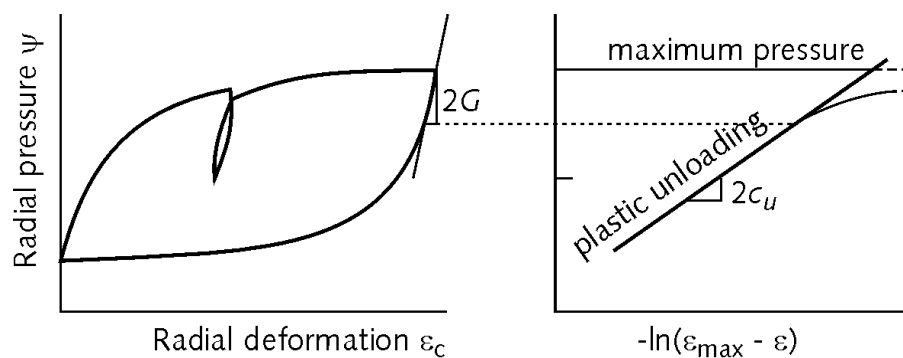


Figure 5.4 Cone pressure meter

References:
<http://WWW.marchetti-dmt.it>

Strain softening equations for pressure meter test analysis, Fugro Ingenieursburo B.V. Report p-542 /02 December 1998
 S. Marchetti, 1998. The flat dilatometer test (DMT) and its applications, Seminars in China
 Briaud, J.L. 1992. The pressure meter Balkema, Rotterdam

Annex 6 Compression test

NEN 5118 provides the rules for the implementation of the compression test. In the Netherlands, a stiff steel ring is placed around a cylindrical sample with a height of 2 cm and a diameter of 6.5 cm (See figure 6.1). The sample is enclosed above and below by means of porous plates, along which run-off is possible. The sample is loaded by means of a loading arm and weights. Usually, 5 load levels are applied; two for the pre-consolidation pressure and three after pre-consolidation pressure. Each load is applied for 24 hours, after which the load is doubled. It is also possible to implement a load-unload cycle. This test can be used to determine primary compression constants C_c and C_s . The secondary compression constant C_α is determined by ascertaining the compression development over time. The slope of the resulting curve can be used to determine the secondary compression constant (see Figure 6.2).

The compression test, or odometer test, is used to determine the compression constants and consolidation characteristics of the soil in the laboratory.

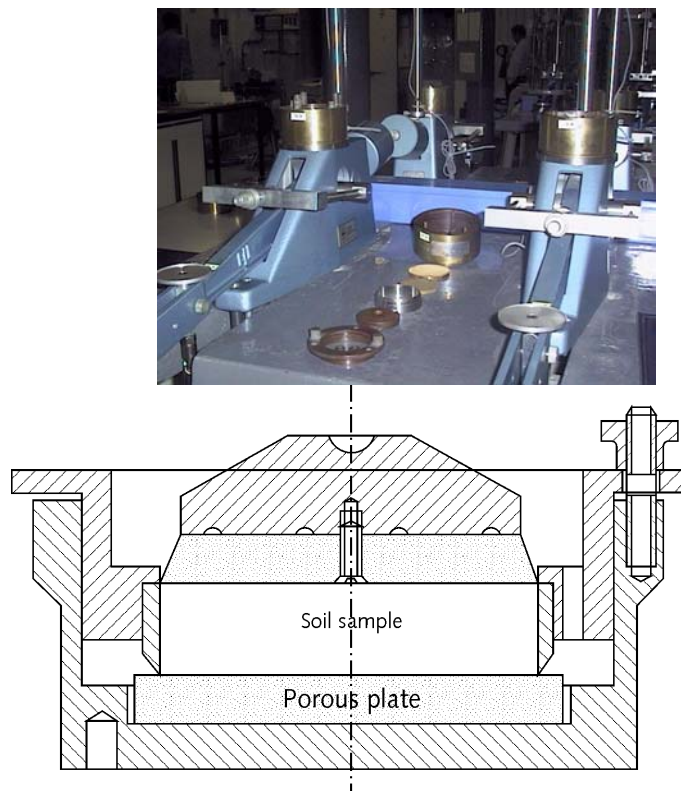
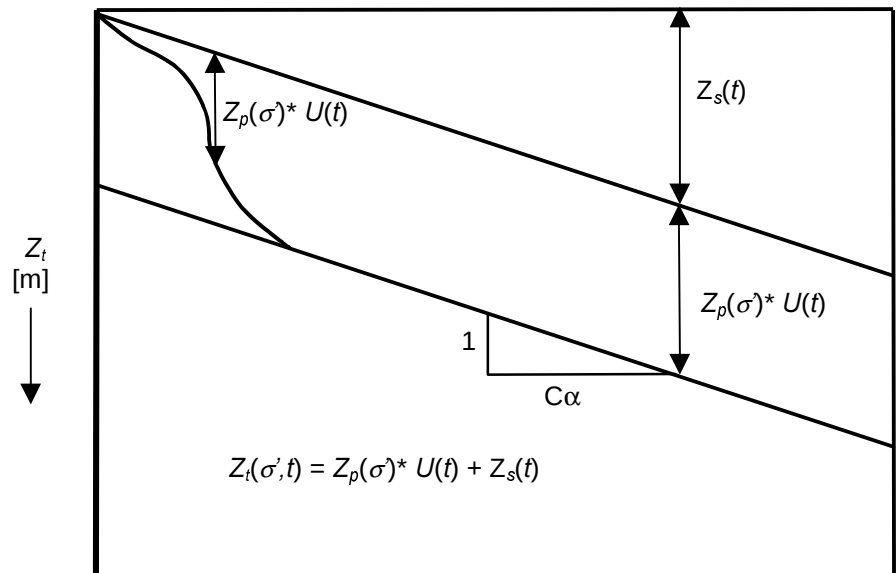


Figure 6.1 Compression test



where:

- t time (day)
- Z_t total consolidation (m)
- $Z_p(\sigma')$ primary consolidation acc. to NEN 6744 (m)
- $Z_s(t)$ secondary consolidation acc. to NEN 6744 (m)
- $U(t)$ consolidation grade (-)

Figure 6.2 Primary and secondary compression

Reference:

NEN5118, Geotechniek, Samendrukkingsproef

Annex 7 Direct shear test

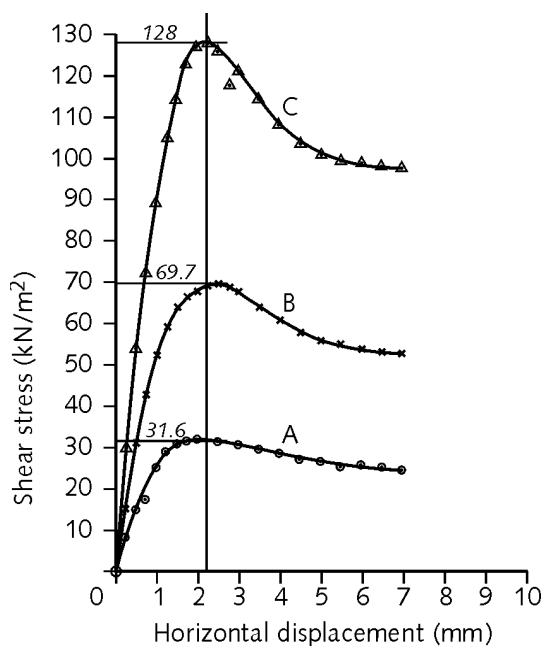
□

In principle the apparatus used for the shear test consists of a rectangular metal box composed of two parts (Figure 7.1). The sample to be tested is placed in the box. The test is performed by sliding the lower section of the box away from the upper section while at the same time a specific load is applied from above. Under this constant vertical load the relative displacement of the two parts of the sample is measured as well as the horizontal force needed. This test is repeated with vertical loads of varying magnitude. Figure 7.2 shows the development for three constant horizontal forces.



Figure 7.1 Set-up for the direct shear test

□
□
□
□
□
□
□
□



□
□
□
□
□
□

Monster	A	B	C
Normal stress (kN/m ²)	36	80	166
Shear stress (kN/m ²)	32	70	128
Displacement (mm)	2	2.5	2.2
Consolidation (mm)	16	15	14
Dry density (Mg/m ³)	1.70	1.70	1.71
Pore number (-)	0.56	0.56	0.55

□

Figure 7.2 Results of a direct shear test

Although the lower half of the box is moved horizontally with respect to the upper half, shearing does not occur along a horizontal plane. It is more likely that shearing will occur along other planes (where the shear stress reaches the critical value sooner). Since it is not possible to determine the stress along these planes in this test the horizontal plane is considered to be the shear plane, which leads to over estimation of the friction properties (Ref. *Grondmechanica*).

The most important disadvantages of the test are summarised below:

- The stress and deformation conditions in the sample are unequal. This is because the sample is more or less enclosed in the two halves of the apparatus and deformation can occur only in the central part.
- The water pressure cannot be accurately measured.
- There is uncertainty in the calculation of the vertical pressure and shear stress because during the test the surface area of the of the sample changes.
- Only a few pressures are known: the constant vertical load and the horizontal shear force.

The test is primarily used to determine the friction properties of sand and the undrained shear strength of clay. A drained shear test is seldom performed on a clay sample because of the long duration of the test. An exception is formed if the angle of friction is measured when there are very high cumulative deformations of clay samples (residual strength). This cannot be done in a triaxial test. This method is also used to test geotextiles and membranes.

Reference *Grondmechanica*, A Verruijt, Delft University Press 1999

Annex 8

DSS-test

□

In the Direct Simple Shear test (DSS-test) the soil sample is loaded with constant vertical forces. After consolidation of the sample (one dimensional) it is pushed aside in a way that permits deformation in one direction only. The DSS-test give a deformation state that is more homogeneous than that of the direct shear test (Ref. *Comparison of truly undrained and constant volume direct simple shear test*). Figure 8.1 shows the shear deformation for the undrained test (the volume of the sample is constant combined with horizontal shear force). This test is primarily used to determine the friction properties of sand and the undrained shear strength of clay.

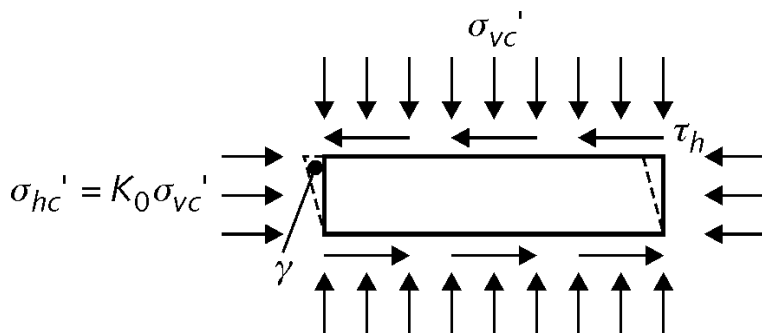


Figure 8 Stresses and deformation

DSS-apparatus

The DSS test is still undergoing development and as yet little used in practice. Various types of apparatus are in use (Ref. *Comparison of truly undrained and constant volume direct simple shear test*), see Figure 8.2):

- The apparatus developed in the University of Cambridge
- The apparatus of the Geotechnical Institute of Norway (NGI).

In both of these the sample is supported sideways so that shear deformations such as those shown in Figure 8.1 are possible, while the horizontal length of the sample remains the same.

- The Cambridge apparatus has hinged walls
- The NGI apparatus has a cylindrical wire reinforced rubber membrane.

NGI simple shear apparatus

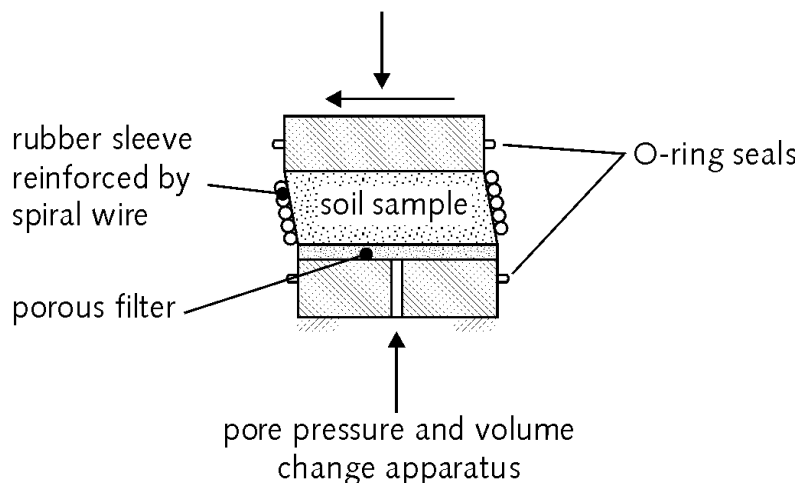
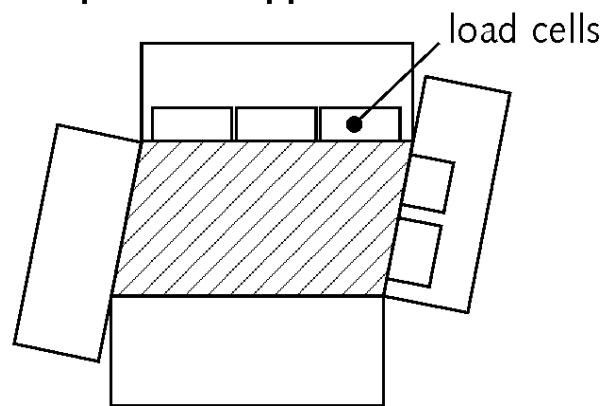


Figure 8.2 DSS apparatus

Cambridge simple shear apparatus



Critical shear plane

In the Direct Simple Shear test the critical shear plane never coincides with the horizontal shear plane. Moreover, with large shear deformation the angle between the critical shear plane and the horizontal shear is smaller (Ref. *Lateral stress measurements during static and cyclic direct simple shear testing*). The orientation of the critical shear plane depends on the entire stress situation of the soil sample. In the standard configuration of most shear strength instruments (as in the direct shear test, see Annex 7) the lateral stress condition is not measured. Only the shear stress ($T/A = \tau_h$) and the vertical normal stress ($N/A = \sigma_{vc}$) are measured. This means that the total stress condition of the sample is not known. Moreover it is assumed that the horizontal (or vertical) shear plane is the critical shear plane. From this the shear strength parameters are derived (c and ϕ).

Recent developments

One of the recent developments in the apparatus of the NHI is the measurement of lateral stresses in the sample (Ref. *Lateral stress measurements during static and cyclic direct simple shear testing*). The performance of these measurements during the shear test ensures that the total stress condition is known. In this case the results are in agreements with the triaxial test.

With most shear test equipment in the standard configuration it is impossible to perform a full undrained shear test in which the pore water pressure is measured (Ref. *Lateral stress measurements during static and cyclic direct simple shear testing*). Instead, shear tests in which the volume of the sample is kept constant are performed. Because the apparatus prevents lateral deformation of the sample it is only the height of the sample that has to be kept constant during the shear test.

References:

- Dyvik, J.R., Berre, T., Lacasse, S. and Br. Radim. 1988. *Lateral stress measurements during static and cyclic direct simple shear testing* Norwegian Geotechnical Institute, NGI number 170.
- Dyvik J.R., Zimmie T.F. 1983 *Comparison of truly undrained and constant volume direct simple shear test*, Norwegian Geotechnical Institute, NGI number 149

Annex 9: Definition of the Concepts

ACTUAL STRENGTH

Strength at the time of assessment

ASSESSMENT PROFILE

A theoretical minimum profile of defined dimensions within which no objects must be located and that must fit within the actual profile. This profile must guarantee that the flood defence is sufficiently strong.

BASIC REPORT

TAW Report containing background information for users of the TAW Guidelines.

BEDDING CONSTANT

A proportional coefficient between the counter pressure delivered by the foundation and the sagging of a surface under a vertical load (a spring constant expressing the stiffness of the subsoil).

BOERTIEN COMMISSION I

Committee that advises the Minister on the policy relating to the strengthening of river dikes.

BREACH

Gap or breach in dike

BY-LAW

A regulation of the Water Board (with a penalty for infringement).

BY-LAW ZONE

The zone within which the by-law is applicable.

CAPILLARY ZONE

The saturated zone of the soil above the phreatic plane.

COHESION

Reciprocal attraction between the fine soil particles of some soil types, by which they are maintained in a fixed mass without the application of external forces.

COLLAPSE

The occurrence of inadmissible large deformations of a structure, such that its cohesion is lost.

CONSOLIDATION

Expulsion of water from the pores of the grain skeleton of poorly permeable compressible soil under the influence of load increase as a result of which reduction of the volume will occur.

CONSOLIDATION COEFFICIENT

A roughly constant variable that determines the course of the consolidation process; the scale of this is determined by the permeability and compressibility of the soil and the volume weight of the water

CONSTRUCTION HEIGHT

Crown height of the dike immediately after completion. The height of the flood defence (parts of this), immediately after completion.

CRITICAL DENSITY)

The limit-value of the density of sand between loose and fixed packing.

CROWN

The highest part of the dike body.

DAMAGE FACTOR

Factor in which the results of collapse are involved.

DESIGN LEVEL

Extreme high water level with a stipulated probability of exceedence. Along the Dutch coast this varies between 1/2000 per year and 1/10,000 per year.

DIKE ENCLOSURE AREA

An area that must be protected from flooding by a system of flood defences, especially during high storm surges, high surface water of one of the major rivers, high water in IJsselmeer or a combination of these. This defence system consists of primary flood defences and high ground.

DIKE BASIS

Substratum directly under the dike or influenced by the presence of the dike

DIKE SECTION

Part of a dike with more or less equal strength properties and loading.

DIKE TABLE HEIGHT

The minimum stipulated crown height.

DUNE FOOT

The transition from beach to dune. The position of the dune foot in a cross-section. Defined by many managers with the aid of a contour that is constant in time (for example NAP +3m)

ELASTO-PLASTIC BEHAVIOUR

Phenomenon in which with increasing loading the soil first undergoes elastic deformation, which is then followed by plastic deformation.

EMBANKMENT

Small dike (kade)

EROSION DISTANCE

The distance over which the foreland or dike is damaged by the occurrence of settlement flow.

EROSION POOL

Deep erosion hollow directly in front of or behind a former gap in a dike. The line of the new dike is diverted round this deep pool.

EROSION PROFILE

The profile remaining after considerable erosion of a high foreland outside the dike.

ESTUARY

Wide river mouth.

FAILURE

No longer satisfying the stipulated criteria

FAILURE MECHANISM

The way in which the structure collapses (shearing piping).

FILTER

Intermediate layer in slope revetment that prevents the washing out of fine material from the subsoil through the upper layer of the revetment.

FIVE YEARLY INSPECTION

Periodic assessment of the safety and strength of a ring dike.

FLOOD DEFENCES

Artificial elevations and those parts of natural elevations or high grounds, including structures constructed in or alongside them that have a water retaining or partial water retaining function and that are indicated in the register as such.

FLOOD DEFENCE ZONE

Zone in the dune or dike area (and possibly inland of this) which is designated as a water retaining area and is managed by the Water Board (Management area. (Bylaw zone).

FLOODING

Inundation.

FLOW CHANNEL

Channel in the bed of a water body eroded by flowing water

FLOW LIMIT

Water content of the soil at the transition between the liquid-plastic state

FORE BANK

The bottom of the water body immediately beyond the toe of a dike.

FORELAND

The shallow water in front of the toe of a dike.

FREEBOARD

The actual height of a crown of a flood defence above a water level or the design levels.

GRADIENT OF SLOPE

Inclination or angle of slope in relation to the horizontal, expressed as a ratio in terms of height to horizontal distance

GRAIN DISTRIBUTION

Grain size distribution according to the size of the particles of a material, expressed in terms of mass

HEAVE

The creation of quicksand by vertical expulsion of groundwater

HIGH GROUNDS

Naturally higher parts of the Netherlands that are not inundated during normative high water. These are indicated on the dike enclosure map as the NAP + 1m line in the case of a threat from the IJsselmeer, the NAP + 2m line in the case of a threat from the sea or if there is a higher along the river, as the contour line corresponding to the normative high water level (NHW).

HIGH WATER RISE

Relative rise of the average high water level (including the NAP fall).

HYDRAULIC GRADIENT

Relation between the difference in height and the distance over which this occurs along the river.

HYDRAULIC RESISTANCE

Characterisation of the vertical permeability of poorly permeable layers of soil.

HYDRODYNAMIC PERIOD

The time needed to reduce the water pressure until it is almost zero (starting from the moment the load is introduced).

INFILTRATION

The penetration of water into the subsoil beneath a dike.

INFLUENCE ZONE

The area forming part of the defence works that actually contributes to the maintenance of stability both inside and outside the defence works (dike).

INNER BERM

Extra extension on the inner side of the dike to give the dike extra support.

INNER SLOPE

Sloping surface on the inner side of a dike

INNER TOE

The under edge of the dike on the inner side (the transition from the dike to the surface of the ground).

INUNDATION

Flooding of an area.

INUNDATION NORM

A legal safety norm indicated as the average probability (per year) of a flood due to the collapse of a primary flood defence.

KREUKELBERM

Horizontal part of the dike on the outer side as transition between the hard revetment and the rest of the slope of the forebank. Term used for transitional berms on sea or lake dikes.

LAKE DIKE

A primary flood defence generally situated along larger non-tidal water bodies other than rivers.

LEAKAGE FLOW

Flow of water under a structure

LEDGER

Document which contains a description of the conditions that flood defence must fulfil in terms of direction, shape, dimensions and construction, and in which the legal (by-law) limitations are given.

LIMIT PROFILE

The minimum profile that must be present as flood defense after erosion during design conditions

LINKING STRUCTURE

The entire longitudinal and transverse profile of an earth structure with a different shape at the junction between a dune, high ground or hydraulic structure.

LNC-VALUES

Landscape, natural and cultural inheritance values.

LOWER RIVER AREA

The catchment area of the rivers Rijn and Maas to the west of the line running between Schoonhoven – Werkendam –Dongemond and including the Hollandsche Diep and Haringvliet but not the Hollandsche IJssel.

LUTUM

Grain fraction consisting of mineral particles smaller than 0.002mm.

MACRO-STABILITY

Resistance to formation of a slip plane in the slope and subsoil.

MANAGEMENT

The whole of activities necessary to ensure that the functions of the flood defence continue to fulfil the (possibly time-dependent) requirements and norms determined for it.

MANAGER

In this guide: the government responsible for the management of the flood defence.

MANAGEMENT REGISTER or TECHNICAL MANAGEMENT REGISTER

Documentation in which the maintenance of data characterising the water retaining function of the structure and the actual state of the primary flood defence are described in detail.

MAIN DIKE, WINTER DIKE OR RIVER DIKE

The dike that encloses contains the winter bed of the river.

MATERIAL FACTORS

Partial factors that are applied to the parameters of the material to take into account uncertainties about the properties of the soil.

MHW

Mean high water level. Average high water ordinary tides.

MICRO-STABILITY

Resistance to erosion of slopes caused by the expulsion of water

MODIFIED DESIGNS

Dike profiles and technical constructions in the dike, the design of which differs from that of the standard the broad sand and clay structure.

MORPHOLOGY

The form of the features of the natural landscape

NAP

(Normaal Amsterdamse Peil) The height of the ground in relation to the average summer flood level of the River IJ at Amsterdam when this was still in communication with the North Sea.

NAP FALL

The fall in the NAP level as a result of movements in the earth's crust. Owing to the lack of a measurable reference point (midpoint of the earth), this fall can be quantified only in combination with the sea level rise.

NORMATIVE HIGH WATER LEVEL (NHW, XXXX, WHERE XXXX IS THE YEAR IN WHICH THE NHW HAS BEEN DETERMINED)

The normative high water level accompanying the area frequency laid down in Article 3.2 of the Flood Defences Act; this water level is included in the book of boundary conditions and if necessary is amended after the five-yearly inspection of the defences

OUTER SLOPE

The sloping surface of the dike on the river/sea side/lake side

OUTSIDE WATER

The surface water, the water level of which is directly affected in the case of high storm tide, high surface water of the major rivers, high water on IJsselmeer or in the case of a combination of these.

OVERFLOW

The phenomenon in which the water flows over the crown of the dike into the hinterland because the water level in the river is higher than the crown,

OVER HEIGHT

Extra amount of soil that is applied to ensure that the desired profile is reached after the settling of the subsoil, (supplementary freeboard).

OVERLOADING

Overloading occurs when the applicable loading- criterion is exceeded.

PHREATIC PLANE

Free groundwater table.

PIEZOMETRIC GAUGE

A water level gauge; a tube placed in the ground to measure the groundwater level of hydraulic head.

PIPING

The phenomenon in which under a dike a hollow pipe-like space is created because the erosion process of a sand carrying spring continues.

PLAN PERIOD

Period (for dikes usually 50 years) in which the anticipated changes in conditions are included in the design of the dike.

PLASBERM

Horizontal part of the dike on the outer side as transition between the hard revetment and the rest of the slope of the forebank. Term used for transitional berms on river dikes.

PLASTIC LIMIT

Water content of the soil at the boundary between plastic to semi-solid condition.

PLASTICITY INDEX

Difference in the liquid content of cohesive soil between the liquid limit (transition from liquid to plastic state) and plastic limit (transition from plastic to semi –solid state).

POLDER DIKES

Dikes on each side of a waterway that is used to carry away surplus water from polder. The water level in this waterway is usually higher than the polder level,

POLDER LEVEL

Level of the surface water within a management area.

POLICY

The total number of management decisions.

PORE CONTENT

The percentage of the volume of the soil taken up by pores.

PRIMARY FLOOD DEFENCE

A flood defence that provides protection against flooding by either forming part of the system enclosing a dike enclosure area, being situated in front of an area enclosed by a dike area.

PRIMARY SETTLEMENT

Vertical deformation of soil layers caused by an increase in the effective pressure.

PROBABILITY OF EXCEEDENCE

Average number of times that a phenomenon reaches or exceeds a specific value.

PROTECTION ZONE

Zone indicated in the by-law on both sides of the (legally defined) flood defence.

RESPONSIBLE AUTHORITY

The government body that is authorised to take the environmental impact report decision. For dike strengthening this is the province

RING DIKE

A dike enclosing an area that has been drained (polder).

RIVER DIKE

A primary water defence, usually along a major river

SAND CARRYING SPRING

Erosion phenomenon in which out-flowing seepage water washes out sand.

SCHAAR DIKE

A dike immediately adjacent to a river.

SCOUR

Erosion of the soil at the toe of a dike resulting from flowing water or wave action.

SEA DIKE

A primary water defence of class 1 that keeps protects against salt water.

SCOUR HOLE)

A deep erosion hollow directly in front of or behind a former gap in a dike. The line of the new dike is diverted round this deep pool.

SECONDARY SETTLEMENT

Continuing vertical deformation of soil layers with constant effective pressure as a function of time: creep.

SEEPAGE

The expulsion of groundwater under the influence of a difference in water level over a defence.

SEEPAGE EMBANKMENT

An low embankment (about 50 cm high) that retains the seepage water.

SEEPAGE DISTANCE

The distance that the seepage water travels below ground before it emerges at the surface.

SEEPAGE SCREEN

An impermeable, usually vertical construction to extend the seepage distance

SEICHE

Resonance phenomenon in basins (including harbours) as a result of low frequency variations in the outside water level. The phenomenon is also known as harbour oscillation.

SETTLEMENT

Decrease in volume of soil, chiefly due to an upper load, own mass and/or the exit of water.

SETTLEMENT FLOW

Flow occurring when as a result of the loss of effective pressure, a saturated sand mass behaves as a liquid. The loss of effective stress.

SHEARING

The movement of part of a soil body due to exceedence of the equilibrium capacity.

SOFTENING

Loss of cohesion of a grain skeleton as a result of an increase in the pore water pressure.

SPRING

Concentrated outflow of seepage water through a hole or along a pile in the covering layer.

SQUEEZING

Sudden occurrence of big horizontal movements in the subsoil under the dike, at an angle to the axis of the dike (ice-cream wafer effect).

STRUCTURE UNCONNECTED WITH WATER RETENTION

Structure unconnected with water retention

SUPPLEMENTARY HEIGHT ALLOWANCE

The actual height of a crown of a flood defence above a water level or the design levels.

TRANSITIONAL BERM

Horizontal part of the dike on the outer side as transition between the hard revetment and the rest of the slope of the forebank.

UPPER RIVER AREA

The catchment area of the rivers Rijn and Maas to the east of the line running between Schoonhoven – Werkendam – Dongemond. In this area water levels are not influenced by the tides of the North Sea.

UPLIFT

Collapse of the ground caused by the lack of vertical equilibrium in the ground under the influence of water overpressure.

WATER LEVEL NORM

The legal safety norm indicated as average exceedence probability – annually – of the highest high water level on which the primary flood defence for the direct retention of water must be calculated, also taking into account of the other factors determining water retaining capacity.

WATER PRESSURE

The pressure in the groundwater.

WAVE RUN-UP

The height above the water level reached by a wave running up the slope. (The 2% wave run-up is exceeded by 2% of the waves.)

WAVE OVERTOPPING

The average amount of water that is thrown over the defence per running metre per tide.

WAVE OVERTOPPING

The phenomenon in which water flows over the top of the dike and reaches the hinterland. The water level is lower than the crown but the water is swept up by waves as a result of wind or shipping movements.

WIND GUST RISE

Individual very short-term rise in water level as a consequence of a heavy wind.

WIND OSCILLATIONS

Irregular fluctuations in the water level with varying periods that occur particularly during heavy storms.

WINTER BED

Transverse profile of the river between the summer bed and the dike. Water meadows.

Annex 10

Nomenclature

Symbol	Dimension	Definition
A_{ont}	m^3/m	Retained sand volume per m
b_f	m	Thickness filter layer
b_k	m	Thickness clay layer
B	m	Width berm
c	Pa	Cohesion soil
c'	Pa	Effective cohesion soil
C_c	-	Primary-compression index
C_{creep}	-	Material constant for piping
$C_{w,creep}$	-	Material constant for piping
C_α	-	Secondary-compressions index
C_s	-	Swelling constant
c_u	Nm^{-2}	Undrained shear strength
C_E	m-1s-1	Grass erosion coefficient
c_v	m^2/s	Consolidation coefficient
d	m	Layer thickness of reference-erosion depth or thickness of mattress/gabion
d_s	m	Hydraulic head parameter
d_{70}	m	70-percentile values of the grain distribution
D	m	Thickness water bearing series/aquifer or characteristic size of revetment element
D_r	-	Relative density $= \frac{(e_{max} - e)}{(e_{max} - e_{min})}$
e	-	Pore number
e_o	-	Pore number at initial effective pressure
E	kNm^{-2}	Elasticity modulus
f_{undr}	Nm^{-2}	Undrained shear strength

F	N	General indication of force
g	ms^{-2}	Acceleration of gravity
ATA	m	Average tidal amplitude
AHW	m	Average high water
ALW	m	Average low water
AWL	m	Average water level
h	m	Thickness of the soil layer or depth of water at the toe of the dike or ground water level in relation to reference level
H_C	m	Height of crown above still water level
h_{op}	m	Storm surge
h_q	m	Run up height for $q = 0,1 \text{ l/m/s}$
ΔH	m	contribution of storm gust to maximum supplementary height
H	m	Wave height
H_s	m	Significant wave height at the toe of the dike (is exceeded by 13,6% of the waves)
i_{kr}	-	Critical hydraulic gradient
I_p	%	Plasticity index of clay
k	ms^{-1}	Permeability
kD	m^2s^{-1}	permeability of an aquifer
k_h	ms^{-1}	Horizontal permeability of soil layer
k_v	ms^{-1}	Vertical permeability of soil layer
L	m	Seepage distance or width of foreland
L_s	m	erosion distance
m	-	Exponent in the Hardening Soil model, assume the stiffness to be dependent on the potential stress level
m_v	m^2/kN	Vertical volume compression coefficient
M_x	kg	Stone mass for which x% of the stones have a greater mass
NHW xxxx	m	Normative high water determined for a specific year.
NCH	m	Normative crown height

n	-	Safety factor or pore content
n_f	-	Pore content of filter layer
n_k	-	Pore content of clay layer
n_r	-	Relative density $= \frac{(n_{\max} - n)}{(n_{\max} - n_{\min})}$
O	m^2	Surface area
q	$m^3/m/s$	Overtopping rate per m
q_t	$m^3/m/s$	Permissible overtopping rate per m
r	m	Extra erosion behind the back of the structure
R	kNm	Average strength on a shear surface
S	kNm	Driving moment
S_{op}	-	H_s/L_{op}
TL	m	Target level
S	div.	Strength parameter
S_w	-	Degree of saturation
SWL	-	Slack water line
t_k	hour	Time spent in impact zone
$t_{N-1}^{0,95}$	-	Student t-factor
t_s	uur	Normative storm duration
T	s	Wave period or shear resistance (N)
U	Nm^{-2}	Pore water pressure or degree of consolidation (-)
y_d	m	Height of breach
y_m	m	Maximum water depth in de current channel
Z	m	High toed slope in relation to reference level
zx%	m	Wave run-up height above SWL, that is exceeded by x% van de incoming waves
z_q	m	Wave run up height for $q = 0,1$ l/m/s

Z_k	%	Sand content = mass percentage of grains > 63 μm
Z_t	m	Total settlement
W	N	Pore water pressure
α	°	softening angle or angle of the slope with the horizontal
β	°	Angle of slope with the horizontal or angle between the wave direction and the de normal on the dike
γ	-	Safety factor or reduction factor wave run up/ wave overtopping
γ_0	-	Stability factor on the basis of average values
γ_d	-	Safety factor relating to the calculation model used (sensitivity factor)
γ_k	kN/m^3	Volumetric weight of the granular material
γ_m	-	Safety factor relating to material parameters
γ_n	-	Safety factor in relation to damage
γ_{wet}	kN/m^3	Wet volumetric weight
γ_R	-	Safety factor for the strength
γ_S	-	Safety factor for the loading
γ_w	kN/m^3	Volumetric weight of water
γ_1	°	Side slope near soil structure
γ_2	°	Angle of slope after settlement
δ	m	Penetration length or correction factor calculation value of wave height
σ_b	kNm^{-2}	Weight of under water revetment
σ_{av}	Nm^{-2}	Average effective pressure
σ_h	Nm^{-2}	Horizontal effective pressure
σ_v	Nm^{-2}	Vertical effective pressure
σ_n'	Nm^{-2}	Effective pressure perpendicular to the surface under consideration

σ'	Nm ⁻²	Effective pressure
σ'_0	Nm ⁻²	Initial effective pressure
τ'_{cr}	Nm ⁻²	Critical shear pressure or shear strength
τ'_m	Nm ⁻²	Drained shear strength
Δ	-	Relative volumetric mass of the top layer
Δ_f	-	Relative volumetric mass of the filter layer
Δ_k	-	Relative volumetric mass of the clay layer
ΔH	m	Gradient over the structure
$\Delta H_{strength}$	m	Critical gradient of the structure
Δh_p	m	Primary compression
Δh_s	m	Secondary compression
Δ	-	Relative density
Δt	day	Duration of the loading
$\Delta \sigma$	Nm ⁻²	Increase in effective pressure
ρ_g	kg/m ³	Volumetric mass of soil
ρ_s	kg/m ³	Volumetric mass of revetment elements
ρ_w	kg/m ³	Volumetric mass of water
ϕ	°	Angle of internal friction
ϕ'	°	Effective angle of internal friction
ϕ	m	Hydraulic head
ϕ_z	m	Hydraulic head directly under the covering layer
ν	m ² s ⁻¹	Kinematic viscosity
$\mathcal{E}_{vol}$	%	Maximum reduction of volume
ψ	°	Dilatation angle
ν	-	Transverse contraction coefficient

Bibliography

- Analysis of the duration of extreme high water levels of the Markermeer. (Analyse duur van extreme meerpeiltoppen Markermeer.* N.N. Lorenz, WL | Delft Hydraulics, Delft, September 2000)
- Applied Geophysics, Code of Practice,* BRGM, CGG, LCPC, Ministere de l'industrie sous direction Qualité-Normalisation République Française, March 1992
- Assessment methodology for renovation alternative for block revetments Zeeland. (Afwegingsmethodiek renovatie-alternatieven blokkenbekledingen Zeeland.* Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, concept, November, 1996)
- Background to numeric modelling of geotechnical constructions Part 1. (Achtergronden bij numerieke modellering van geotechnische constructies deel 1,* CUR-publication 178, Gouda, 1995)
- Background to numeric modelling of geotechnical constructions Part 2. (Achtergronden bij numerieke modellering van geotechnische constructies deel 2.* CUR-publicatie 191, Gouda, 1997)
- Bank protection materials. (Oeverbeschermingsmaterialen,* CUR-publicatie 168 en 168a, Gouda, 1994)
- Basic set-up of the Technical Report on Water pressure. (Basisopzet Technisch Rapport Waterspanningen.* Fugro Ingenieursbureau B.V., June 2000)
- Cases, nature and extent of field and laboratory research. (Cases aard en omvang terrein- en laboratoriumonderzoek.* GeoDelft CO-319830/20 July 19910
- Cement-bentonite screens. (Cement-bentonietschermen.* CUR-publicatie 189, Gouda, 1997)
- Chance in Civil engineering, Part 1: Probabilistic design in theory. (Kansen in de civiele techniek, Deel 1: Probabilistisch ontwerp in theorie.* CUR/Rijkswaterstaat, March 1997)
- Characteristic values for shear strength in relation to stability analyses. (Karakteristieke waarden voor schuifsterkte t.b.v. Stabiliteitsanalyses.* E.O.F. Calle, GeoDelft CO-350000/43, July 1996)
- Comparison of truly undrained and constant volume direct simple shear test.* J R. Dyvik, T. Berre, S. Lacasse, Br. Raadim, Norwegian Geotechnical Institute, NGI-nummer 170, 1988
- Expropriation for dike reinforcement (Onteigening voor dijkversterking,* Ministerie van Verkeer en Waterstaat, 1996)
- Conceptual design of rubble mound breakwaters.* J.W. van der Meer, Waterloopkundig Laboratorium, publicatie 483, Delft, 1993
- Constructing with soil: Earth-structures on and in sub-soils with little bearing capacity and with highly compressed sub-soils. (Construeren met grond, Grondconstructies op en in weinig draagkrachtige en sterk samendrukbare ondergrond.* CUR-publicatie 162, Gouda, 1993). English version *Soils. Design and construction of earthstructures both on and into highly compressible subsoils of low bearing capacity.* (Balkema)
- Dams barrage and weirs on porous foundations.* W. G. Bligh, Engineering News, 1910
- Delta Commission Reports 1, 4 en 6 (Deltacommissie, rapport 1, 4 en 6.* Staatsdrukkerij en Uitgeversbedrijf, Den Haag, December 1960)
- Determination of the undrained shear strength with field vane test. (NEN 5106, Geotechniek.* Bepaling van ongedraineerde schuifsterkte. Terrein-vinproef, NNI, 1991)
- Dikes and revetments.* K. W. Pilarczyk (editor), Balkema, Rotterdam, 19980

End constructions for dikes and dune foot defences in Zeeland. (Eindconstructies van dijken en duinvoetverdediging in Zeeland. Rijkswaterstaat, Rijksinstituut voor Kust en Zee, P. Roelse, Werkdocument GWW-93.843X, Middelburg, June 1993)

Fundamentals of water retaining structures. (Grondslagen voor waterkeren. Technische Adviescommissie voor Waterkeringen, Uitgeverij Balkema, 1998)

General principles on reliability for structural design. International Association for Bridge and Structural Engineering, 1981

Geophysical techniques for soil investigation. (Geofysische technieken voor grondonderzoek. CUR-publicatie 182, Gouda0

Geo-synthetic fabrics and river dike improvement. (Geokunststoffen en rivierdijkverbetering. CUR-publicatie 186, Gouda, 1996)

Geotechnical classification of soft samples. (NEN 5104, Geotechniek. Classificatie van onverharde monsters, NNI, 1989

Geotechnical compression test. (NEN 5118, Geotechniek. Samendrukkingsproef, NNI, 1991)

Geotechnical determination of the shear resistance. (NEN 5117, Geotechniek. Bepaling van de schuifweerstand van grond. Triaxiaalproef, NNI, 1991)

Geotechnical determination of the volumetric mass of solid soil particles. (NEN 5111, Geotechniek, Bepaling van de volumieke massa van vaste (grond) delen – Pyknometermethode, NNI, 1995)

Geotechnical laboratory determination of permeability to water with the aid of the “Constant head” method. (NEN 5123, Geotechniek. Laboratoriumbepaling van de waterdoorlatendheid van grond met behulp van de “Constant-head” methode, NNI, 1997)

Geotechnical laboratory determination of permeability to water with the aid of the “Falling head” method. /NEN 5124, Geotechniek. Laboratoriumbepaling van de waterdoorlatendheid van grond met behulp van de “Falling-head” methode, NNI, 1997

Geotextiles in Hydraulic engineering. /Geotextielen in de waterbouw, CUR-publicatie 174, Gouda, 1995

grondslagen voor bouwconstructies, TGB 1990, Algemene basiseisen, NNI, Delft, April 1991

Guidelines on soil investigation in and close to of water retaining structures. /Leidraad bij bodemonderzoek in en nabij waterkeringen. Uitgeverij Waltman, Delft, May 1988

Guidelines on choice methodology for dike and bank revetments. /Leidraad Keuzemethodiek dijk- en oeverbekledingen, TAW, VNC, VBW-ASFALT, Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, September 1988

Guidelines on micro stability/Leidraad Toetsing, Microstabiliteit, H. den Adel, GeoDelft CO-333600/76, 1996

Guidelines for sandy coasts. /Leidraad Zandige Kust. Technische Adviescommissie voor de Waterkeringen, 1995

Guidelines on sea and lake dikes: Basic report. /Leidraad Zee- en Meerdijken Basisrapport. Technische Adviescommissie voor Waterkeringen, 1999

Guidelines on safety monitoring. /Leidraad Toetsen op Veiligheid. Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, 1999

Guidelines for the design of river dikes: Part 1 – Upper River area. (Leidraad voor het ontwerpen van rivierdijken; deel 1 – bovenrivierengebied, Uitgeverij Waltman, Delft, September 1989)

Guidelines for the design of river dikes: Part 2 – Lower River area. /Leidraad voor het ontwerpen van rivierdijken; deel 2 – benedenrivierengebied, Uitgeverij Waltman, Delft, September 1989

Guidelines the design of river dikes: Part 1 – Upper River area. /Leidraad voor het ontwerpen van rivierdijken: deel 1 – bovenrivierengebied. Staatsuitgeverij, 's Gravenhage, September 1985

Handbook for nature-friendly banks. /Handboek voor natuurvriendelijke oevers. CUR/Rijkswaterstaat, DWW, 1994

Handbook for the dimensioning of placed slope revetments/ Handboek voor dimensionering van gezette taludbekledingen s. CUR, commissie C74, Gouda, 1991

Handbook on settlement flow. /Handboek Zettingsvloeiingen. GeoDelft CO-353260/10, 1994

Hydraulic boundary conditions for primary water retaining structures. /Hydraulische randvoorwaarden voor primaire waterkeringen. Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, July 1996

Hydraulic boundary conditions for the IJsselmeer and Markermeer. /Hydraulische randvoorwaarden IJsselmeer en Markermeer. Rijkswaterstaat, RIZA, 1999

Interaction Water Motion and Closing Elements. K. W. Pilarczyk in *The Closing of Tidal basins*, Delft University Press, 1984

Inventory of techniques for and RWS-experience of the construction of a digital field model in a river area. /Inventarisatie van technieken voor en RWS-ervaringen bij het vervaardigen van een digitaal terrein model in een rivierengebied, Fugro Ingenieursbureau B.V. Rapport N-0481, December 1997

IPO-guideline for determining the safety level of polder embankments. /IPO-richtlijn ter bepaling van het veiligheidsniveau van boezemkaden. IPO Adviescommissie MWLN, March 2000

Lateral stress measurements during static and cyclic direct simple shear testing. R. Dyvik, T.F. Zimmie, Norwegian Geotechnical Institute, NGI-nummer 149, 1983

Loads and deformations. NEN 6702, TGB 1990, Belastingen en vervormingen, NNI, December 1991

Structural design manual. (/Handeiking Constructief Ontwerpen. Technische Adviescommissie voor de Waterkeringen, 1994

Manual on the use of Rock in Hydraulic Engineering, section 6.3, CUR-publicatie 169, Gouda 1995)

Memorandum on the resistance against shearing of a stone revetment. (Notitie weerstand tegen afschuiven van een steenbekleding. Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, concept February 2000)

Micro stability: Overview of modelling. /Microstabiliteit: overzicht modellering. GeoDelft CO-393440/81, August 2000

Modern technology for dikes. /Moderne technologie voor dijken, Cursus PAO, D16 Beleidsanalyse dijkversterking Sliedrecht, W. Korf, Delft, 1988

Moderne technologie voor dijken, Cursus PAO, D4-Bezwijk- en faalmechanismen, M. B. de Groot, Delft, 1988

Nature –friendly banks. /Natuurvriendelijke oevers CUR-publicatie 168, Gouda, 1994

NEN 6744

On the mechanism of piping under impervious structures, J. B. Sellmeijer, proefschrift, TU Delft, 1989

Open slope revetment Part D, revised Black Box model Open taludbekledingen, deel D, vernieuwd Black Box model, Waterloopkundig Laboratorium, verslag A2.94.26, April 1996

Orientating study into soil mechanical aspects of erosion holes/ Oriënterende studie naar grondmechanische aspecten bij ontgrondingskuilen, F. Silvis, GeoDelft CO-291720/12, 1988

PIANO User Manual, Fugro Ingenieursbureau B.V., December 1998

Piping due to flow towards ditches and holes, J. B. Sellmeijer, Proc. Euromech. 143, Delft, pp. 69-72, 1981

Plaxis guidelines. /Handleiding Plaxis. Balkema Rotterdam, 1998

Proba Phase AIII, Part 2, Probabilism in geotechnology, spatial variability, guideline on length-effects/*Proba fase AIII, onderdeel 2*, Probabilisme in de geotechniek, ruimtelijke variabiliteit, richtlijn lengte-effecten, Rapport GeoDelft, January 1998

Proba Phase AIII: Part 1, Probabilism in geotechnology, spatial variability, conceptual model, *Proba fase AIII: onderdeel 1*, Probabilisme in de geotechniek, ruimtelijke variabiliteit, conceptueel model, Rapport GeoDelft en Fugro, CO-361410.95, December 1997

Probabilistic design/Probabilistisch ontwerpen, Dictaat b3, A. C. W. M. Vrouwenvelder en J. K. Vrijling, TU Delft

Regulations for the design of steel bridges./NEN 1008, 6788, VOSB 1963, Voorschriften voor het ontwerpen van stalen bruggen, NNI, 1963, 1991

Requirements for steel transport pipes. NEN3650, Eisen voor stalen transportleidingen, NNI, 1992

Research report on the determination of the actual strength of river dikes/Onderzoeksrapport voor de bepaling van de actuele sterkte van rivierdijken Onderzoeksrapport voor de bepaling van de actuele sterkte van rivierdijken, Technische Adviescommissie voor de Waterkeringen, June 1996

Residual strength of dike revetments, Part III, Stability of placed stone and underlying clay layer/Reststerkte van dijkbekledingen, Deel III, stabiliteit van steenzetting en klei-onderlaag, Waterloopkundig Laboratorium en GeoDelft, verslag A2.93.27, June 1993

Results of the preliminary investigation of Zeeuwse currents/Resultaten van het vooronderzoek langs Zeeuwse stromen, Rapport 75.2, Rijkswaterstaat, Directie Waterhuishouding en Waterbeweging, Studiedienst Vlissingen, 1979

Scour manual, Balkema, Rotterdam, 1997

Security from under-seepage masonry dams on earth foundations, E. W. Lane, ASCE-proceedings, 1919

Sheet pile constructions. /Damwandconstructies. CUR-publicatie 166, Gouda, 1994

Soil investigation using the static penetration test method. /s NEN 3680, Grondonderzoek. Statische sondeermethoden, NNI, 1982

Soil Mechanics. /Grondmechanica. A. Verruijt, Delft University Press, 1999

Standard RAW-Regulations/Standaard RAW-bepalingen, CROW, 2000

Statistical processing of cell and triaxial tests/ Statistische verwerking van cel- en triaxiaalproeven, Rapport GeoDelft, SE-50676, February 1997

Summer boundary conditions/Zomerrandvoorwaarden, Rijkswaterstaat, RIKZ, IT.95.180, 1998

Supplementary regulations for non- steel pipes in crossings with important hydraulic structures. /NEN 3652, Aanvullende eisen voor niet-stalen leidingen in kruisingen met belangrijke waterstaatkundige werken, NNI, 1998

Supplementary regulations for steel pipes in crossings with important hydraulic structures NEN 3651, Aanvullende eisen voor stalen leidingen in kruisingen met belangrijke waterstaatkundige werken, NNI, 1994

Technical basis for building constructions. / NEN 6700, Technische

Technical report on the monitoring of polder embankments/dikes/ Technisch Rapport voor het toetsen van boezemkaden, Technische Adviescommissie voor de Waterkeringen, June 1993

Technical report on piping in river dikes/Technisch Rapport Piping Rivierdijken, TAW-B richtlijn, E. O. F. Calle en J. B. Weijers, 1994

Technical report on demands for clay for dikes/ Technisch Rapport Eisen Klei voor Dijken
 Technische Adviescommissie voor Waterkeringen, Groene versie, November 1994

Technical report on sand bearing springs/Technisch Rapport Zandmeevoerende wellen,
 Technische Adviescommissie voor Waterkeringen, 1999

*Technical report on wave run-up and overtopping of dikes/ Technisch Rapport Golfoploop en
 overslag bij dijken,* Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, 1999

Testing linking structure/Toetsing aansluitingsconstructies, Waterloopkundig Laboratorium,
 H. J. Steetzel, Delft, 1995

The interaction between soil, water and bed or slope protection, M. B. de Groot, Proc.
 Modelling Soil-Water Interactions, Balkema, Rotterdam, 1988

Theoretical developments relating to uplift/Theorieontwikkeling rondom opdrijven, M.A. Van
 en A. van Hoven, SE-52029/2, February 2000

Validation MStab lift/Validatie MStab lift , SE-50020/2, M.A. Van en A. van Hoven ,
 September 2000

Vertical drainage/Verticale drainage CUR publicatie C68/2-03, Gouda 1990

Water retaining structures Act/Wet op de waterkering Staatsblad 1996, 8, SDU uitgeverij, 's
 Gravenhage, 1996

Water retaining structures, Lecture notes for higher education/ *Waterkeringen,* Dictaat voor
 het Hoger Onderwijs, Rijkswaterstaat, Dienst Weg- en Waterbouwkunde, 1993

*Wave overtopping and forces on vertical water retaining structures. Golfoverslag en
 krachten op verticale waterkeringsconstructies,* Waterloopkundig Laboratorium, Rapport
 H2014, 1998

*Wind turbine foundations, safety assessment for the location of wind turbines on dikes and
 dams/Windturbinefunderingen, veiligheidsbeoordeling van plaatsing van windturbines
 op dijken en dammen,* Fugro Ingenieursbureau B.V., June 1989

*Windmills and water retaining structure, Technical requirements for the placing of windmills
 on or close to dikes/Windmolens en Waterkeringen, Technische eisen voor het plaatsen
 van van windmolens op of nabij waterkeringen,* Rijkswaterstaat, Dienst Weg- en
 Waterbouwkunde, November 2000

Bank protection materials. (Oeverbeschermingsmaterialen, CUR-publicatie 168 en 168a,
 Gouda, 1994)
 Deze publicatie is vervangen door CUR/RWS uitgave 202 Natuurvriendelijke oevers.
 Oeverbeschermingsmaterialen (Nature friendly banks. Shore protection materials, uitgave
 1990.)

Risk analysis in civil engineering, Part 1: Probabilistic design in
 theory. Publ. 190 (Kansen in de civiele techniek, Deel 1: Probabilistisch ontwerpen in
 theorie. CUR/Rijkswaterstaat, March 1997)

Handbook for nature-friendly banks. (Handboek voor natuurvriendelijke oevers.
 CUR/Rijkswaterstaat, DWW, 1994)
 Nature friendly banks. (Natuurvriendelijke oevers CUR-publicatie 168, Gouda, 1994)

Deze is dus vervallen en vervangen door een serie van 6 rapporten over natuurvriendelijke oevers: 200 Natuurvriendelijke oevers: aanpak en toepassingen /Nature friendly banks: approach and applications

201 Idem: belasting en sterkte/loading and strength

202 idem: zie hierboven

203: idem: fauna

204 idem: vegetaties langs grote wateren/veg. along major rivers

205 idem: water- en oeverplanten/water and waterside plants (?)

Text for Figures TAW Technical Report on Water-retaining Soil Structures

CHAPTER ONE

Page 14

Fundamentals of Water Retaining Structures	Guidelines per type of water-retaining structure,	Technical Reports covering the various main aspects of water retaining structures,	Technical Reports covering individual issues
And	including:		including:
Guidelines on Safety Monitoring	Guidelines on Sea and Lake Dikes	Including: Technical Report on Water-retaining Earth Structures	Technical Report on Sand-carrying Springs

Figure 1.1.1 Relationships between the various TAW publications

CHAPTER TWO

Page 19

<i>buitenzijde</i>	Outer side
<i>Binnenzijde</i>	Inner side
<i>Kruin</i>	Crown
<i>Buitentalud</i>	Outer slope
<i>Binnentalud</i>	Inner slope
<i>Overgangstalud</i>	Transitional slope
<i>Dijkkern</i>	Dike core
<i>Kreukelberm</i>	Transitional berm
<i>Vooroever</i>	Fore bank
<i>Binnenberm</i>	Inner berm
<i>berm-/dijksloot</i>	Berm/dike ditch
<i>Dijkbasis</i>	Dike base
<i>Voorland</i>	Foreland
<i>Achterland</i>	Hinterland

Figure 2.3.1 General profile of a sea or river dike without special elements

Page 29

<i>Abrupt</i>	Abrupt	
<i>Geleidelijk</i>	Gradual	
<i>Land</i>	Land	
<i>Achterzijde afslag</i>	Rear-side erosion	
<i>Voorzijde constructie</i>	Front of the structure	
<i>Zee</i>	Sea	
<i>Voorbeelden gesloten eindconstructies</i>		Examples closed junction structures
<i>Achterzijde afslag</i>	Rear side erosion	
<i>Voorzijde constructie</i>	Front of the structure	
<i>Zee</i>	Sea	
<i>Voorbeelden open eindconstructies</i>		Examples open junction structures

Page 30

Figure 2.8.2 Connection dike to high ground

CHAPTER THREE
Page 34

NB: Figure 3.1.2 is incorrectly numbered in the Dutch text

<i>Stroming langs de dijk</i>	Current along the dike
<i>Erosie voorland en buitentalud, schade aan de bekleding</i>	Foreland and outer slope erosion, damage to revetment
<i>Lokale instabiliteit buitentalud</i>	Local instability outer slope
<i>Instabiliteit totale dijklichaam</i>	Instability entire dike body
<i>Golfaanval</i>	Wave attack
<i>Ijsgang</i>	Floating ice
<i>Kan belasting [2] genereren</i>	May generate loading [2]
<i>Hoge waterstand</i>	High water level
<i>Opbarsten door wateroverspanning</i>	Heave as a result of water pressure
<i>Lokale instabiliteit binnentalud</i>	Local instability inner slope
<i>Kwel tgv groot verhang</i>	Seepage as a result of large hydraulic gradient
<i>Verweking binnentalud</i>	Softening inner slope
<i>Erosie inwendig (piping)</i>	Internal erosion (piping)
<i>Erosie kruin/binnentalud door overslag/overloop..</i>	Erosion crown/inner slope as a result of overtopping/overflow
<i>Schadeontwikkeling</i>	Damage development
<i>Lage waterstand na hoge</i>	Low water level following high water level
<i>Locale instabiliteit buitentalud</i>	Local instability outer slope
<i>Samendrukbare ondergrond</i>	Compressible subsoil
<i>Kruip/opdrijven via [2]</i>	Creep/uplift via [2]
<i>Zakking kruin</i>	Falling crown height
<i>Schadeontwikkeling via [2]</i>	Damage development via [2]
<i>Overige</i>	Other
<i>"erosie" en nadien schadeontwikkeling via [1]</i>	'erosion' followed by damage development via [1]

Figure 3.1.1 Relationship between loading and damage development

Page 37

MHW + excess *)
MHW – AGA + excess *)
Surge h

Oosterschelde
Other
Hours

Figure 3.3.2 Development of the storm surge

CHAPTER FOUR
Page 45

Stap	Step
Globaal/ conservatief	General/Conservative
Conservatief ontwerp / toetsing	Conservative design/Testing
Scherp ontwerp / toetsing	Closely-defined design /Testing
Laagopbouw	Layer structure
Grondgedrag	Soil behaviour
Grondonderzoek	Soil analysis
Geohydrologische gegevens	Geo-hydrological data
Grondwaterstromingsmodel	Ground water flow model
schematisatie en beoordeling faalmechanismen	Representation and assessment failure mechanisms
Leeswijzer	Reference

Figure 4.1.1 Data and calculation models soil mechanics / geo-hydrology

Page 47

Jonge duinen	Young dunes
(Zee)afzettingen van Duinkerken	Marine deposits from Dunkirk
Oude duinen	Old dunes
Pleistocene- en oudere formaties aan of nabij de oppervlakte	Pleistocene and older formations on or near the surface
holocene veen- en rivierafzettingen	Holocene peat and river deposits
Landwaartse grens van de afzettingen van Calais	Landward border of the Calais deposits

Figure 4.2.1 Geological map of the Netherlands

Page 55

Figure 4.3.1	The Mohr-Coulomb collapse criterion	shear surface
--------------	-------------------------------------	---------------

Page 56

- 1 Firmly- packed sand
- 2 Loosely-packed sand
- 3 Over-consolidated clay
- 4 Normal consolidated clay
- 5 Peat

Figure 4.3.2 Pressure deformation behaviour of various soil types

Page 58

(maciaveld) Ground level

Figure 4.3.3 Possible variation pattern for undrained shear strength in a weak clay layer (note: in this figure, the undrained shear strength is represented by the international figure c_u instead of the national symbol f_{undr}).

Page 63

One day line
Pore number

Effective pressure

Figure 4.3.4 Relationship between porosity and pressure

Page 64

t[day]

Figure 4.3.5 Primary and secondary compression

Page 65

Tijdsfactor Time factor
Consolidatiegraag Consolidation level

Figure 4.3.6 Relationship between consolidation level U and time factor T

Page 74

Geo-electric profile
Apparent electrical resistance
Resistance line at measurement depth
Depth in m with respect to NAP
Depth sand subsoil
Cone resistance

Figure 4.4.1 Example of geo-electric profiling

Page 75

Local friction
Cone resistance
Water pressure

Figure 4.4.2 Penetration test cone

Page 76

Local friction resistance Cone resistance Friction number

Continuous penetration test performed according to NEN with electric friction sleeve
Measurement reach cone 1MPa

Measurement reach friction sleeve

Figure 4.4.3 Example of a penetration test graph

Wrijvings getal Friction number
Cylindrical, electrical friction sleeve cone

Conus weerstand cone resistance

Plaatselijke wrijving Local friction

Figure 4.4.4 Relationship between probe resistance, local friction and soil type

Page 78

Conusweertand cone resistance *Schuifweertand* Shear resistance

Silt
Clay with shells
Clay, humus, reed debris
Peat
Clay, reed debris
Clay, humus, reed debris
Clay sand, reed debris

Figure 4.4.5 Development of undrained strength with depth

Page 80

Cone resistance Pore water pressure
Depth in relation to NAP

Figure 4.4.6 Example of penetration test with water pressure measurement

Page 85

Triaxial extension test direct simple shear test triaxial compression test.

Figure 4.4.7 Sliding plane and soil investigation

Page 86

Phreatic line
Consolidation
Compaction
Dike
Holocene
Pleistocene

Figure 4.5.1 Holland profile and holding capacity

Phreatic line
Hydraulic head of sand subsoil

Figure 4.5.2 Ground water flow in dike profile and subsoil

Page 88

<i>Peilbuis</i>	Piezometric gauge
<i>Vochtverdelingen in grond</i>	<i>Distribution of moisture in soil</i>
<i>Werkelijk</i>	Actual
<i>Benadering</i>	Approach
<i>Capillaire zone</i>	Capillary zone

<i>Freatische lijn</i>	Phreatic line
<i>Verzadigingsgraad</i>	Degree of saturation
<i>Waterspanningsverdeling</i>	Water pressure distribution

Figure 4.5.3 Capillary zone

Page 89

Uplift zone
Without uplift
With uplift

Figure 4.5.4 Water pressure flow development in water-bearing layer with uplift

CHAPTER FIVE

Page 102

Outer side	Loss of stability	Polder side
		Weak layers
		Sand

Figure 5.3.1 Shearing on the inner side

Page 103

<i>Macrostabiliiteit</i>	Macro-stability
<i>Binnentalud</i>	Inner slope
<i>Buitentalud</i>	Outer slope
<i>MHW (Maatgevend Hoog Water)</i>	NHW (Normative High Water)
<i>Extreme neerslag</i>	Extreme precipitation
<i>Val van MHW</i>	Drop in NHW
<i>¹⁾ Veiligheid tegen opdrukken</i>	<i>¹⁾ Resistance to push up</i>
<i>Bishop</i>	Bishop
<i>Evt.</i>	Possible
<i>Spencer rechte glijvlakken of EEM</i>	<i>²⁾ Spencer straight slide planes or FEM</i>
<i>Altijd een controle met Bishop</i>	Always check with Bishop
<i>Opdrijven?</i>	Uplift?
<i>Drukstaaf berekening</i>	<i>²⁾ Pressure bar calculation</i>
<i>Heave:</i>	Heave
<i>Eventueel optimalisatie met EEM (bijvoorbeeld Plaxis)</i>	<i>²⁾ Possible optimisation with FEM (e.g. Plaxis)</i>

¹⁾ To determine the resistance to uplift, it may be necessary to take into account a less thick covering layer in the stability zone

²⁾ Horizontal deformation on the inner toe may not exceed 10 centimetres

Figure 5.3.2 Flowchart for the assessment of macro-stability

Page 107

<i>STABILITEITSVERLIES (OPDRIJVEN)</i>	as	Loss of stability (uplift)
<i>BUITENZIJDE</i>		Outer side
<i>POLDERZIJDE</i>		Polder side
<i>ZAND</i>		Sand

Figure 5.3.3 Loss of stability (uplift)

STABILITEITSVERLIES (OPBARSTEN)
 BUITENZIJD
 POLDERZIJD

Loss of stability (heave)
 Outer side
 Polder side

Figure 5.3.4 Loss of stability (heave)

Actieve zone	Active zone
Passieve zone	Passive zone
Buitenzijde	Outer side
Drukstaaf	Pressure bar
Opdrijfzone	Floating zone
Polderzijde	Polder side
Slappe Lagen	Weak layers
Zand	Sand

Figure 5.3.5 Forces taken into consideration in the 'pressure bar' method

Page 109

Passieve zone	Passive zone
Actieve zone	Active zone
Buitenzijde	Outer side
Polderzijde	Polder side
Slappe lagen	Weak layers
Zand	Sand

Figure 5.3.6 Diagram for heave: Bishop with $c' = 0$ and $\phi = 0$ in passive zone

Page 110

Dijkontwerp	Dike design	
Berekening stabiliteit met neutrale gronddruk naast de dijk		Calculate the stability with neutral soil pressure next to the dike
Ontwerp aanpassen	Adapt the design	
Berekening stabiliteit met passieve gronddruk naast de dijk		Calculate the stability with passive soil pressure next to the dike
Berekening vervorming	Calculate deformation	
Vervorming acceptabel	Acceptable deformation	
Optimalisatie mogelijk	Optimisation possible	
Stop	Stop	
γ = stabiliteitsfactor	γ = Stability factor	

Figure 5.3.7 Flowchart for the calculation of uplift

Page 119

Type of dike	Micro-stability	Stability with wave overtopping
Clay dike	No problem	Infiltration and shearing
Sand dike	Out-wash	Erosion of inner slope
Sand dike with clay covering	Push off /shearing	Infiltration and shearing
		Infiltration and micro-instability
		Shearing of upper slope

Figure 5.4.1 Comparison between micro-stability and stability with wave overtopping

□□□□□□□□

Phreatic line
Seepage surface

Figure 5.4.2 Upward pressure and pushing off of slope revetment in combination with internal erosion

□□□□ **122**

Point of influence
Seepage surface

Figure 5.4.3 Damage profile

□□□□□□□□

Permissible difference in hydraulic head
Lifting

Shearing
Angle of slope

Figure 5.4.4 Admissible hydraulic head for sand cores with clay revetments

Graph valid if $c = 2\text{kN/m}^2$ per m' $\tan \phi = 0.4$
 $\rho_g = 1630 \text{ kg/m}^3$ $\rho_w = 1000 \text{ kg/m}^3$

infiltratie Infiltration

Figure 5.5.1 Infiltration caused by wave overtopping

Page129

<i>Start</i>	<i>Start</i>
<i>Dichte kern (klei)</i>	<i>Closed core (clay)</i>
<i>Open kern (zand)</i>	<i>Open core (sand)</i>
¹⁾ <i>Wordt voldaan aan Joustra / Edelman criterium</i>	¹⁾ <i>Satisfies the Joustra/Edelman criterion</i>
²⁾ <i>Grondwaterdruk berekening + stabiliteitsanalyse</i>	²⁾ <i>Groundwater pressure stability analysis</i>
<i>calculation +</i>	
³⁾ <i>Grondwaterstromingsberekening + stabiliteitsanalyse</i>	³⁾ <i>Ground water flow</i>
<i>calculation +</i>	<i>stability</i>
<i>Wordt norm gehaald</i>	<i>Is the norm met</i>
<i>Goed</i>	<i>Good</i>
<i>Verbeteringsmodel</i>	<i>Improvement model</i>
<i>Ja yes nee no</i>	

Figure 5.5.2 Flow chart showing processes for analysing stability in infiltration

Page133

Angle of slope
Soil category

Insufficiently stable

Figure 5.5.3 Indication of the area of shearing

Page138

<i>Opdrijfzone</i>	<i>Uplift zone</i>
<i>Opbarstlocatie</i>	<i>Heave location</i>
<i>Theoretisch intreepunt</i>	<i>Theoretical point of entry</i>
<i>stijghoogte uit watex berekening</i>	<i>Hydraulic head from Watex calculation</i>
<i>grenspotential</i>	<i>Limit potential</i>
<i>Werkelijke stijghoogte (na opbarsten/opdrijven)</i>	<i>Actual hydraulic gradient (after heave/uplift)</i>

Figure 5.6.1 Dike with foreland and substrate; hydraulic head in sand layer, limit potential, heave location and uplift zone

Page138

Figure 5.6.2 Determination of the effective layer thickness for uplift near ditch

Page139

Figure 5.6.3 Seepage screen in a dike

Page140

Admissible

Figure 5.6.4 Calculation results 'Fragment Method' (see Figure 5.6.3)

Page145

Heave

Test with Lane method
investigation
Not always conservative

Unsatisfactory or unusable

Supplementary

Satisfactory

Test for heave Unsatisfactory

Satisfactory

Evaluation of observations

Indications of heave? Yes

No

No risk of heave

Further specialist
investigation

Figure 5.6.5 Heave in soil structures

Page146

Piping

Test for heave

No heave heave

Test with Bligh's method

Unsatisfactory Supplementary investigation

satisfactory

Test with Sellmeijer method

Unsatisfactory

Satisfactory

Evaluation of observations

Indications of piping Yes

No

No risk of piping

Further specialist investigation

Figure 5.6.6 Piping in soil structures

Page147

Profile before instability
Profile after instability

Figure 5.7.1 Two-dimensional absorption model with unlimited absorption

Page148

Hydraulische morfologisch condities	Hydraulic morphological conditions
Maximale ontgrondingsdiepte	Maximum erosion depth
Aanzethelling	Run-up slope
Aanvullende maatregelen	Additional measures
Geen maatregelen (veilig)	No measures (safe)
Aanzethelling na instabiliteit los/vast gepakt	Run-up slope after instability loosely/firmly packed
Inwendige hoek van wrijving	Angle of internal friction
Geotechnische condities	Geo-technical conditions

Figure 5.7.2 □ Relationship between strength and loading parameters

Page152

Onder	Under
Boveb	Above
Dijk	Dike
Geul	Channel
Step 1	
Step 2	
Step 3	

Figure 5.7.3 Schematization of saturated dike body in the vicinity of a channel

Boundary conditions/assumptions

Short crested waves

Shallow water

Wave approach perpendicular to dike

Smooth revetments for slopes 1:3 and 1:5 ($\gamma = -1$)

Grass covered revetments for slopes 1:8 ($\gamma = 0.95$)

Revetment extends from 3m under the berm to half the wave run up height above the berm

For slopes of 1:8 a cover of grass is used

Price of revetment material

1:3 £ 120/m²

1:5 £ 80/m²

1:8 £ 0/m² (grass)

Berm £ 50/m²

	Dike table height [m]								
	Slope								
	overtopping								
Berm width B	0 m								
	5 m								
	10 m								
	15 m								

	Volume of the transverse profile per metre of dike [m ³ /m] (Price of soil at £ 15/m ³)								
	slope								
	overtopping								
Berm width B	0 m								
	5 m								
	10 m								
	15 m								

	Width of revetment (Price at £ 17/ m ²)								
	slope								
	overtopping								
Berm width B	0 m								
	5 m								
	10 m								
	15 m								

	Wave run-up, area of slope revetment, cost of slope revetment								
	slope								
Berm	0 m								

width B	5 m									
	10 m									
	15 m									

	Total cost									
	slope									
	overtopping									
Berm width B	0 m									
	5 m									
	10 m									
	15 m									

Figure 6.3.1 Example showing the influence of the gradient of the outer slop on the transverse profile

Page 173

Zee	sea
Steenzetting	placed stone revetment
klei	clay
bestrating	paving
beton goot	concrete gutter
asfalt verharding	asphalt metalling
asfalt beton	asphalt concrete
talud bescherming	Slope protection
stortsteen	Rubble
kopersakblokken	Copper slag blocks
mijnsteen op nylandoek	mine stone on nylon cloth
drainleiding	drainpipe
PVC afvoereiding	PVC drain pipe
Betongoot met kolk	concrete gutter with outlet
Doorgrooiesteenen	soil filled cellular cement blocks
Kleidijk	clay dike
Steenachtige materialen	stony materials
Klinkers	clinkers
Straatzand	road sand
Zandbed	sand bed

Dike reinforcement on inner side

Dike reinforcement over old dike

Dike reinforcement on outer side

6.6.1 Various ways of reinforcing dikes

CHAPTER 7

Page182

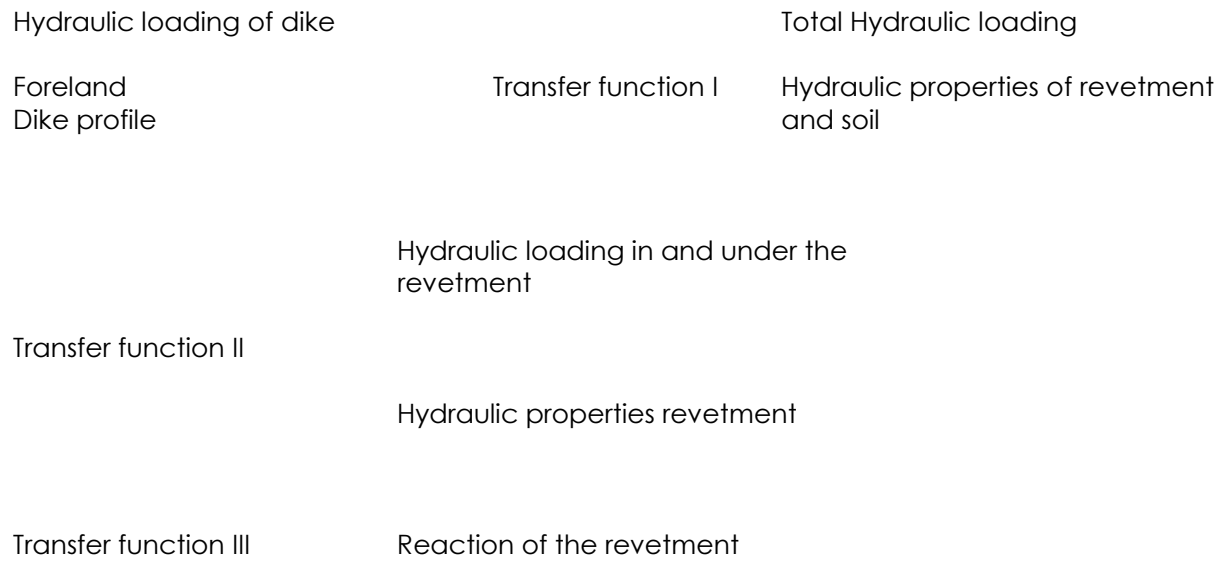


Figure 7.3.1 Components transferring the hydraulic load

Page183

Very open slightly open very closed

Figure 7.3.2 Basic types of revetment

CHAPTER 8

Page187

River Polder

Figure 8.2.1 Example of an assessment profile

River Polder

Figure 8.2.2 Example assessment profile with erosion screen

Page188

Testing zone
Daily crown
Residual crown
Stable inner slope

Figure 8.2.3 Principle of the erosion profile

Page190

Tree Profile	Crown width Crown height Slopes Dike structure Stability	Ample sufficient	Approve
Type of tree	See overview	Suitable	
Soundness	Expert inspection	Sound	
Outer slope	Erosion Macro-stability Micro-stability Damage profile	Adequate	
Crown adequate	Crown height Damage profile Stability Revetment	Satisfactory	
Foreland	Determine influence Entry resistance and response	Satisfactory	
Inner slope	Macro-stability Micro-stability Damage profile Revetment	Satisfactory	
Hinterland	Macro-stability Micro-stability Damage profile Revetment	Satisfactory	
Hinterland	Test for piping	Satisfactory	
Monitoring and management	Satisfactory	Exception	
	Approve	Reject	

Figure 8.2.4 Flow diagram assessment trees on water defences

Page192

Building Profile	Crown width Crown height Slopes Dike structure	Ample sufficient
------------------	---	------------------

	Stability	
Condition	Soil-retaining wall Basement floor (see addendum)	Satisfactory
	Location of the building	
Outer slope and crown Erosion Macro-stability Micro-stability Damage profile		Inner slope Erosion Macro-stability Revetment micro-stability
Foreland Determine influence entry resistance and response		Hinterland Test for piping Satisfactory
Exception		Monitoring and management
Reject		Accept/approve

Figure 8.2.5 Assessment buildings on water defences

CHAPTER 9 0

CHAPTER 10

CHAPTER 11 Page 224

Figure 11.1.1 Location of the model profile the Driehoek along the Zwarte Water.

Page 226

Pb = Piezometric gauge
Bb = Drilling
s = Penetration test
w = Water pressure meter

Clay
Peat
Sand

Figure 11.2 .1 Profile at the Driehoek

Shear pressure [kPa]
Normal pressure [kPa]

Figure 11.2.2 Calculation values for strength parameters according to the soil-mechanical investigation

Hydraulic head (metres) with respect to NAP
Time

Figure 11.3.1 Water pressure measurements in the model profile

Page 234

Design for inward stability $SF = 0.451$

Figure 11.3.2 Test for inward macro-stability stability

Page 236

Design for outward stability $SF = 0.608$

Figure 11.3.3 Test for outward macro-stability

Page 237

Design for inward stability $SF = 1.236$

Figure 11.3.4 Dimensions of the inner support berm. Length = 10.5 m thicken = 0.9

Page 238

Design for outward stability $SF = 1.100$

Figure 11.3.5 Dimensions of the outer support berm. Length = 6.5 m thickness = 1.0

ANNEX 1**Page 248**

Maaiveld Surface

Figure 1.2.1 Possible variation pattern for undrained shear strength in a weak clay layer

Page 250

Variance reduction factor

Figure 1.3.1 Variance reduction factor of averaging in the horizontal direction

Page 251

Variance reduction factor

Figure 1.3.1 Variance reduction factor of averaging in the vertical direction

ANNEX 2**Page 257**

A = Active zone
D = Direct simple shear zone
P = Passive zone

Figure 2.1 Undrained failure of an ideal plastic material

Page 258

Compressive zone ATC	Direct simple shear test DSS	Extension test ATE
Active failure	Horizontal failure	Passive failure

Figure 2.2 Slide cylinder with relevant laboratory tests

Page 259

Angle of slope of the slide plane

Figure 2.3 Anisotropy of the undrained shear strength

Page 261

Gem. Schuifvervorming	av. Shear deformation
--------------------------	--------------------------

Figure 2.4 Determination of the undrained shear strength (and) taking into account the compatibility of the strain and assuming a rigid slide plane.

ANNEX 3

Page 264

Drill tube
Buffer Stoot ring
steel tube
loose steel ball/valve ball
steel penetration tube

sizes in mm

Figure 3.1 Ackerman drilling

Figure 3.2 Continuous penetration apparatus

Page 265

Figure 3.3 Core obtained by using continuous penetration apparatus

ANNEX 4
Page 264

Rotation rod
Ball bearing
Vane housing
Rod
vane

Measuring unit
casing

Figure 4.1 Geometry of the vane apparatus

Figure 4.2 Vane apparatus with casing
Page 268

Silt
Clay with shells
Clay, humus, reed debris
Peat
Clay, reed debris
Clay, humus, reed debris
Clay sand, reed debris Silt

Figure 4.3 Development of undrained strength with depth

ANNEX 5
Page 271

Figure 5.1 Marchetti dilatometer

Figure 5.2 Dilatometer blade

Page 272

Figure 5.3 Cone pressure meter

Radial pressure
Radial deformation

maximum pressure
plastic unloading

Figure 3.4 Cone pressure meter, unloaded and reloaded curves

ANNEX 6
Page 275

Soil sample
Porous plate

Figure 6.1 Compression test

Page 276

T time [day]
Z_t
Z_p(σ') primary settlement according to NEN 6740 [m]
Z_s(t) secondary settlement according to NEN 6740 [m]
U(t) degree of consolidation [-]

Figure 6.2 Primary and secondary compression

APPENDIX 7

Page 277

Figure 7.1 Direct shear test

Shear stress
Horizontal displacement

Sample
Normal pressure [kN/m²]
Shear stress [kN/m²]
Displacement [mm]
Settlement [mm]
Dry density [Mg/m³]
Pore number [-]

Figure 7.2 Results of Direct Shear test

ANNEX 8

Page 279

Figure 8.1 Stress and deformation

Page 280

Figure 8.2 DSS apparatus
