A Clay Layer as a Revetment for Sea Dikes
The Behaviour of Clay under Wave Loading

Master Thesis

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Photograph on the cover:
Dike with a revetment consisting of a thick clay layer, located at the test section at Het Verdronken Land van Saeftinghe, Zeeuws-Vlaanderen, The Netherlands.

Preface

This report is the result of the research performed for my master thesis at the faculty of Civil Engineering and Geosciences of Delft University of Technology.

This research project looks at the possibility of using a thick clay layer to form a sustainable and feasible revetment for the outer slope of sea dikes. More specifically, it deals with the behaviour of clay under wave loading.
This research has been performed in conjunction with GeoDelft, National Institute for Geo-Engineering, and Delft University of Technology.

I would like to thank my graduation committee, prof. drs. ir. J.K. Vrijling, ir. H.J. Verhagen, dr. ir. A.L.A. Fraaij, dr. ir. A.R. Koelewijn and ir. T.P. Stoutjesdijk. Also, I would like to thank GeoDelft for the use of their facilities. Furthermore I would like to thank the people who helped to correct my writings, and especially Vanessa and Ineke for their advice. Finally, I want to thank Job for his patience and support.

Marieke de Visser

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Summary

Environmental awareness and sustainable development have been popular themes over the last few years. Thanks to this awareness and due to European legislation, governments and companies are forced to give more consideration to the environment.

The water defences along the Dutch coast need strengthening due to the rise of the sea level and heavier hydraulic loads. These required adaptations offer the opportunity for new integral solutions. Besides safety, attention is paid to the themes landscape, nature, cultural heritage (the so called LNC-values) and to recreation.

To improve dikes along the estuaries where a high foreland with natural areas of salt marshes is present, alternatives to the traditional stone revetments are examined. From a natural and aesthetic point of view, a slope covered with vegetation is preferred over a stone revetment, because in this way a more gradual transition between the salt marshes and the dike with the hinterland is obtained.

Revetments which obtain their strength from the grass cover or the clay layer are possible alternatives to the traditional stone revetments. It is however difficult to obtain a good quality grass cover along the coast due to the presence of swash marks, salt water and irregularities in the sod.

This research project looks at the possibility of using a thick clay layer to form a sustainable and feasible revetment for sea dikes in the presence of a high foreland, with sufficient strength and stability to withstand a normative load, a design storm.

Background
The concept of a revetment consisting of clay was monitored for a number of years at two test sections at Het Verdronken Land van Saeftinghe, Zeeuws Vlaanderen (The Netherlands), where a natural salt marsh area is located in front of the dike. A high foreland can give a reduction in the wave load on the slope, because the waves break earlier. During the test period no erosion was observed and the vegetation in the clay was well developed. However, no storms occurred during the monitoring period.
Based on the monitoring and due to a lack of more data, Rijkswaterstaat considers the use of a clay layer to be a suitable revetment for sea dikes if a high foreland is present in front of the dike and the significant wave height is lower than 2m. This alternative revetment is therefore occasionally applied at suitable locations in Zeeuws Vlaanderen. A design method for clay revetments has been developed, but is considered to be unsure.

This research project concentrates on the behaviour of clay under wave loading and the influencing factors on this behaviour, because there is still little knowledge on this behaviour.
All performed large scale experiments which involve clay under wave loading are investigated and compared in this research. This leads to a complete overview of all relevant large scale experiments, which is used to verify if a comparison of these experiments results in a broader view on the behaviour of clay under wave loading. The results are analysed and provide new valuable conclusions and are used to validate existing conclusions and assumptions.
Comparisons with the erosion behaviour of sand and with observations of outer slope damage in actual storms give additional conclusions on the behaviour of clay and a validation of the conclusions from the experiments.
A semi quantitative model has been developed on the basis of the results of the experiments on clay. This model can be further extended if more influencing factors on the erosion process of clay are quantified.

The behaviour of clay under wave loading

A clay revetment can be schematised into four layers on top of a core of sand. See Figure 1.

1. Vegetation on top of the soil
2. Vegetation layer in the soil, root structure
3. Structured clay
4. Unstructured clay
5. Sand (core of the dike)

The behaviour of clay is researched with respect to three different subjects: the process of erosion development, the location of erosion and the shape of erosion profiles.

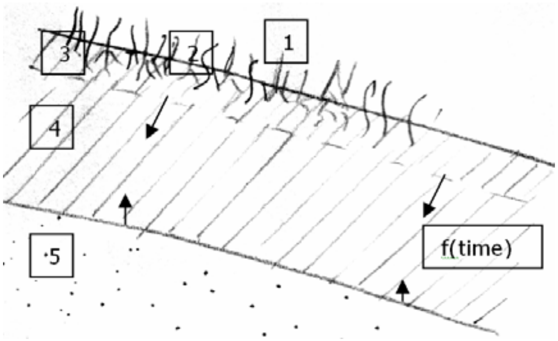


Figure 1 Schematisation of a clay layer on a dike.

The development of erosion

Comparison of large scale experiments on clay

On the basis of a comparison of all the experiments on clay under wave loading it can be concluded that the clay type, expressed as the sand percentage of the clay, is not the major influencing factor on the erosion rate of clay under wave loading.

The influence of the clay condition on the strength of the clay under a wave load is significant. The clay condition is the amount of soil structure development in the clay, which develops over time and in depth based on several influencing factors such as the sand content, vegetation growth, animal digging and compaction.
Based on the experiments, the clay condition can be divided in three conditions: structured clay, moderately structured clay and unstructured clay, as can be seen in Figure 1. Unstructured clay has a significant lower erosion rate than structured clay. Moderately structured clay forms a transitional condition. See also Figure 2 and Figure 3.

Within this subdivision in clay conditions, the clay type and the wave conditions are of influence. A higher sand percentage and a higher wave height result roughly in a higher erosion rate. The clay type is however more of influence than the wave characteristics. Unstructured clay with a low sand percentage has a very high erosion resistance.

The breaker type and thus also the slope angle, probably influence the erosion process. A different breaker type results in a different load on the slope and therefore in a different erosion mechanism.

An empirical formula of the residual strength of clay under wave loading was developed by WL|Delft Hydraulics (2006b), which determines depth of erosion over time. This formula fits well to the data it is based on, however it delivers no useful results when compared to the data of the other experiments.

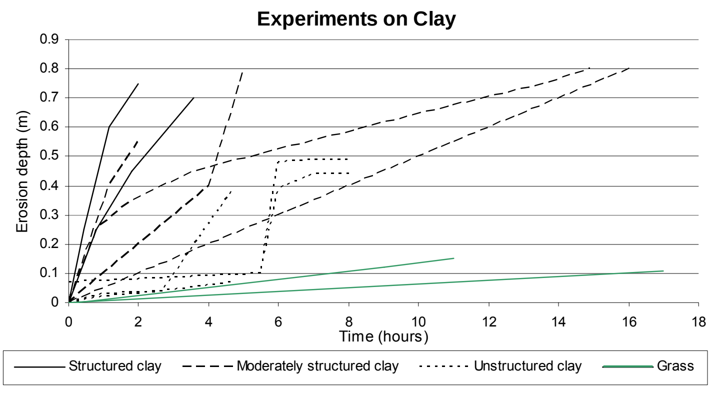


Figure 2 All Delta Flume experiments on clay: structured clay, moderately structured clay and unstructured clay.

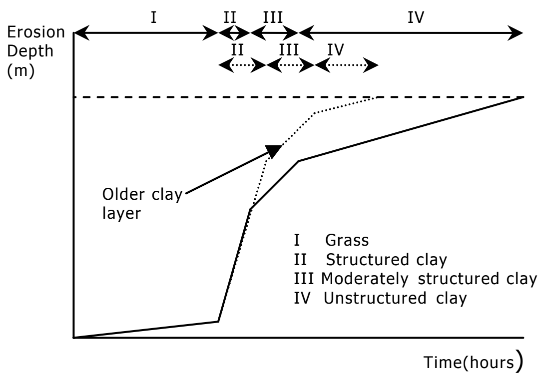


Figure 3 Schematisation of the development of erosion of a clay layer under wave loading over time.

Based on data of limited experiments on grass under wave loading it can be concluded that a grass cover has a higher erosion resistance against wave loading than clay. The erosion rate of unstructured clay with a low sand percentage is approximately similar to the erosion rate of grass. If more information on the erosion development of grass is obtained, this can be combined with the erosion development of clay in the semi quantitative model (and step I in Figure 3). Some vegetation always will be present on top of a clay layer and can provide additional strength.

Comparison with erosion of sand
A comparison with the erosion behaviour of sand is made to obtain more insight into the behaviour of clay under wave loading in respect to that of sand. The erosion of dunes has been studied extensively over the past years and therefore much more is known on the erosion behaviour of sand under wave loading. Delta Flume experiments on sand are used to make this comparison.

The comparison shows that clay has a significantly lower erosion rate than sand. The erosion is measured perpendicular and horizontal to the slope, both give a different erosion development in time. See Figure 4. The development of the erosion of clay and sand horizontally measured over time are similar, but the amount of erosion of clay is lower. For unstructured clay the horizontal erosion development over time is different and therefore the comparability of the determined horizontal erosion rate of unstructured clay is debatable.

- The erosion of clay measured perpendicular to the slope is approximately a factor of 2-10 lower than of sand.
- Horizontally measured, the erosion of structured clay and moderately structured clay is approximately a factor of 1.5-3.5 lower than of sand.

Comparison with actual storms
The experiments show less erosion development over time than in reality was observed. See Figure 4. Observations during the storm surge disaster of 1953 show more horizontally measured erosion on dikes than observed in the experiments on sand. Measured perpendicular to the slope, the observed amount of erosion of dikes and dunes is approximately similar to the experiments on sand.
The erosion of all dikes was developed through the clay layer into the sand core of the dike. Unexpectedly the observed erosion of the dikes was larger than that of the dunes. However, the erosion data of 1953 are not accurate and the wave heights were generally higher than those used in the experiments.

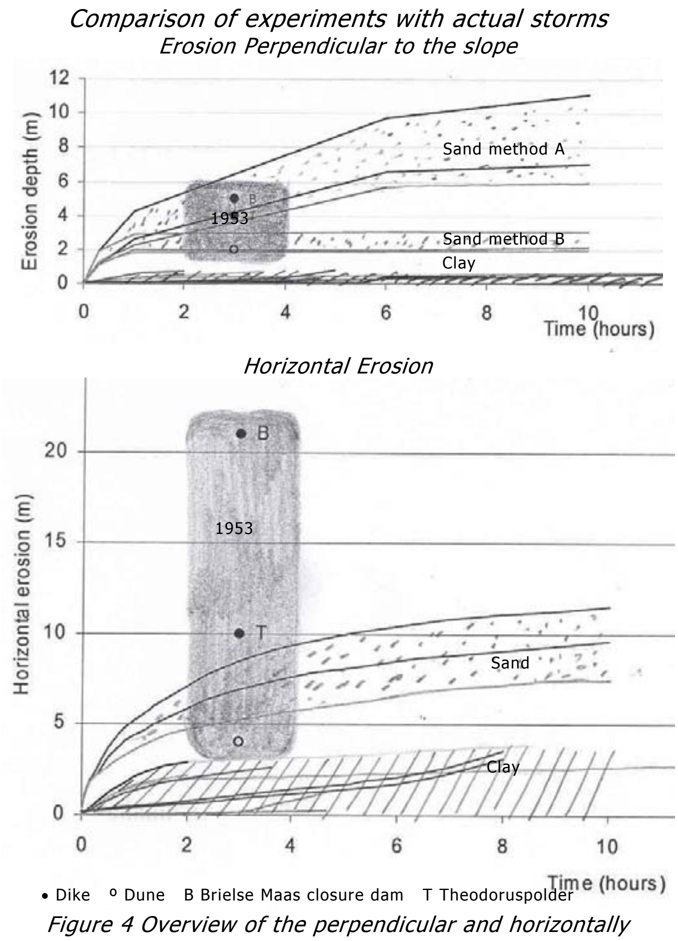


Figure 4 Overview of the perpendicular and horizontally measured erosion found in the Delta Flume experiments on clay and sand and observations during the storm surge disaster of 1953.

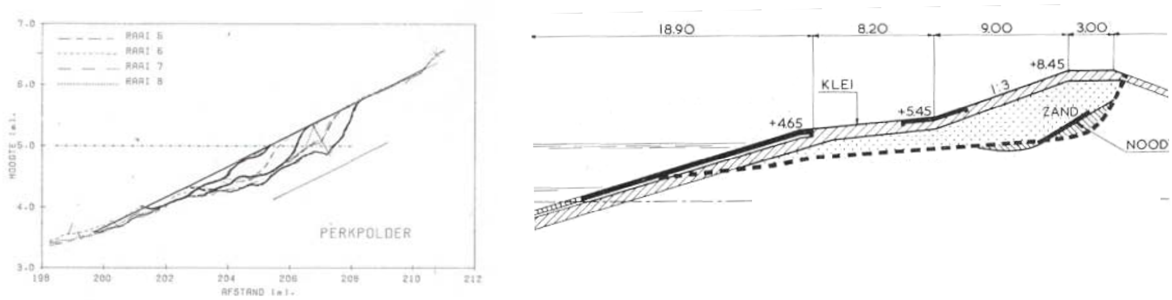
Several factors probably influence the erosion process of clay on dikes and are likely the cause of the difference in results of the experiments and actual damage. These factors include:

- Weak locations will be present in a dike and will be normative for the start of erosion.
- Sand and debris will be present in the water during storm surges, which could initiate or increase erosion by scouring the slope. It also could function as bulkhead after deposition in front of the dike.
- The salt in the seawater could have a certain influence on the erosion development of clay.
- The wave conditions and breaker types in actual storm surges could be different from those used in the experiments.

The location of erosion
The location of erosion is different for clay and sand. The erosion of clay starts below the water level by the development of a hole, approximately between $0.3H_s$ and $1.2H_s$ below the water level. The hole can extend and develop till above the water level, if the erosion process has sufficient time to continue. The erosion of sand develops above the water level. These conclusions are confirmed by the observations made in the storm surge disaster of 1953. This difference is probably the result of different erosion mechanisms of clay and sand.

The defined theory of erosion zones by Smith (1994a) does not completely correspond to all experiments.

The shape of the erosion profile
After an initial erosion hole has developed in the clay, the erosion profile of clay has a steep slope, the shape of a cliff.
In sand a steep cliff develops, which retreats in a horizontal direction. The cliffs developed in clay are mutually more different and sometimes gentler than in sand.
These conclusions are confirmed by the observations made in storm surges in history.



Experiment on moderately structured clay *Erosion of a dike after a actual storm*
Figure 5 Examples of erosion profiles.

The conclusions drawn in this research are based on all available, but limited amounts of data.

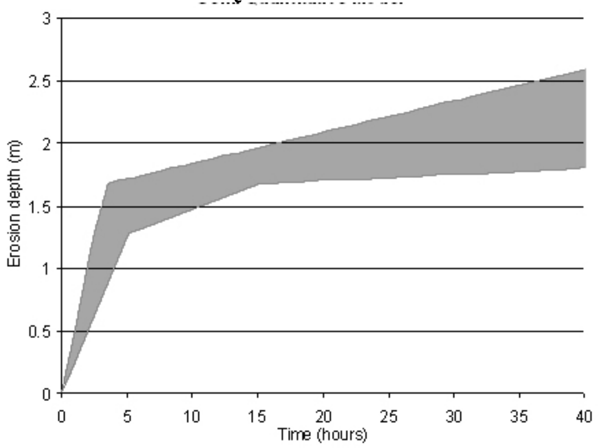
Modelling the behaviour of clay under wave loading

A semi quantitative model to determine the erosion depth over time was developed in this research. See left of Figure 6. This model is based on the determined subdivision of the experiments on clay in the three different clay conditions. The semi quantitative model consists of a combination of the linear development over time for each clay condition, with the gradient based on the results from the experiments on clay. The transitions between the clay conditions are dependent on the soil structure present in the clay layer. This can be determined with a formula for the soil structure development over time and in depth, developed in this research. See right of Figure 6.

On the basis of the present available data of the large scale experiments, it is not possible to quantify the influence of the clay type and wave characteristics. Only an approximate influence of the wave height can be defined.
The semi quantitative model was developed in a way that it can be extended if the influencing factors on the erosion process of clay are quantified.

The semi quantitative model determines approximately the average erosion over time in comparison with the developed empirical model by WL|Delft Hydraulics (2006b). The empirical model however appeared not to be applicable for all Delta Flume experiments on clay.
The design method developed by the engineering firm INFRAM (2003) determines between approximately 3 and 10 hours less erosion than the semi quantitative model. During longer storm durations, the erosion of the design method is larger than the semi quantitative model, as expected.

Semi Quantitative Model



Development of the soil structure over time and in depth

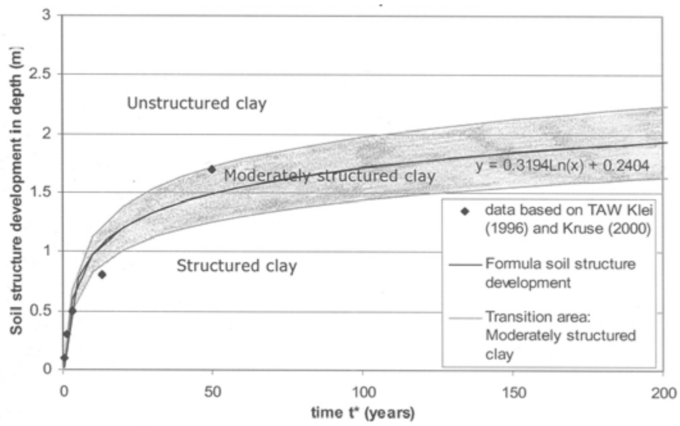


Figure 6 Semi Quantitative Model (left), developed in this research, to determine the erosion depth of a clay layer over time under wave loading. This model is based on the determined subdivision of the experiments on clay in the three different clay conditions and a developed formula for the development of the soil structure over time and in depth (right).

The use of clay as a revetment for sea dikes

On the basis of this research it can be concluded that a thick clay layer can possibly form an environmentally acceptable alternative to the traditional stone revetment, however research is not yet sufficient enough.

The clay erosion of a clay layer over time can be determined with the semi quantitative model that has been developed as a part of this research project. As far as this research demonstrates the wave load should not be severe (H_s=1.5m is maximum in used data).
A case study of Het Verdronken Land van Saeftinghe was made with the semi quantitative model developed in this research. This study showed that a 100 year old clay layer would erode approximately 0.8m-1.6m in 3 hours, 1.6m-2.2m in 10 hours and 2.0m-2.7m in 30 hours. The semi quantitative model should not be used in practice, without further research and enhancing of the model.
Especially the differences of experimental results with observations made in actual storms should be included in this research.

During the construction of clay dikes, special attention has to be given to obtaining an unstructured clay condition, for example by compacting during construction and the use of clay with a low sand percentage.

The results of this research project are not limited to sea dikes. It is also possible to determine if a thick clay layer can be applied on other locations, for example on river dikes.

Before actually using a clay revetment in sea dikes, the feasibility and sustainability of this alternative revetment should be elaborated further upon the aspects of technological feasibility, costs, maintenance, availability of material, chances for nature development and obtaining a social basis.

Samenvatting

Milieubewustzijn en duurzame ontwikkeling zijn populaire thema's van de laatste jaren. Door deze bewustwording en door Europese regelgeving zijn overheden en bedrijven gedwongen aandacht te schenken aan natuur en milieu.

De waterkeringen langs de Nederlandse kust vereisen versterking als gevolg van zeespiegelstijging en hogere hydraulische belastingen. Deze vereiste aanpassingen bieden mogelijkheden voor nieuwe, integrale oplossingen, waarbij naast veiligheid, ook aandacht geschonken wordt aan de thema's landschap, natuur, cultureel erfgoed (de zogenaamde LNC-waarden) en aan recreatie.

Voor de versterking van de dijken langs de Nederlandse estuaria, waar een hoog voorland aanwezig is met natuurgebieden bestaande uit schorren, wordt gezocht naar alternatieven voor de traditionele steenbekledingen. Vanuit natuurlijk en esthetisch oogpunt heeft een dijktalud bekleed met vegetatie de voorkeur boven een steenbekleding, omdat op deze manier een meer geleidelijke overgang tussen de schorgebieden en de dijk met het achterland verkregen wordt.

Bekledingen die hun sterkte verkrijgen uit de grasbedekking of uit de kleilaag zijn mogelijke alternatieven voor de traditionele steenbekledingen. Het is echter moeilijk om een goede kwaliteit grasbekleding te verkrijgen langs de kust door de aanwezigheid van veek, zout water en onregelmatigheden in de graszode.

Dit onderzoeksproject kijkt of het gebruik van een dikke kleilaag mogelijk een duurzame en haalbare oplossing kan zijn voor de bekleding van het buitentalud van zeedijken in de aanwezigheid van een hoog voorland. Een bekleding met voldoende sterkte en stabiliteit om een maatgevende belasting, een ontwerpstorm, te weerstaan.

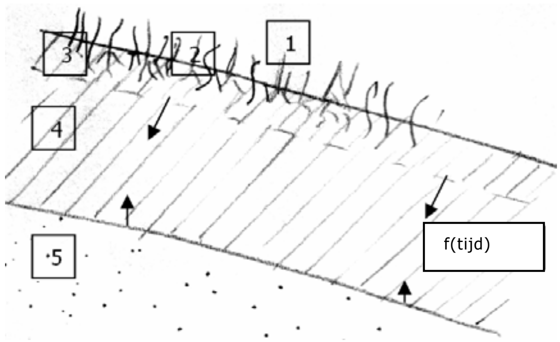
Achtergrond
Het concept van een kleibekleding is een aantal jaren getest op twee testlocaties bij Het Verdronken Land van Saeftinghe (Zeeuws Vlaanderen, Nederland), waar een natuurgebied van schorren aanwezig is voor de dijk. Een hoog voorland voor de dijk kan een vermindering van de golfbelasting op het talud geven, omdat de golven eerder breken. Tijdens de testperiode is geen erosie geobserveerd en vegetatiegroei in de klei was goed ontwikkeld. Er zijn echter geen stormen geweest tijdens deze periode.
Gebaseerd op deze testperiode en bij gebrek aan meer data beschouwt Rijkswaterstaat een kleilaag als een goede bekleding voor zeedijken, mits een hoog voorland aanwezig is voor de dijk en de significante golfhoogte lager is dan 2m. Deze alternatieve bekleding wordt daarom incidenteel toegepast op hiervoor geschikte locaties in Zeeuws Vlaanderen. Er is een ontwerpmethode voor deze kleibekledingen ontwikkeld, maar deze wordt als onzeker beschouwd.

Dit onderzoeksproject is geconcentreerd op het gedrag van klei onder golfbelasting en de factoren van invloed op dit gedrag, waarover nog steeds weinig kennis is.
In dit onderzoek zijn alle uitgevoerde grootschalige experimenten van klei onder golfbelasting onderzocht en met elkaar vergeleken. Dit leidt tot een compleet overzicht van alle relevante grootschalige experimenten, dat is gebruikt om te verifiëren of een vergelijking van al deze experimenten resulteert in een bredere kijk op het gedrag van klei onder golfbelasting. De resultaten zijn geanalyseerd en leveren nieuwe waardevolle conclusies en zijn gebruikt om bestaande conclusies en aannames te valideren.
Een vergelijking met het erosiegedrag van zand en met waarnemingen van schade in echte stormen geven aanvullende conclusies over het gedrag van klei en een validatie van de conclusies van de experimenten op klei.
Een semi-kwantitatief model is ontwikkeld op basis van de resultaten van de experimenten op klei. Dit model kan verder uitgebreid worden als meer factoren, die van invloed zijn op het erosieproces van klei, gekwantificeerd zijn.

Het gedrag van klei onder golfbelasting

Een kleibekleding kan worden geschematiseerd door vier lagen op een kern van zand. Zie Figuur 1.

1. Vegetatie op de bodem
2. Vegetatie in de bodem, de wortellaag
3. Gestructureerde klei
4. Ongestructureerde klei
5. Zand (kern van de dijk)



Figuur 1 Schematisatie van een kleilaag op een dijk.

Het gedrag van klei is onderzocht op drie verschillende onderwerpen: de erosieontwikkeling, de locatie van de erosie en de vorm van het erosieprofiel.

De erosieontwikkeling

Vergelijking van grootschalige experimenten op klei

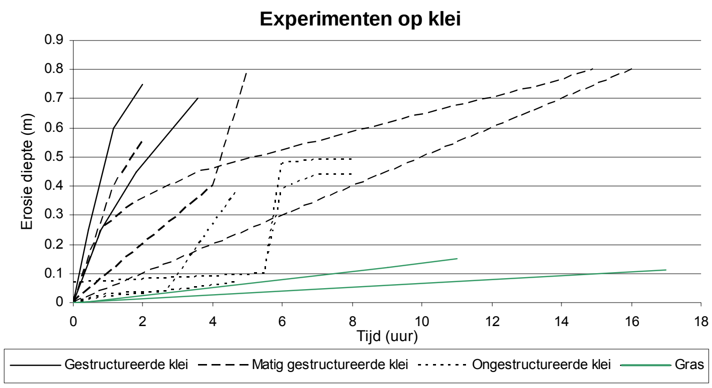
Op grond van de vergelijking van alle experimenten op klei onder golfbelasting kan worden geconcludeerd dat het kleitype, uitgedrukt als het zandpercentage van de klei, niet de maatgevende factor is op de erosiesnelheid van klei onder golfbelasting.

De invloed van de kleiconditie op de sterkte van de klei onder golfbelasting is significant. De kleiconditie is de hoeveelheid bodemstructuurvorming in de klei, wat zich ontwikkeld in tijd en in de diepte afhankelijk van verschillende factoren, zoals het zandpercentage, vegetatiegroei, het graven van dieren en de verdichting.
Op basis van de experimenten kan de kleiconditie onderverdeeld worden in drie condities: gestructureerde klei, matig gestructureerde klei en ongestructureerde klei, zoals is te zien in Figuur 1. Ongestructureerde klei heeft een significant lagere erosiesnelheid dan gestructureerde klei. Matig gestructureerde klei vormt een overgangsconditie. Zie ook Figuur 2 en Figuur 3.

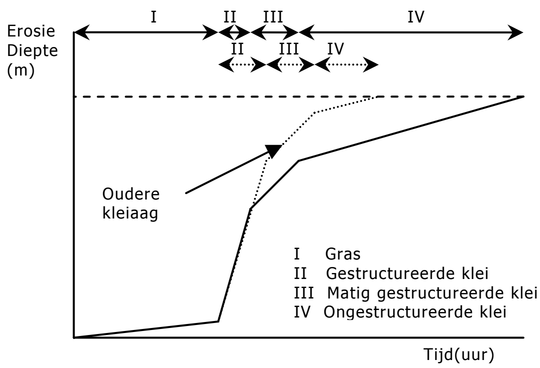
Binnen deze onderverdeling in kleicondities, zijn het kleitype en de golfcondities van invloed. Een hoger zandpercentage en een hogere golfhoogte resulteren bij benadering in een hogere erosiesnelheid. Het kleitype is echter meer van invloed dan de golfkarakteristieken. Ongestructureerde klei met een laag zandpercentage heeft een zeer hoge weerstand tegen erosie.

Het brekertype, en dus ook de taludhelling, zijn waarschijnlijk van invloed op het erosieproces. Een ander brekertype resulteert in een andere belasting op het talud en daardoor in een ander erosiemechanisme.

Een empirische formule voor de reststerkte van klei onder golfbelasting, die de erosiediepte in de tijd bepaald, is ontwikkeld door WL|Delft Hydraulics (2006b). Deze formule geeft goede resultaten voor de data waarop het is gebaseerd, maar niet voor de andere experimenten.



Figuur 2 Alle Deltagoot experimenten op klei: gestructureerde klei, matig gestructureerde klei en ongestructureerde klei.



Figuur 3 Schematisatie van de erosieontwikkeling in een kleilaag onder golfbelasting in de tijd.

Op grond van de data van een beperkt aantal experimenten op gras onder golfbelasting kan worden geconcludeerd dat een graslaag een hogere erosieweerstand heeft dan klei. De erosiesnelheid van ongestructureerde klei met een laag zandpercentage is ongeveer gelijk aan de erosiesnelheid van gras. Als meer informatie over de erosieontwikkeling van gras is verkregen, kan dit worden gecombineerd met de erosieontwikkeling van klei in het semi-kwantitatieve model (en als stap I in Figuur 3). Enige vegetatie zal altijd aanwezig zijn boven op een kleilaag en kan een aanvullende sterkte verlenen.

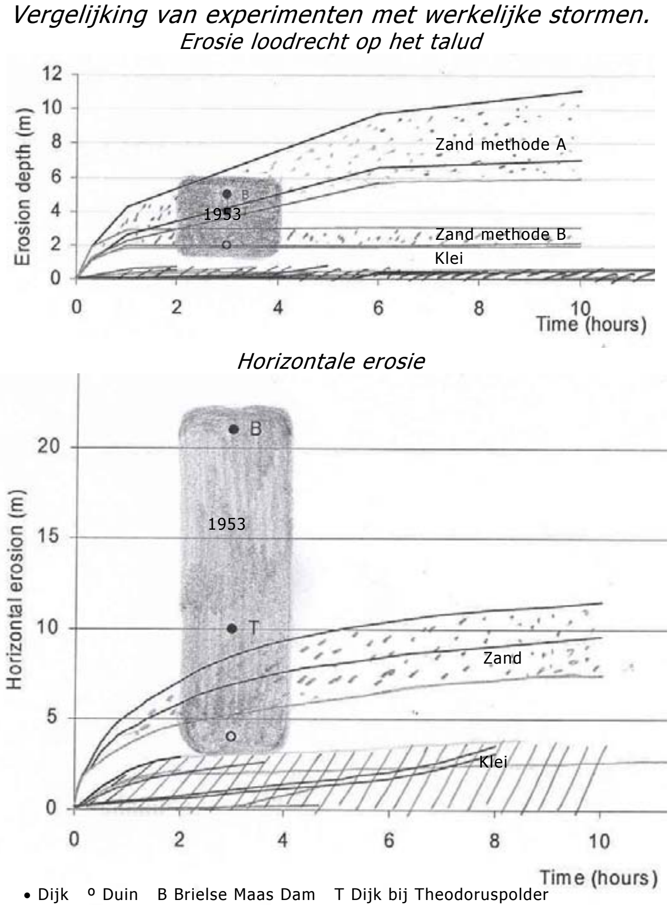
Vergelijking met de erosie van zand
Een vergelijking met het erosiegedrag van zand is gemaakt om meer inzicht te krijgen in het gedrag van klei onder golfbelasting ten opzichte van dat van zand. De erosie van duinen is afgelopen jaren uitgebreid bestudeerd en daarom is er over het erosiegedrag van zand meer bekend. Voor deze vergelijking is gebruik gemaakt van Deltagoot experimenten op zand.

De vergelijking laat zien dat klei een significant lagere erosiesnelheid heeft dan zand. De erosie is gemeten loodrecht en horizontaal op het talud. Beide meetmethodes geven een verschillende erosieontwikkeling in de tijd. Zie Figuur 4. De ontwikkeling van de horizontaal gemeten erosie van klei en zand in de tijd zijn gelijk, maar de hoeveelheid erosie van klei is lager. Voor ongestructureerde klei is de horizontale erosieontwikkeling in de tijd verschillend en daarom is de vergelijkbaarheid van de vastgestelde horizontale erosiesnelheid van ongestructureerde klei discutabel.

- De erosie van klei gemeten loodrecht op het talud is bij benadering een factor 2-10 lager dan zand.
- Horizontaal gemeten is de erosie van gestructureerde klei en matig gestructureerde klei bij benadering een factor 1,5-3,5 lager dan zand.

Vergelijking met werkelijke stormen
De experimenten geven minder erosieontwikkeling in de tijd dan in werkelijkheid is geobserveerd. Zie Figuur 4. Waarnemingen na de stormvloed van 1953 geven meer horizontaal gemeten erosie op dijken dan is geobserveerd in experimenten op zand. Loodrecht op het talud gemeten, is de waargenomen erosie van dijken en duinen bij benadering gelijk aan de experimenten op zand.
De erosie van alle dijken heeft zich ontwikkeld door de kleilaag in de zandkern van de dijk. Onverwachts is de waargenomen erosie van de dijken groter dan van de duinen. De erosiedata van 1953 zijn echter niet nauwkeurig en de golfhoogtes zijn over het algemeen groter dan gebruikt in de experimenten.

Vergelijking van experimenten met werkelijke stormen.



Figuur 4 Overzicht van de loodrecht en horizontal gemeten erosie gevonden bij Deltagoot experimenten op klei en zand en de waarnemingen na de stormvloed van 1953.

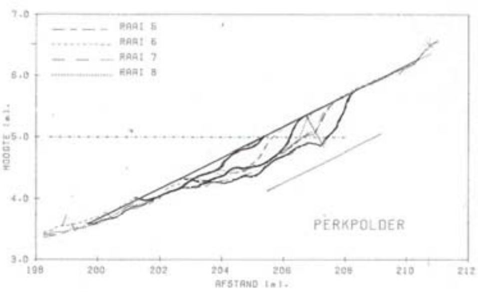
Verschillende factoren beïnvloeden het erosieproces van klei in dijken en ook de verschillen in de resultaten van de experimenten en de werkelijke schade:

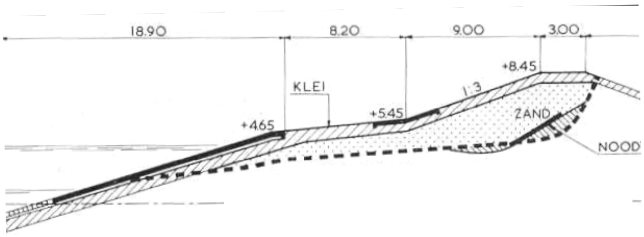
- De zwakke plekken in dijken zullen maatgevend zijn voor de start van erosie.
- Zand en puin zullen aanwezig zijn in het water tijdens een storm, wat erosie kan initiëren of vergroten door schuring op het talud. Het zou ook kunnen functioneren als beschoeiing na aanzanding voor de dijk.
- Het zout in het zeewater zou een bepaalde invloed kunnen hebben op de erosieontwikkeling van klei.
- De golfcondities en brekertypes in echte stormen kunnen verschillen van degenen die gebruikt zijn in de experimenten.

De locatie van erosie
De locatie van erosie is verschillend voor klei en zand. De erosie van klei begint onder de waterspiegel met de ontwikkeling van een gat, bij benadering tussen $0,3H_s$ en $1,2H_s$ onder de waterspiegel. Als het erosieproces voldoende tijd heeft om zich voort te zetten kan dit gat zich uitbreiden en zich ontwikkelen tot boven de waterspiegel. De erosie van zand ontwikkelt zich boven de waterspiegel. Deze conclusies worden bevestigd door de waarnemingen gemaakt na de stormvloed van 1953. Dit verschil in locatie is waarschijnlijk het gevolg van verschillende erosiemechanismen van klei en zand.

De ontwikkelde theorie van de erosiezones van Smith (1994a) komt niet volledig overeen met alle experimenten.

De vorm van het erosieprofiel
Nadat zich een beginnend erosiegat heeft ontwikkeld in de klei, krijgt het erosieprofiel van de klei een steile helling, in de vorm van een klif.
In zand ontwikkelt zich een steile klif, die zich terugtrekt in horizontale richting. De kliffen die zich in klei ontwikkelen zijn onderling meer verschillend en soms flauwer dan in zand.
Deze conclusies zijn bevestigd door de waarnemingen gemaakt na diverse stormen in het verleden.





Experiment op matig gestructureerde klei *Erosie van een dijk na een echte storm*

Figuur 5 Voorbeelden van erosieprofielen.

De conclusies in dit onderzoek zijn gebaseerd op alle aanwezige, maar beperkte hoeveelheid data.

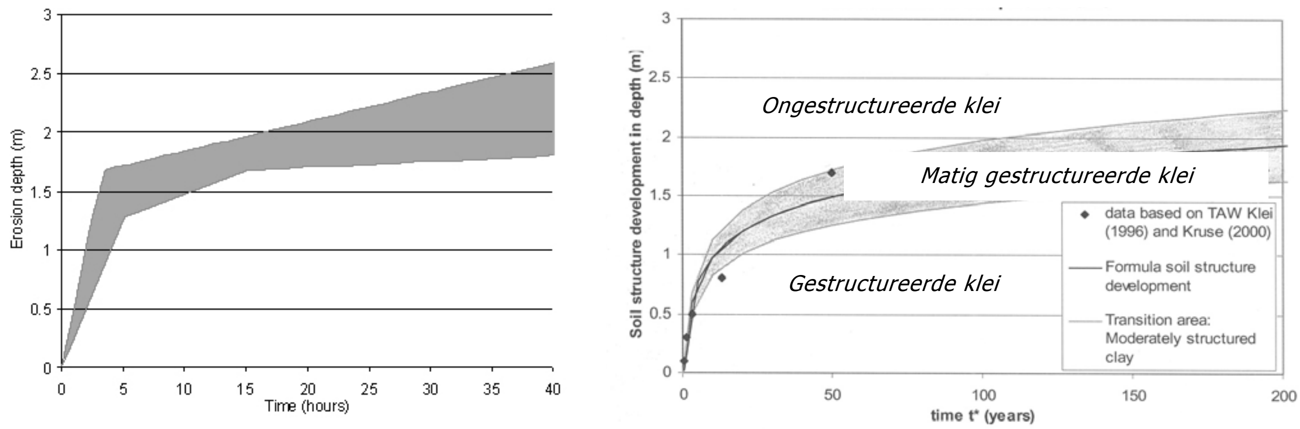
Het modelleren van het gedrag van klei onder golfbelasting

Een semi-kwantitatief model ter bepaling van de erosiediepte in de tijd is ontwikkeld in dit onderzoek. Zie Figuur 6, links. Dit model is gebaseerd op de onderverdeling van de experimenten op klei in drie verschillende kleicondities. Het semi-kwantitatieve model bestaat uit een combinatie van de lineaire ontwikkeling in de tijd van elke kleiconditie, met een gradiënt gebaseerd op de experimenten op klei. De overgang tussen de kleicondities is afhankelijk van de aanwezige bodemstructuur in de kleilaag. Dit kan worden bepaald met een in dit onderzoek ontwikkelde formule voor de bodemstructuurvorming in de tijd en in de diepte. Zie Figuur 6, rechts.

Op basis van de op dit moment beschikbare data van de grootschalige experimenten, is het niet mogelijk de invloed van de kleitype en golfkarakteristieken te kwantificeren. Alleen een benaderde invloed van de golfhoogte kan worden bepaald.
Het semi-kwantitatieve model is zo opgezet dat het kan worden uitgebreid als de factoren van invloed op het erosieproces van klei zijn gekwantificeerd.

Het semi-kwantitatieve model bepaalt bij benadering de gemiddelde erosie in de tijd in vergelijking met het empirische model van WL|Delft Hydraulics (2006b). Het empirische model bleek echter niet bruikbaar voor alle Deltagootexperimenten op klei.
De ontwerpmethode die is ontwikkelde door ingenieursbureau INFRAM (2003) geeft bij benadering tussen 3 uur en 10 uur minder erosie dan het semi-kwantitatieve model bepaalt. Voor een langere stormduur geeft de ontwerpmethode zoals verwacht meer erosie dan het semi-kwantitatieve model.

Semi-kwantitatief model *Ontwikkeling van een bodemstructuur in de tijd en in de diepte*



Figuur 6 Semi-kwantitatief model (links), ontwikkeld in dit onderzoek, ter bepaling van de erosiediepte van een kleilaag in de tijd onder golfbelasting. Dit model is gebaseerd op de bepaalde onderverdeling van de experimenten op klei in drie verschillende kleicondities en een opgestelde formule voor de ontwikkeling van een bodemstructuur in de tijd en de diepte (rechts).

Het gebruik van klei als bekleding voor zeedijken

Op grond van dit onderzoek kan worden geconcludeerd dat een dikke kleilaag mogelijk een milieuvriendelijke bekleding voor zeedijken kan vormen, maar dat het onderzoek tot nu toe nog niet voldoende is.

De erosie van een kleilaag in de tijd kan worden bepaald met het semi-kwantitatieve model dat is ontwikkeld als onderdeel van dit onderzoeksproject. Voor zover het onderzoek laat zien, moet de golfbelasting niet te hoog zijn. (Bij gebruikte data: H_s=1,5m.)
Een casestudie van Het Verdronken Land van Saeftinghe met het semi-kwantitatieve model laat zien dat een 100 jaar oude kleilaag bij benadering 0,8m-1,6m zal eroderen in 3 uur, 1,6m-2,2m in 10 uur en 2,0m-2,7m in 30 uur. Het semi-kwantitatieve model kan in de praktijk nog niet gebruikt worden, zonder verder onderzoek en verdere uitbreiding.
Zeker de verschillen tussen experimenten en waarnemingen na echte stormen moeten deel uit maken van vervolgonderzoek.

Bij de constructie van een kleibekleding moet aandacht worden besteedt aan het verkrijgen van een ongestructureerde kleiconditie, bijvoorbeeld door verdichten tijdens de uitvoering en het gebruik van klei met een laag zandpercentage.

De resultaten van dit onderzoeksproject zijn niet beperkt tot het gebruik bij tot zeedijken. Het is ook mogelijk om te onderzoeken of een dikke kleilaag kan worden toegepast op andere locaties, zoals bij rivierdijken.

Voor het echt in gebruik nemen van een kleilaag als bekleding van zeedijken, moet de haalbaarheid en duurzaamheid van deze alternatieve bekleding verder onderzocht worden op de aspecten technologische haalbaarheid, kosten, onderhoud, beschikbaarheid van materiaal, kansen voor natuurontwikkeling en het verkrijgen van een maatschappelijk draagvlak.

Table of contents

Part 1

Introduction and problem description

1 Introduction and problem description

1.1 General introduction

The Netherlands has a large tradition of living with water. This owes the country a great part of its prosperity, but gave also struggles because many flood disasters have passed. Till the Middle Ages people were living on the higher grounds, river embankments and artificial embankments ('terpen' and 'wierden').

Increasing population growth resulted in an intensifying use of lower lying areas, which were used for agriculture and the extraction of peat as a fuel. As a consequence, the surface subsided and these areas became more vulnerable to flooding. Therefore dikes were constructed, at first only as an addition to the height of existing higher lying areas. Intensive use of the lower areas and the development of drainage techniques resulted in subsidence of the surface level and the construction of more dikes.

Sea level rise and the subsidence of the ground, led to the fact that large parts of The Netherlands became lying under the sea level. In combination with an increasing population growth resulted this in increasing risks of flooding, which emphasizes the importance of the water defences, including the dikes. (Figure 1.1)

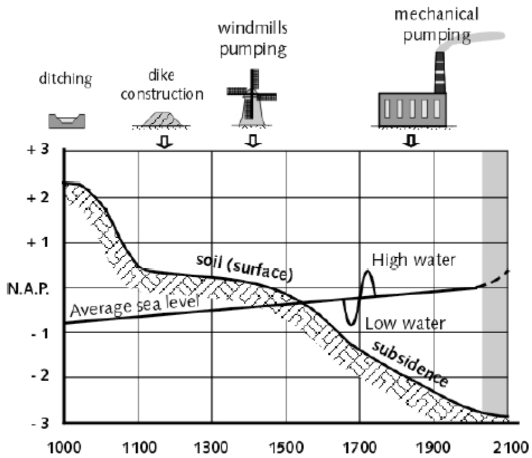


Figure 1.1 The subsidence of peat areas in the Netherlands [TAW GvW, 1998]

The vulnerability of The Netherlands was painfully emphasised during the storm surge disaster of February 1953, when the western part of the Netherlands was flooded due to dike breaches. This led to many deaths, casualties and damage. Before 1953, the height of the dikes was determined by the highest known water level and a certain safety margin. After a flooding a dike was raised, by making it a bit higher than the occurred water level. After the flooding of 1953, the question was raised how to deal with safety against flooding. Therefore in 1960, a different design philosophy was developed by the Delta Committee, with safety standards based on the probability of exceedance of a certain water level. See also Appendix I. [TAW GvW, 1998]

Environmental awareness
It becomes clear from history (see the previous section) that after a (near) flooding more political and public attention is paid to the reinforcement of the water defences. This logical reaction has in the last years been influenced by a growing environmental awareness and attention for sustainability on one side, and lack of space in The Netherlands on the other side. This resulted in finding so called 'Integral Solutions' in the safety approaches, combining the water retaining function of the water defences with other functions, like development of nature and recreation.

This process started during the development of the Oosterschelde barrier, a part of the Deltaworks. Originally a dam was planned to close the Oosterschelde, but in 1970s environmental protests arose

because the tidal movement and salt water would disappear. This would not only impact upon the ecosystem, but also the fishing industry and therefore an open barrier was chosen.
The Committee Boertien (report in 1993), which was installed to apply a safety approach for the river dikes, recommended after protest by environmental organisations, to pay attention to the loss of landscape, nature and cultural heritage (so called LNC-values).

European legislation of the last years prescribes governments to include environmental aspects in their national legislation. Environmental Impacts Assessments have to be performed for the development of large scale projects. These assessments require the development of a Most Environmental Friendly alternative and to look for compensating measures when nature is harmed. The Compensation of Nature Principle requires preventing or mitigating the loss of important natural values. This implies for infrastructural projects that nature, which disappears due to the project, has to be developed in another area.
This is a current issue in Zeeland, where the deepening of the Westerschelde needs to be compensated. This can be done by many small projects, for example stimulation of the development of natural areas like salt marshes on foreshores or nesting areas for birds. (See Figure 1.2) [Project ProSes: www.proses.nl and www.scheldenet.nl]

The Birds and Habitats Directive implies adaptations in the execution of the reinforcement of the sea dikes, because in the brood and migration periods no construction work is allowed in certain areas. Also for Water Boards this requires adapted maintenance of the sea dikes.

This leads however also to conflicting opinions. For decades The Netherlands fought water by poldering of land and reinforcing dikes. Due to an increase in environmental awareness, land is given back to the sea. These measures result in lack of understanding and confidence by local inhabitants of coastal areas, especially in Zeeland, where the flood disaster of 1953 had its biggest impact.

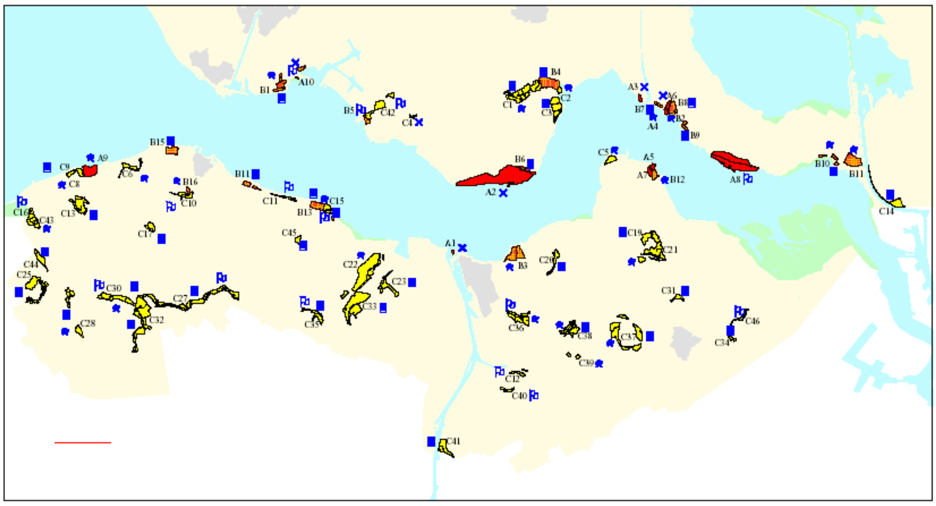


Figure 1.2 Locations of compensation of nature due to the deepening of the Western Scheldt. [www.scheldenet.nl, July 2005]

Improvements of sea dikes
A part of the Dutch coast and estuaries are protected by sea dikes, which form a part of the primary water defence system of the Netherlands. Besides this primary function of water defence they fulfil, sometimes unintentional, also other functions: as landscape, nature and cultural heritage, recreation, agriculture and traffic. [TAW ErG, 1998]

The core of a dike is made of sand and/or clay. After the storm surge disaster of 1953 many dike sections, also the ones that did not breach, needed repair and reinforcement because they were too low or too steep. This asked for large amounts of material which were not all present in the neighbourhood. Therefore concessions were made and sand was used as core material, covered by a clay layer to make the dike impermeable and to prevent erosion. On the outer slopes a 'solid' revetment consisting of stones is generally applied to protect the dike against wave attack.

Sea level rise and new insights on the hydraulic loads on the coast, like increasing storm frequencies and larger wave periods, imply heavier loads on the water defences as was known so far. Parts of the revetments of the dikes do not fulfil the current safety demands and need therefore strengthening. Attention is also paid to the LNC-values and recreation, which gives the opportunity for integral solutions.

For the dikes along the estuaries where a high foreland with natural areas of salt marshes is present, is looked at alternatives to the traditional stone revetments. From a natural and aesthetic point of view a slope covered with vegetation is preferred over a stone revetment, because in this way a more gradual transition between the salt marshes and the dike with the hinterland is obtained. A dike with a revetment which obtains its strength from the grass cover or from the clay cover are possible alternatives.

In the report 'Kustrijke Kans' (GeoDelft, 2003) is referred to these alternatives as so called 'grass dikes', which obtain their strength from the grass and so called 'clay dunes', which obtain their strength from the clay layer.

It is difficult to obtain a good quality grass cover along the coast due to the presence of swash mark, salt water and irregularities in the sod.
The use of a thick clay layer as an alternative revetment for sea dikes in the presence of a high foreland is investigated in this research. The high foreland can give a reduction of the wave loads on the slope, because the waves break earlier.

1.2 Present projects

The development of visions and knowledge of living with water in The Netherlands, protection against flooding and environmental awareness, are visible in current projects concerning flood protection. Selected projects concerning the European coasts are given below, including one project concerning rivers because the development of a different viewpoint by the government to flood protection can be clearly seen here.

'Ruimte voor de Rivier'
The project 'Ruimte voor de Rivier'(literally 'Space for the River') is not concerned with the coast, but gives an example of the different viewpoint of how The Netherlands deals with water.

In 2000 the project Ruimte voor de Rivier was initiated by the Dutch government as a starting point for a new sustainable approach for managing high water levels in the river areas. During the flood waves of 1993 and 1995 several dike sections were close to breaching and in 1995 inhabitants were evacuated. The 1995 Major River Delta Act ('Deltaplan Grote Rivieren') made rapid reinforcement of the dikes possible, due to faster legal procedures, but more measures are needed. Climate change will lead to higher river discharges which lead, together with subsidence of the surface level, to heavier loads on the water defence structures.

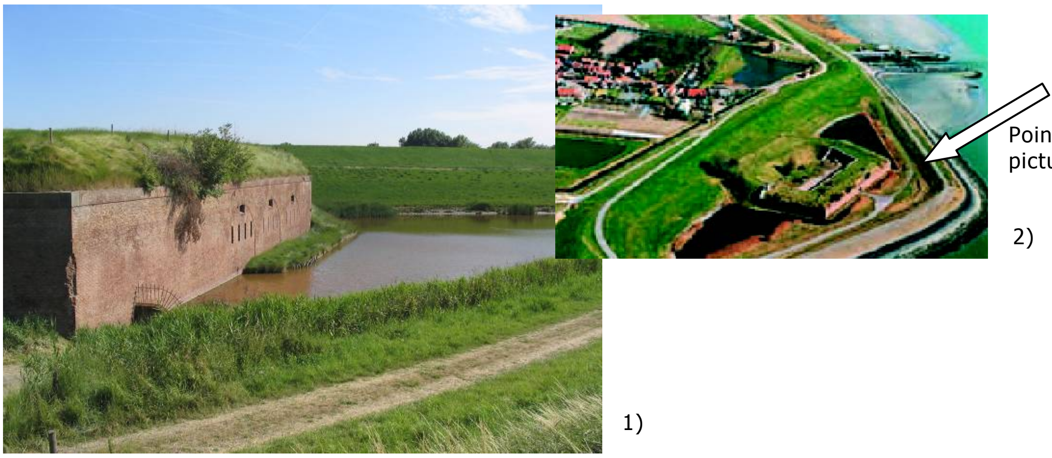
Instead of reinforcing the dikes and raising the crest level, possibilities are investigated in which the water receive more space during high discharges. This is accompanied with possibilities for economical, ecological and spatial improvements of the river areas. This will therefore result in a more integral approach of combining the discharge of water with for example urban development, natural development or recreation. Possible measures are the deepening of the river forelands, moving dikes further inland or designating temporary overflow areas during high water levels. [www.ruimtevoorderivier.nl]

ComCoast
ComCoast, COMbined functions in COASTal defence zones, is a European project of the North Sea countries Belgium, Germany, Denmark, United Kingdom and The Netherlands. It started in April 2004 aims to cope with the increasing loads on the present flood defences, especially sea dikes, in an innovative way. The concept is to develop multifunctional water defence zones, without the further traditional raising of dike crest levels, but by developing a gradual transition from sea to land. It includes the spatial adaptation of adjoining hinterlands which have to store the excess water that occasionally overtop the primary sea dikes. This must be done in an multifunctional way to offer high-value activities in the water defence zones and provide a sustainable and economical solution.

The project involves different technical research subjects:
- The increase of wave overtopping of the sea dikes, by strengthening of the inner slope and improving the flood retention capacity of the hinterland areas.
- Improvement of the wave breaking effect of the foreshores.
- Creating salty wetland conditions with tidal exchange in the primary sea defence (by culvert constructions or realigning of the coastal defence system)
- The increase of salt intrusion.

Besides these technical aspects, also public and political support needs to be gained.

Different pilot projects act as the visualisation of the ComCoast concept and are used for monitoring. See for example Figure 1.3 for pictures of the pilot Fort Ellewoutsdijk in Zuid-Beveland, Zeeland, where a water defence zone is obtained by two dikes with a monumental fort in between. [www.comcoast.org; ComCoast, 2000]


Point of view picture 1)

2)

1)

Figure 1.3 ComCoast pilot Fort Ellewoutsdijk in Zuid-Beveland, Zeeland. [2) www.comcoast.org, June 2005]

FLOODsite

FLOODsite is a European project on flood risk management supported by the European Commission (under the Sixth Framework Programme, 2002-2006). It is an integrated project and covers the physical, environmental, ecological and socio-economical aspects of floods from rivers, estuaries and the sea. The individual factors related to flooding are reasonably well understood. However, to come to real flood mitigation the complex interactions between the different factors has be known. Researchers and practitioners from governmental, commercial and research organisations work on different tasks on the project.

The goal of the project is to deliver an integrated European methodology for flood risk analysis and management, with techniques and knowledge to support this. Thus sharing of knowledge is one of the key-points. Pilot applications of river, estuary and coastal sites are used to analyse the theoretical methods.
[www.floodsite.net]

Building projects

The opportunities of combining different functions in the water retaining zone are also seized by building companies. Projects are developed where houses were adapted to a regular water inconvenience, like floating, or floatable houses in river forelands. Also research is performed to multifunctional flood defences for cities, where retaining of water is combined with living.
[www.waterbouw.tudelft.nl/public/stalenberg]

Conclusion

From this paragraph it becomes clear that more knowledge concerning flood defences and the environment has led to the development of a more integral approach, where safety requirements are combined with environmental conservation, environmental development and other functions in the water retaining zone. This development is sometimes legally supported by governments of European and national level.

1.3 Sustainability

1.3.1 Sustainable development

Definition

The definition of sustainable development is according to the 'Brundtland Report', 'Our Common Future', the report of the World Commission on Environment and Development:
'Sustainable development is development that meets the needs of the present generation without compromising the ability of the future generations to meet their own needs.' (WCED, 1987:8)

This means that the people living today should take into account the future consequences of their actions and decisions on both the environment and the needs of the people living in the future.

Sustainable development tries to balance and combine ecological, economical and socio-cultural values. It focuses thus not solely on environmental issues. This balance is often represented by the three P's, the three pillars of sustainable development: people, planet and profit, see Figure 1.4. These three P's should be in balance as much as possible to obtain a sustainable world. . If this combination is not in balance, one or two of the elements will suffer from the other one(s).

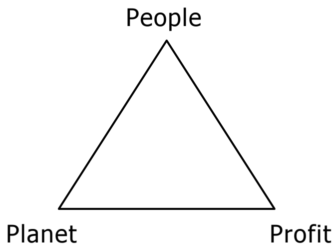


Figure 1.4 People, Planet, Profit, the three pillars of sustainable development.

Sometimes a fourth pillar, politics, is added. Especially for large scale projects, where different parties are involved, with conflicting opinions, politics must not be underestimated.

1.3.2 Sustainable development and civil engineering

Sustainable development in a civil engineering context can be implemented on different levels and aspects of designing:

- Way of approaching the problem and involving influencing factors
  Keeping in mind the balance between all factors (people, planet, profit) during problem analysis and design and trying to focus on a long term solution.
- Type of solution chosen
  Keeping in mind the balance between all factors (people, planet, profit) during problem analysis and design and focussing on a long term solution. Environmental impact assessments can be involved as instruments in the design phase.
- Use of materials
  The type of materials, the source of supply and manner of transport all have an influence on the environment.
- Manner of construction
  The execution and building of a construction gives a load on the environment: emission of harmful gases due to fuel use and damage to the local area around the building site which disturbs nature temporary or permanently. These effects should be kept in mind during the executing work. It is not possible to prevent all these effects, but being aware of the situation can sometimes result in a different approach.
- Management
  Keeping in mind the balance between all factors (people, planet, profit) during problem analysis and design and focussing on a long term solution.

Integrated Coastal Zone Management

Sustainable development focussed on coasts: Integrated Coastal Zone Management

During the United Nations Conference on Environment and Developments of 1992, Integrated Coastal Zone Management (ICZM) was confirmed to be the recommended approach to planning and decision making for marine and coastal areas. This means that "each coastal state should consider establishing appropriate coordinating mechanisms for integrated management and sustainable development of coastal and marine areas and their resources at both local and national level."(United Nations Conference on Environment and Developments, 1992, Brasil, Chapter 17, Agenda 21.)

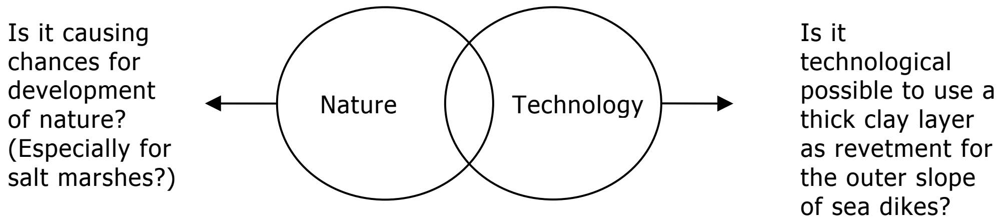
ICZM wants to balance the benefits from environmental, economic, social, cultural and recreational objectives over the long term, within the limits set by the natural dynamics. The goal of ICZM is to improve the quality of life of human communities who depend on coastal resources while maintaining the biological diversity and productivity of coastal ecosystems. 'Integrated' means integration of objectives and all instruments needed to meet these objectives on all policy areas, sectors and levels of administration. This results in a dynamic balance between people and their activities on one hand and the state of the coastal environment on the other hand.
[Hoozemans, 1995]

1.3.3 Sustainable development and this study

In this study an alternative revetment for the outer slope of sea dikes is researched. This consists of a thick clay layer which allows by the possibility to vegetation growth, a better connection of the natural salt marsh area with the dike and hinterland. If this alternative has sufficient strength and stability to form a revetment, and this desired vegetation growth can be obtained, this thick clay layer could form a sustainable revetment if it is compared with traditional revetments on other aspects as well, like material and maintenance. This is elaborated in Chapter 12.

The subject of this research, applying a clay cover layer as dike cover layer on sea dikes, deals in general with two fields, technology and nature. The technological aspect is the question is if it is possible to use a thick clay layer as revetment for sea dikes and if this has sufficient strength and stability to withstand a normative load. The natural aspect for this research is the question if this type of cover layer develops chances for development of nature, especially for salt marshes.

It is possible to state that this type of cover layer is a good solution if only the technological aspect is confirmed, without any attention to the natural aspect. However, the natural aspect can, if it is confirmed, give an extra valuable property to this type of revetment.



Next to this also the availability of the material, maintenance, costs and social basis are aspects of consideration for obtaining a feasible and sustainable solution. These aspects are elaborated in elaborated in Chapter 12.

The focus of this study is on the technological aspect.

1.4 Problem analysis

1.4.1 Problem description

For the improvement of dikes along the estuaries where a high foreland with natural areas of salt marshes is present is looked at alternatives to the traditional stone revetments. From a natural and aesthetic point of view a slope covered with vegetation is preferred over a stone revetment, because in this way a more gradual transition between the salt marshes and the dike with the hinterland is obtained.

Revetments which obtain their strength from the grass cover or the clay layer are possible alternatives for the traditional stone revetments. However, it is difficult to get a good quality grass cover along the coast due to the presence of swash mark, salt water and irregularities in the sod. The use of a thick clay layer is an interesting subject of research.

Clay cover layers as a revetment for sea dikes are occasionally used in Zeeuws Vlaanderen, Zeeland, The Netherlands, if a high foreland is present in front of the dike. The concept of a clay revetment was monitored for a number of years at two tests sections at the Verdronken Land van Saeftinghe, where a natural salt marsh area is located in front of the dike. The high foreland can reduce the wave load on the slope, because the waves break earlier. During the test period no erosion was observed and the vegetation in the clay was well developed. However, no storms occurred during the monitoring. Based on the monitoring, Rijkswaterstaat considers the use of a clay layer to be a suitable revetment for sea dikes, if a high foreland is present in front of the dike, and the significant wave height is lower than 2m. This alternative revetment is therefore occasionally applied at other locations in Zeeuws Vlaanderen where a high foreland is present.

A design method for clay revetments was developed by engineering firm INFRAM (2003), which formed the basis of the current design manual used by Projectbureau Zeeweringen. Both methods are however considered to be unsure and there is still little knowledge on the behaviour of clay under wave loading. Therefore more research is necessary into the behaviour of clay under wave loading and the influencing factors on this behaviour.

1.4.2 Problem definition

General

Would it be possible for thick clay layer, in the presence of a high foreland, to form a sustainable and feasible revetment with sufficient strength and stability to withstand a normative load (a design storm)? What would be the requirements for the application of this clay cover layer?

From a literature study is concluded there is still little knowledge on the behaviour of a clay layer under the influence of wave loading. Therefore the general question is adjusted to the more specific research question:

Specific

What is the behaviour of a clay layer under wave loading and what are the influencing factors on this behaviour?

1.4.3 Objectives

- Analysing the use of clay as a cover layer material on dikes.
- Gaining insight into the behaviour of a clay layer under wave loading.
- Making a start with the description of the failure process and damage development of a revetment consisting of a thick clay layer.
- Making a start with the analysis if a clay cover layer can form a sustainable and feasible revetment.

1.4.4 Limitations

The focus of this research is on:
- The outer slope of dikes.
- The erosion of clay under wave loading, with wave loading as the main load. Erosion by currents will not be investigated in detail, but can not be neglected.
- The failure mechanism of erosion of the outer slope, but the other failure mechanisms can not be completely left out of consideration. Especially sliding of the slope can occur if slopes of saturated dikes are steep.
- The behaviour of the clay layer. A possible vegetation layer can deliver additional strength, but will not be investigated in detail in this research.

1.5 Approach of the research and structure of the report

Approach of the research
- Initial study to revetments, clay, grass, maintenance, clay dikes in Zeeuws Vlaanderen and the design methods of clay layers on dikes.

- An initial literature study was performed on the behaviour of cohesive material under wave loading: small scale and large scale experiments on the residual strength of clay, and the accompanying conclusions, analysis and modelling, research to cohesive river banks, scour of cohesive material, research in Germany.
  Little accurate knowledge on the behaviour of clay under wave loading appeared to be present. However assumptions on this behaviour have several times been made in recent history.

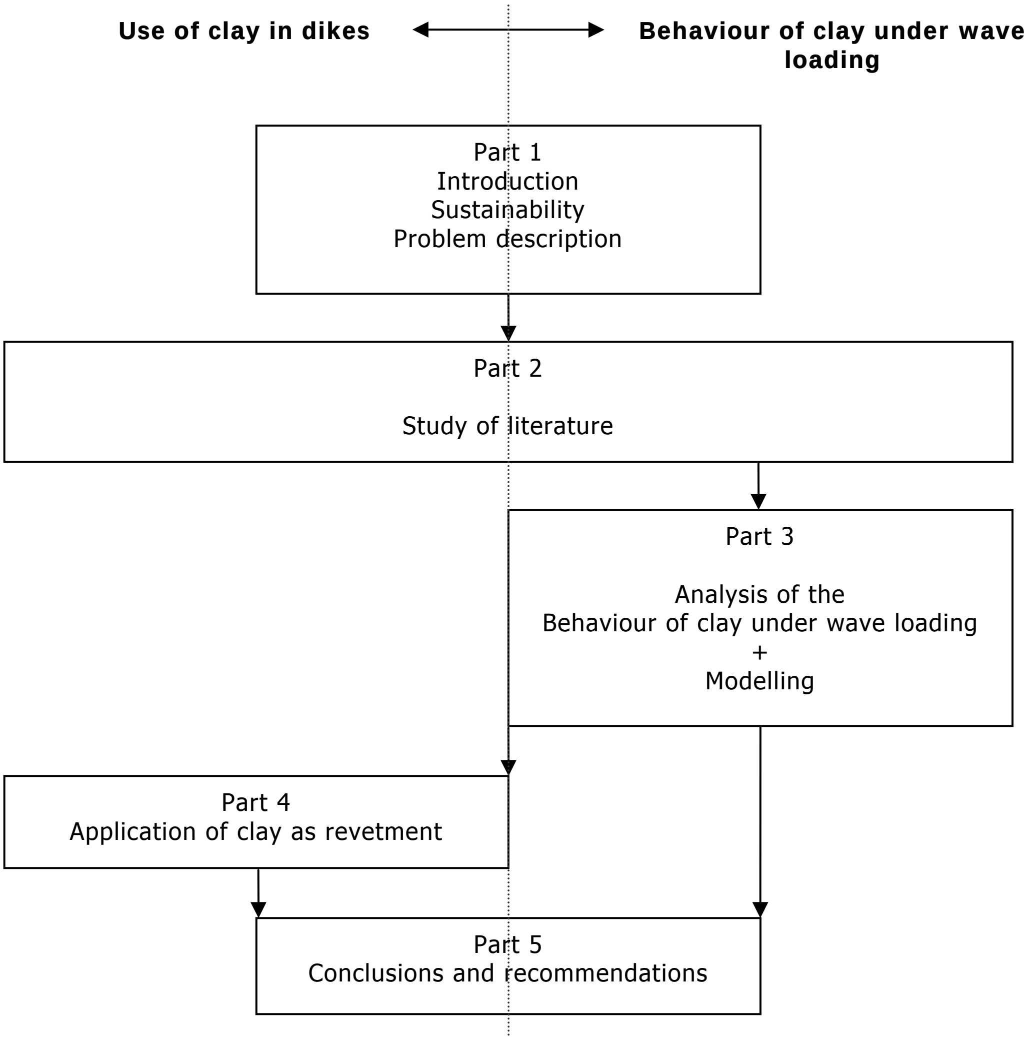
- All large scale experiments performed on clay under wave loading are investigated and compared to each other. This was done in order to avoid making new assumptions on the behaviour of clay under wave loading.
  This resulted into a complete overview of all relevant large scale experiments, which is used to verify if a comparison of these experiments results in a broader view on the behaviour of clay under wave loading. The results are analysed, new conclusions are drawn and existing conclusions and assumptions are validated.

- A semi quantitative model to determine the erosion depth over time is developed in this research. On basis of the available data it was however not possible to quantify the influencing parameters on the erosion process.

- In order to obtain more insight into the behaviour of clay under wave loading, comparisons with the erosion behaviour of sand and observations of outer slope damage in actual storms are made. This gives additional conclusions and a validation of conclusions obtained from the experiments.

- A start with research on the use of clay in dikes is made, by means of a Case Study for Het Verdronken Land van Saeftinghe with the semi quantitative model and by defining requirements for the use of a clay layer as revetment for sea dikes.

Structure of the report
The report is divided in several parts, as given in the following schematization.

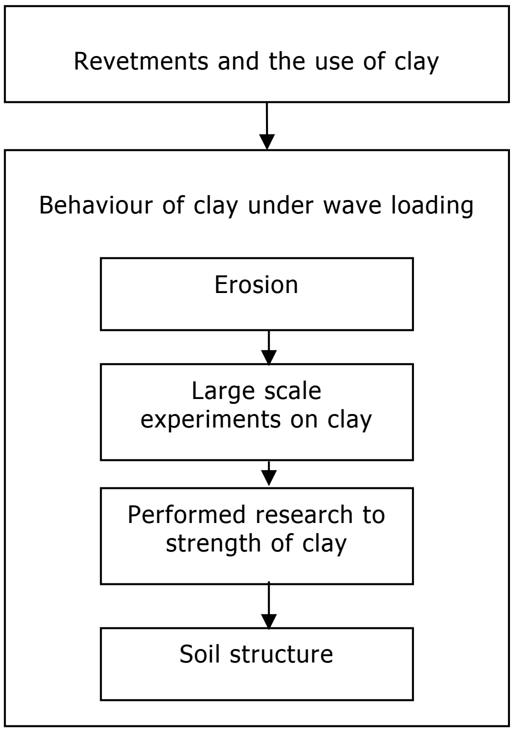

Use of clay in dikes
Behaviour of clay under wave loading
Part 1
Introduction
Sustainability
Problem description
Part 2
Study of literature
Part 3
Analysis of the
Behaviour of clay under wave loading
+
Modelling
Part 4
Application of clay as revetment
Part 5
Conclusions and recommendations

Part 2

Study of literature

This part of the report contains a short overview of background information on revetments and the present use of clay in dikes. The major part consists of relevant background information on the behaviour of clay under wave loading. This information forms the basis of the analysis of this behaviour, given in Part 3.

Part 2
Study of literature



Clay types

In the different experiments a distinction is made between different kinds of clay. Often is referred to these differences with the term 'quality':

High quality clay is category 1 clay, erosion resistant clay according to the requirements from 'Clay for Dikes' (TAW Klei, 1996). This is clay with a low sand percentage (<40%) and a high flow limit (>45%). In Dutch also referred to as 'vette klei'.

Moderate quality clay is category 2 clay, moderate erosion resistant according to the requirements of 'Clay for Dikes'. This clay has a low sand percentage (<40%) and a low flow limit (>45%).

Low quality clay is category 3 clay, little erosion resistant clay according to the requirements from 'Clay for Dikes'. This is clay with a high sand percentage (>40%). In Dutch also called 'schrale klei'.

Because the term 'quality' can be explained in different ways, other terms should be used to make the distinction between the different clay types more clear. For the moment the following terms will be used in this report:

Clay types

1. Clay with a low sand percentage
2. Clay with a moderately high sand percentage
3. Clay with a high sand percentage

Table 0.1 Requirements for use of clay in dikes.

Requirement	Erosion-resistance category		
	1 Erosion-resistant	2 Moderately erosion-resistant	3 Little erosion-resistance
Flow limit (w_l) Plasticity index (I_p) Sand content	> 45% and > 0.73*(w_l-20) and < 40%	< 45% and > 18 and < 40%	< 0.73*(wl-20)% and < 18 and > 40%
Organic material content	< 5%	< 5%	< 5%
Salt content	< 4g/l	< 4g/l	< 4g/l
Chalk content	< 25%	< 25%	< 25%
Water content for working Top layer Core	 $I_c \geq 0.75$ $I_c \geq 0.60$	 $I_c \geq 0.75$ $I_c \geq 0.60$	 $I_c \geq 0.75$ $I_c \geq 0.60$
Extreme colouring from excavation or dying	Not allowed	Not allowed	Not allowed
Strong smell	Not allowed	Not allowed	Not allowed

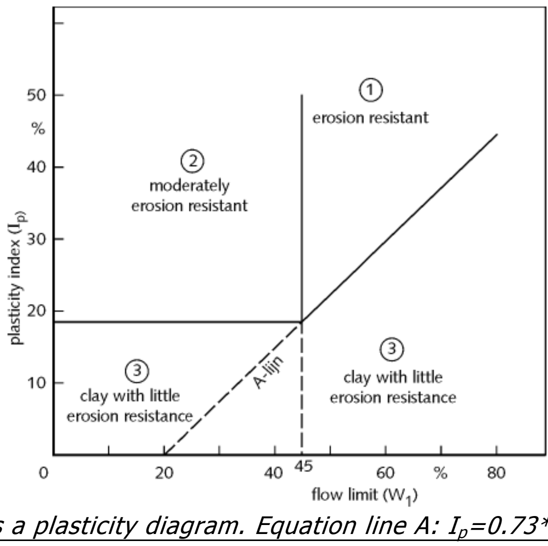


Figure 0.1 Erodibility as a plasticity diagram. Equation line A: I_p=0.73(wl-20). [TAW Klei, 1996]*

2 Revetments and the use of clay

2.1 Dike revetments in The Netherlands

2.1.1 Use of revetments for dikes through history

The history of dike construction starts in the Middle Ages. At first people in the Netherlands lived on the higher lands or artificial embankments ('terpen' and 'wierden') just above the highest known water level. The first dikes were constructed of compressed eelgrass or peat to protect the living areas. These dikes had to be raised continuously because of settling of the peat. Around 1030 in Flanders and 1105 in Holland, dikes along the coast were constructed to protect areas from flooding by salt water. These dikes were placed on the marine clay and were also made out of this clay. Around the 'Zuiderzee', nowadays the 'IJsselmeer' many kelp dikes were built. Layers of kelp were constructed and kept together by timber piles. Small stones were sometimes placed in front of this dike construction for additional protection, see Figure 2.1. At first these dikes have only been built around one or more farms with their fields, later in the tenth century larger dikes were constructed to protect several villages.

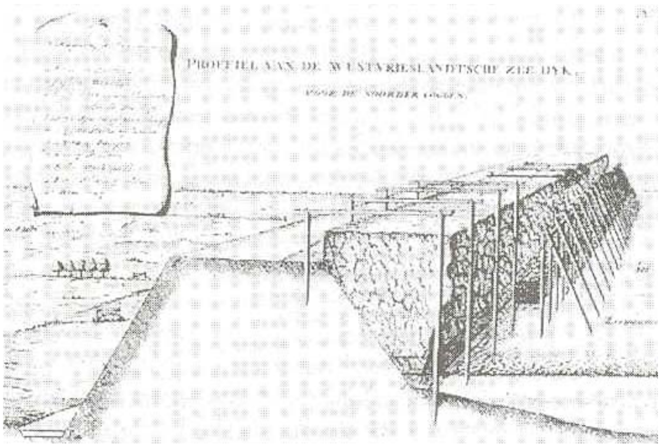


Figure 2.1 The construction of a kelp dike. A rectangular shaped mound of kelp is supported by wooden piles. The kelp is partly covered by stones. [Bijker, 1996]

Important sea dikes in the Netherlands like the 'Westkapelse Zeedijk' (15th century) and the 'Hondsbossche Zeewering' (18th century) were constructed because the dunes were retreating too much and villages lying behind the dunes needed protection. In 'Westkapelle' for example the inhabitants worked for centuries on the maintenance and construction of the dike.

Until the 16th century stones were only very limited available and therefore hardly used in dike building. The dikes were strengthened by rows of piles on the sea-side. Between the dike and these piles pelt (seaweed) or reed was applied. This resulted in a steep wall on the sea side which functioned to break the waves. Especially in Flanders stones were used to form a protective layer on top of the dike to break the wave impact. Also wooden groynes were constructed perpendicular to dikes to reduce the wave impact. In some cases they were connected by boards and sometimes it was even possible to moor ships there during high water.

In the 18th century however ship worms, imported from Asia, affected the constructed wooden sea defences along the coast. All bulkheads ('beschoeiingen'), piles and osier wood ('rijshout') decreased in strength or were destroyed by the water. Secondary dikes ('slaperdijken') were constructed for safety behind the sea defences.

The pile defences were replaced by an earth dike of clay with a slope, covered with stones till the water line and with rows of piles ('staketwerk') above the water line to protect the dike against wave attack. Mattresses of reed or osier wood ('krammatten') were sometimes connected to the piles or the compartments between the piles were filled with basalt, see Figure 2.2. These basalt pillars were placed on a foundation of clay and hardcore. For the defence of beaches and dunes ‘staketwerk’ was also often applied. The earth dikes were supposed to be constructed of clay, but often the constructor placed peat, garbage, straw, dead animals or tons in the dike body too.

Figure 2.2 Left: Men making mattresses of osier wood (‘krammatten’) and right: Rows of piles with basalt in between (‘staketwerk’).

In the beginning of the 19th century plans were made to use stones from Drente or Norway to protect the sea defences, but these were too expensive. After the flooding of 1825 and more damage due to ship worms, dikes with an oblique slope and stones were constructed. At critical places basalt was placed to strengthen the dike.

In the beginning of the 20th century experts investigated the quality of the dikes. They concluded that the dikes were almost everywhere too low and not wide enough. The government however did not pay enough attention to this problem and not enough money was available to strengthen of the dikes.

Ir. R.R.L. de Muralt, head of the technical service of Water Board Zeeland from 1903-1913, came with an inexpensive method to raise the dikes. He suggested to raise the existing dikes by placing a concrete wall of several decimetres on top of the dikes. Widening of the dikes was not necessary anymore. This wall consisted of three or four concrete slabs placed between concrete piles. Between 1906 and 1935 many sea dikes in Zeeland, around 120 km, were heightened by his system of so called Muralt-walls. However, during the storm surge disaster of February 1953 the walls were no good solution and they were not used again since then.

Ir. de Muralt also invented a system of concrete revetments which was cheaper than the used basalt revetments. This system withstood the storm surges of 1906 and 1911 almost without damage.

In 1932 the Zuiderzee was closed by constructing the ‘Afsluitdijk’. Clay was still the most important dike construction material. For the closure of the Zuiderzee boulder clay was used, which is a material deposited long ago by glaciers.

During the Second World War no money was available for strengthening the dikes, however the Stormvloedcommissie investigated the dikes again, and again was concluded that the dikes were insufficient by width and height. After the flooding of 1953 a many dikes sections needed repair and reinforcement. Large amounts of material were needed, which were not all present in the neighbourhood. Therefore concessions were made and sand was used as core material instead of clay, covered by a clay layer to make the dike impermeable and to prevent erosion. On the outer slopes a revetment of often stones was applied to protect the dike against wave attack. The old clay dikes were supplemented with sand and have therefore a core of partly clay and partly sand.

[Bijker, 1996; Fergusson, 1976; Muralt, 1931; TAW LZM, 1999; www.deltawerken.com; www.geschieniszeeland.nl]

Conclusion
Thus from the start of dike construction the outer slopes were protected from eroding, first with piles and kelp, later with stones which were available in limited quantities. Clay was the most common dike construction material for centuries.

2.1.2 Maintanance

Every five year the primary water defences dikes need to be evaluated according to the criteria in the Voorschrift Toetsen op Veiligheid (TAW VTV, 2004). The dikes are tested on the different failure mechanisms to investigate if the dike satisfies the legal safety demands. On the basis of these results is determined if maintenance or improvement is necessary. Also new knowledge on increasing loads on the water defences is taken into account. Maintenance concerns a limited amount of measures, for which a formal plan procedure, an environmental impact assessment, is not necessary. For improvements an environmental impact assessment is compulsory, article 7 of the Flood Defence Act is applicable. The manager has to make a plan of the design which has to be approved by the Provincial Government. LNC-values and social functions need to be taken into account. [TAW VTV, 2004]

Projectbureau Zeeweringen is working on improvements of the dikes along the estuaries of Zeeland at the moment, because the stone revetments which are present on a large number of dikes did not satisfy with the safety demands. During the construction of the improved revetments, LNC values of the dike are taken into account. See Figure 2.3 for pictures of the execution works. Because of safety reasons (the storm season) the work takes only place in a part of the year, from April 1st till October 1st. In this period also brood- and migration periods of birds need to be taken into account, according to the Birds- and Habitat Directive. [www.zeeweringen.nl]

Figure 2.3 Execution of dike reinforcements of Projectbureau Zeeweringen in Zuid-Beveland, Zeeland.

2.2 Failure

There is a distinction to be made between failure and collapse of a structure. A structure fails if one or more of its functions are not fulfilled. Collapse of the structure takes place if there is loss of coherence or large scale change in geometry. A flood defence can fail by collapsing and vice versa. The primary function of a flood defence structure is the water retaining capacity, with high water levels and waves as the main threats. If a structure collapses, failure of this safety function will obviously occur if repair is not done quickly. Therefore collapse is generally covered by the term failure. Initial failure of a part of the flood defence does not have to lead to collapse of the structure, because a certain residual strength will be present. The residual strength can be considerable, but in practice this is most of the times not taken into account. [TAW GvW, 1998]

The different ways in which the water retaining capacity can be lacking are called the failure mechanisms. See Figure 2.4 for an overview. For the judgment of a water retaining structure however, the coherence between these different mechanisms is of great importance. [TAW GvW, 1998]

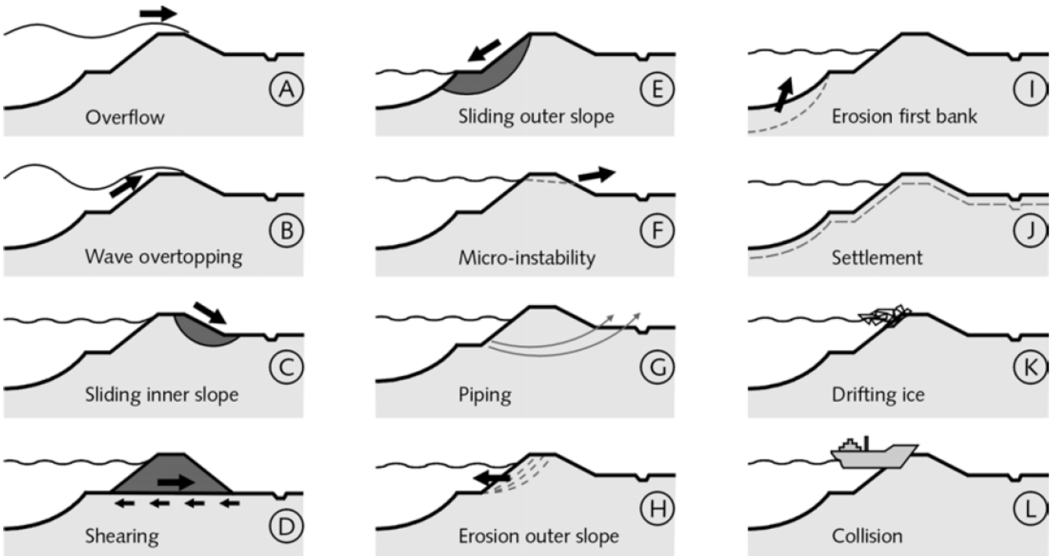


Figure 2.4 Failure mechanisms of soil structures. [TAW GvW, 1998]

2.3 Grass covers

This research focuses primarily on the use of clay as material for dike cover layers, but because vegetation will always develop on a clay layer, some research was also performed on grass as cover layers for dikes. In the North of Germany and Denmark many grass covers are used instead of stone covers. This is accompanied by a gentle outer slope and a wide foreland.

2.3.1 The use of grass as dike cover material

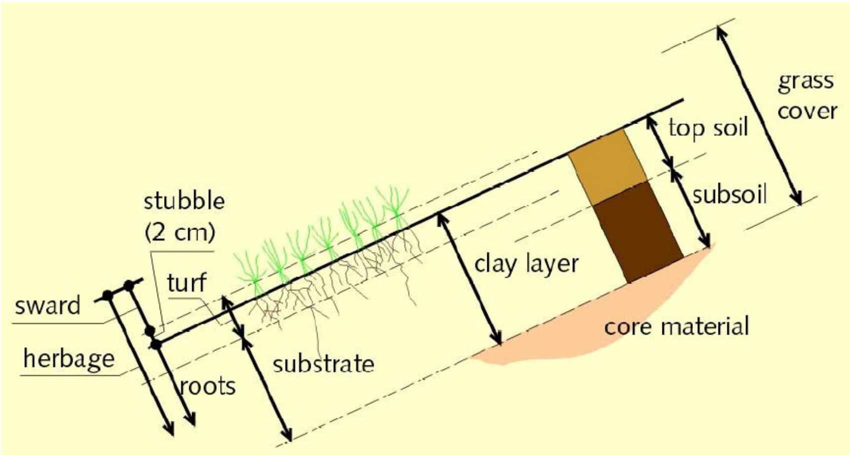


Figure 2.5 Structure of grass cover. [TAW ErG, 1998]
(Dimensions are not given)

A well developed grass cover is highly erosion resistant. This erosion resistance is obtained by the structure of the root system. The roots are responsible for the development of a soil structure of aggregates and cracks in the soil, but they also support the development of cementing substances. Chemical processes in the vicinity of the roots develop into cementing substances which stick the soil particles so to say together. This results in an elastic network which provides a strong and flexible layer which can deform without cracking. The erosion resistance of a grass cover is to a large extent based on this. Because of the flexibility the cover is able to resist wave impacts and the root system prevents washing away of the soil particles. The grass leafs contribute to the resistance against flowing water and has therefore a function on the inner slope during the wave overtopping. The development of a good quality grass cover with a well developed root system takes several seasons, on average 4 years.

To guarantee the erosion resistance the soil, as well as the grass cover, has to fulfil to several minimum requirements. The top layer of the soil (0.15m) develops different properties than the deeper lying layers because of soil structure development. Due to this soil structure development an erosion resistant root system is able to develop in this layer. This requires clay with a high sand percentage, because in this roots can develop faster and the root structure has a larger density than in clay with a low sand percentage.

The vertical structure of a well rooted grass cover is schematically composed as given in Table 2.1, starting at the top layer. The specific position of the different layers dependents on the soil type and the age of the vegetation.

Table 2.1 Vertical structure of a grass cover. [TAW ErG, 1998]

Layer	Thickness layer	Composition soil	Erodibility
1 (top)	1-3 mm	Loose soil and remains of vegetation.	Easily eroded by waves.
2	5-50 mm	Loose turf, densely rooted.	Slow erosion by waves.
3	5- 50 cm	Less loose turf, less roots.	Erosion after long-lasting wave load.
Deeper into the soil		Packing of the soil increases and the rooting decreases.	

Since 1990 research to grass as dike cover material has been done (in the so called 'Grasplan'). Different experiments have been performed to the erodibility of grass, but there are still many uncertainties. Results of modelling and an overview of the performed experiments are given in Appendix II and Appendix III.

In practice, obtaining a good quality grass cover appears to be very difficult.

[Sprangers, 1996; TAW ErG, 1998; TAW Grasmat, 1999]

2.3.2 Grass dikes in North-Germany and Denmark

Along the North Sea coasts of North-Germany and Denmark, several green sea dikes are located. Here the wave loading hits, in contrast with the Netherlands, the grass covers. There are several differences in comparison with the Netherlands in the foreland, dike profile, slope, clay composition and applied management.

History of coastal management
The development of dike building is comparable with the Netherlands. After the flood disaster of 1953 in the Netherlands also in these areas research was performed to the quality of the coastal defence zone. From this appeared that the demands on safety and risk reduction were not fulfilled. After the flood disaster of 1962, which led to great damage in the German Wadden area, in Germany a General Plan for dike reinforcement (seawards extension with sand core and gentle slopes), dike shortening and coastal protection was formulated. The probability of exceeding of the floods was lowered to a chance of 1:60 till 1:100 per year, instead of 1:20 per year. The responsibilities for the management of the water retaining dikes were moved to the federal state government, through which the fragmented management was stopped. The General Plan was actualised in 1977 in which minimum profiles for the dikes were determined. In 1985 the Wadden Sea was appointed to national park from 150m of the outer toe, through which next of the protection of the coastal areas, also the present natural values have to be taken into account.

Green dikes
In most situations there is an extensive foreland (minimum width of 400m) present in form of the dikes, which developed over the years due to accretion above the average water level. The water therefore reaches the dike only during storms, on average 20 times per year. The grass dike fluently changes into the foreland. If a foreland is not present, or the channel pattern runs along the toe of the dike, a rubble layer is placed in the wave loading zone at mean high water. Above this level a maintenance road is located, which also provides protection against overflowing water. Above this road the slope is covered by a grass layer.

Reasons for the construction of dikes with grass covers are:
- The presence of extensive forelands with heights till above mean high water. These are preserved by land reclamation and on behalf of coastal protection.
- The subsoil with low bearing capacity. Instead of soil improvements is chosen for spreading of the loads by the construction of low, wide dikes because of costs.

A cost comparison with steep, rigid dikes has never been made, because of the strong preference for green dikes.

By nature a sharp zone (0.5-1m) develops between salt and fresh vegetation at 0.7m above MHW. In the fresh part of the slope the vegetation is sowed. For the salt part, sods are obtained from the foreland and placed on the dike slope (Thickness sods 3-4 cm).

The dimensions of the cross-shore profile which are used as basic principles in the General Plan are given in Table 2.2.

Table 2.2 Indication of the cross-sections of sea dikes in North-Germany and Denmark.

Cross-sectional profile	Basic principle of General Plan (Perhaps meanwhile changed, but many dikes will still have these dimensions)
Crest level	7.5-8.5m + N.N. (With a change of flooding of 1% per year.)
Crest width	2.5-3.0m
Outer slope	1:6 – 1:8 – 1:10 till 1:20
Inner slope	1:3
Thickness clay layer (core of sand)	Inner slope 0.5-0.8m Outer slope 1.0m

[Muijs, 1997]

2.3.3 Conclusions

- It is difficult to obtain a good quality grass cover that has a large resistance against erosion.
- The strength and resistance to erosion of grass is still not well known.
- Grass covers on the outer slopes of sea dikes, ‘green dikes’, are often used in Germany and Denmark. These outer slopes are more gentle (1:6, 1:8, 1:10 or 1:20) than commonly used in The Netherlands (1:4).

2.4 Revetments of clay

First the properties of clay in dikes are described. Then, the use of clay as cover material in dikes is treated as well as the maintenance and management. After this, the existing dikes with clay covers in Zeeuws Vlaanderen in Zeeland, the Netherlands, are described together with the developed design method for the use of a clay layer in dikes.

2.4.1 Properties of clay in dikes

The properties of clay are divided in properties of clay as mineral soil fraction, stemmed form chemical and physical properties, and properties of clay as natural soil, the civil engineering properties. See Table 2.3 and Appendix IV for the descriptions of these properties.

Table 2.3 Subdivision of the properties of clay.

Properties of clay	
Mineral soil fraction	Natural soil
Water retention capacity Cohesion	Soil structure Permeability Erodibility Coherence Workability

Especially the soil structure development is very important and has also influence on other properties like the permeability and coherence. The soil structure development is the development of a crumbly structure of the clay which develops over time under the influence of weather conditions, vegetation and animal activity. In this way a structure of aggregates arises which for example increases the permeability and decreases the shape retaining properties see Chapter 6. Also compaction during the installation of the clay layers is of great importance for obtaining a good erosion resistant clay layer.

For the use in dikes the clay quality needs to fulfil several requirements. A subdivision into three erosion categories has been made, see
Table 2.4 and Figure 2.6. [TAW Klei, 1996]

See part 3 and 4 for more information on the erodibility of clay.

Table 2.4 Requirements for use of clay in dikes.

Requirement	Erosion-resistance category		
	1 Erosion-resistant	2 Moderately erosion-resistant	3 Little erosion-resistance
Flow limit (w_l) Plasticity index (I_p) Sand content	> 45% and > 0.73*(w_l-20) and < 40%	< 45% and > 18 and < 40%	< 0.73*(wl-20)% and < 18 and > 40%
Organic material content	< 5%	< 5%	< 5%
Salt content	< 4g/l	< 4g/l	< 4g/l
Chalk content	< 25%	< 25%	< 25%
Water content for working Top layer Core	 $I_c \geq 0.75$ $I_c \geq 0.60$	 $I_c \geq 0.75$ $I_c \geq 0.60$	 $I_c \geq 0.75$ $I_c \geq 0.60$
Extreme colouring from excavation or dying	Not allowed	Not allowed	Not allowed
Strong smell	Not allowed	Not allowed	Not allowed

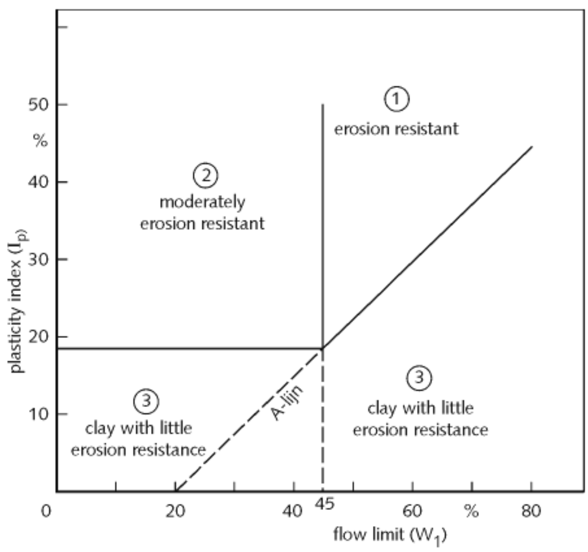


Figure 2.6 Erodibility reproduced as a plasticity diagram. The equation of line A: I_p=0.73(wl-20). [TAW Klei, 1996]*

2.4.2 The use of clay as dike cover material

The cover layer on the outer side of the dike consists in most cases out of a stone cover in the zone where the waves hit the dike, extended to the outer berm. Above the berm the cover layer consists most of the times of a grass cover, sowed on a 0.8m thick clay layer.

In some cases a stone cover is not desired. When, for example, natural areas like salt marshes are present in front of the dike. From natural and environmental point of view is a cover layer which fits more in the environment more desirable. A cover layer overgrown with preferably the original vegetation forms an appreciated solution. A grass cover layer is in this an obvious solution, but this is not a good alternative at locations where the grass cover is not able to develop to a good quality. A thick clay layer could also be used as dike cover layer. If this is applied in the zone where the waves loading, the clay layer has to have sufficient thickness, so that erosion will not lead to failure of the cover layer.

Along the Westerschelde, Zeeland, but also at other locations in the Netherlands, many existing stone covers on the sea dikes are considered to be not strong enough and the revetments need strengthening. In Zeeuws Vlaanderen, at the protected natural area Het Verdronken Land van Saeftinghe, extensive salt marshes are present in front of the dike. The revetments of these dikes also needed improvements. A stone cover is not preferred here because of the salt marshes. There is contact with salt water and swash marks are washed ashore, thus obtaining a good quality grass cover is not possible. Therefore an alternative with a clay cover layer was developed. See also 2.4.4. This type of cover layer requires a somewhat gentler slope, to reduce the wave load.

If the wave height is smaller than 1.6m, construction of so called green dikes (with grass cover) or brown dikes (with clay cover) are at the moment allowed, according to DWW and Projectbureau Zeeweringen. This is present in cases of a high foreshore. Before application of the clay layers, the existing old stone covers have to be removed, because else a slip circle can occur above the stones along which the clay cover can slide. If necessary, superfluous sand will be excavated. The clay cover is made of erosion resistant clay with erosion resistant category 1 and is applied in layers of approximately 0.5m. Each layer is compressed after placement. This generally happens with a bulldozer. This compaction needs to prevent soil structure development and thus prevents reduction of the strength.
An additional top layer of 0.5m can be applied on the erosion resistant clay, consisting of the original clay of the location. This can be sowed with a seed mixture of the local existing vegetation. This top layer is to support the new growth of the original existing vegetation. This layer has a lower erosion resistance, but it is better suitable for vegetation growth. This layer does not contribute to the strength of the revetments, but makes a fluent transition between the foreland and the dike. It also prevents the clay beneath from dehydrating and soil structure development, through which the actual clay can maintain its erosion resistance. The top layer can catch storms which occur previously of a design storm. Erosion of this layer is thus no problem and this top layer can therefore be called a 'make-up' layer. If the vegetation growing in this layer can develop into

well erosion resistance turf, it can even give an additional strength. See Figure 2.7 for an example of design of a clay dike.

During the construction of the clay revetment care should be taken that the applied clay is not too wet, in view of the processibility. During the construction is especially the compaction is very important for obtaining a good quality clay cover.
Risks during the construction phase are [van Etten, 1999]:

- No clay available of erosion resistance category 1.
- Logistic problems.
- The original clay is too wet for reuse as top layer.
- High water levels, which complicate the building.
- Precipitation.
- Transport of sand extracted from the dike.

Two test sections with clay covers were constructed in 1999 and monitored for two years. During this period no changes of the dike geometry were observed. However no severe storms have taken place in this period. The development of the vegetation layer in the clay is good and the clay dike is for the time being considered to be a satisfying alternative [van Etten, 2001]. This type of revetment with a clay cover is now more often used in Zeeuws Vlaanderen if a high foreland is present in front of the dike. See 2.4.4.

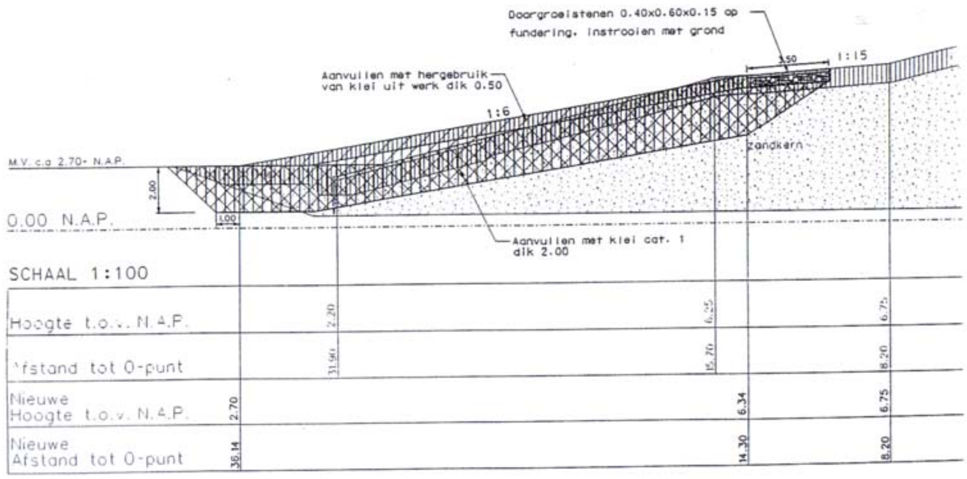


Figure 2.7 Cross section design clay dike, including the old stone revetment. [van Etten, 1999]

Obtaining good erosion resistant clay is the difficult point in the application of clay dikes. Clay from the neighbourhood of the building location is preferred, because especially the transport costs of large amounts of clay are high. In order to obtain the same vegetation as before, clay with the same structure and properties as found on the location site can be used. This is often to be found in the neighbourhood. Clay extracted after deepening salt marshes is a possible example, but this very rare. The use of an additional top layer of original clay with a mixture of seeds of the original vegetation can also for a good solution for keeping the original vegetation.
[van Etten, 1999; van Etten, 2001]

In a cost estimation of van Etten (1999) is concluded that a clay cover layer could be a cheaper solution than stone revetments, even when a gentler slope was applied and maintenance costs are included. According to Projectbureau Zeeweringen (2005) the costs of stone and clay revetments are around the same.

2.4.3 Maintenance

Small damage of the clay layer is not repaired. Only if holes larger than 0.5m are developed, repair can be considered necessary. The places, which need to be repaired, have to be partly excavated and the new clay has to be applied in layers, with compaction after each layer. This has to maintain the coherence of the cover layer. [van Etten, 1999]

The vegetation can, if desired, be mowed regularly dependent on the composition, to obtain a good erosion resistant grass cover. (See also 2.3)

2.4.4 Clay dikes in the Netherlands

In Zeeuws Vlaanderen, Zeeland, was started with two test sections with clay covers at Het Verdronken Land van Saeftinghe in 1999. Due to satisfying results this was followed by a new project in the Hellegatpolder. In 2005 a clay dike has been constructed close to Voorland Nummer Een and the area between the two test strips in Het Verdonken Land van Saeftinghe. This is done by Projectbureau Zeeweringen. See Figure 2.8 for the locations of the clay dikes in Zeeuws Vlaanderen.

1 Het Verdronken Land van Saeftinghe
2 Hellegatpolder
3 Voorland Nummer Een

Figure 2.8 Locations of clay dikes in Zeeuws Vlaanderen, Zeeland.

Het Verdronken Land van Saeftinghe

In 1999 two test sections were constructed with a clay cover by Het Verdronken Land van Saeftinghe, Koningin Emma polder and Van Alstein polder. The existing concrete blocks did not fulfil the demands anymore and needed to be replaced. Because of the location nearby the protected salt marshes, an alternative construction with clay was preferred. A good grass cover could not develop there, because of the salt sea water and swash mark, which are washed ashore on the Emma polder. The swash mark can choke the grass cover and thus prevent the development of a good erosion resistant turf. At both locations a clay layer of 2m thick was constructed with an additional top layer of 0.5m, with a slope of 1:6. [van Etten, 1999]

The test sections were monitored till 2004. In that period no storms have occurred and therefore no erosion was observed. A grass cover with a reasonably good quality grass cover developed and no significant changes have occurred on the constructed dike profile. [van Etten, 2001] (See the cover of this report for a photograph of the test section at the Koningin Emmapolder of July 2005.)

In 2006 the dike section between the two test sections was also covered with a clay layer.

Hellegatpolder

Hellegatpolder, east from Terneuzen, was covered with stones, and above the berm with grass. The stone cover did not meet the requirements anymore and needed strengthening. In front of the dike a high foreland with natural salt marshes is present (NAP+2.5m). Therefore a clay cover with a gentle slope (1:8.5) was preferred above a new stone cover. On top of the clay a 0.5m thick layer was placed, sowed with a grass mixture. The upper part of the slope, above the berm, has a 'normal' grass cover. See Figure 2.9, also here no erosion is noticed up till now.

Figure 2.9 Clay dike Hellegatpolder and Voorland Nummer Een.
[2nd photo: www.zeeweringen.nl June 2005]

Voorland Nummer Een
The dike close to Nummer Een has been under construction for a length of 2900m in 2005. Between the toe of the dike and the dike itself, a wide berm (NAP+2.5m) is located with a width varying form 30 till 130m. (See Figure 2.9.) This area with salt marshes is frequently used by birds and is two or three times a year flooded. This dike system consisted of an old dike with a basalt cover along the water line and a newer dike, a Deltadike (NAP+10m), behind the wide berm. The basalt cover is strengthened by stones and penetrated with asphalt. On top of that a cover of crushed Vilvoortse stone is placed to support the vegetation growth. The newer Delta dike is covered by a 2.5m thick clay layer of good erosion resistant clay and a top layer with the original clay, with a slope of 1:3.5. [www.zeeweringen.nl]

This location is in an area with a higher wave load compared to Het Verdronken Land van Saeftinghe. So it maybe will be possible to gain experience with erosion here in the coming years.

2.4.5 Design method

Despite little information on the behaviour of clay under wave loading, a design method is developed to determine a thickness of a clay layer as revetment on a dike. This is done to give a rough idea for a certain expected needed thickness of a clay layer.

The use of a clay cover is not treated in the Leidraad Zee- en Meerdijken (TAW LZM, 1999) or the Voorschrift Toesten op Veiligheid (TAW VTV, 2004) and therefore the engineering firm INFRAM (2003) has developed a design method by order of Dienst Weg- en Waterbouwkunde, Rijkswaterstaat. On basis of this design method a chapter is written in the design manual of Zeeland, which is used by Projectbureau Zeeweringen.

INFRAM
INFRAM (2003) has developed a design method to determine a certain required thickness of a clay layer as revetment on a dike. Due to little knowledge, this design method is based on little available data and is therefore considered to be unsure.
The method is based on Chapter B6.9 (Residual strength of filter and clay layer) of the Leidraad Zee- en Meerdijken (TAW LZM, 1999) and on Delta Flume tests to the residual strength of clay under stone covers (Wouters, 1993). The residual strength of clay layers, given in Table B6.9.1 (TAW LZM, 1999), are linearly extrapolated to a design graph, which determines the minimum layer thickness as function of the duration of the load. See Figure 2.10. The erosion by wave loading is considered to be normative. See Appendix V for a description of this design method.

The design method is considered to be unsure. The following points of attention are given in INFRAM (2003):
- The extrapolation of the erosion duration from 11 to 35 hours.
- The possible decrease of the erosion velocity of slopes gentler than 1:4.
- Experiences, gained at the test strips of the clay dike of Saeftinghe are not included.
- Small scale erosion tests should be performed on clay which is used in Het Verdronken Land van Saeftinghe.

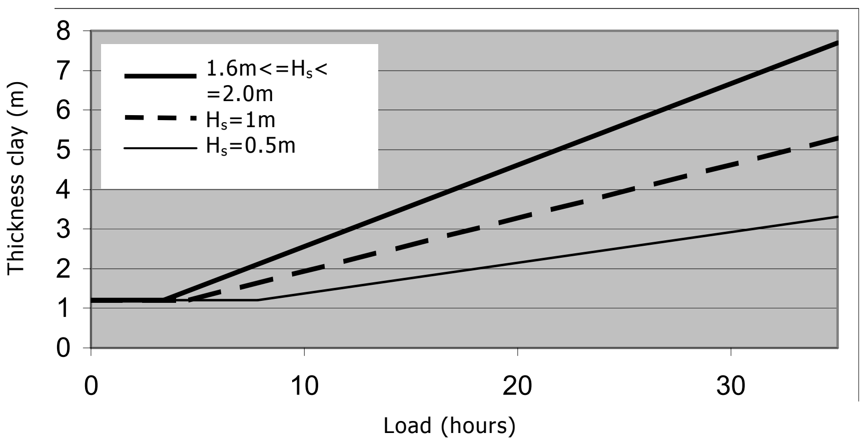


Figure 2.10 Design method INFRAM (2003), minimum required thickness of a clay layer for a certain load.

Design manual DWW
On basis of the design method of INFRAM (2003), Dienst Weg- en Waterbouwkunde formulated a design method for clay dikes:

With the flow schedule of Figure 2.11 it can be determined if the use of a clay cover layer is possible. The determination of the required thickness of the clay layer is done with the use of a spreadsheet. This spreadsheet is a back-box model. The formulas within this spreadsheet are unknown and the input is formed by:

- the position of the dike
- the wave boundary conditions
- the lowest point of the dike
- the design level
- the mean high water level
- the slope of the dike under the berm

The outcome of the back-box spreadsheet has to be increased by 0.5m to determine the layer thickness of the clay layer. This is to catch the soil structure development and damage development preceding a normative storm (considered design period of 50 year). The minimum required layer thickness is 1.5m or γH_s+0.5m (γ is a safety factor, mostly equal to 1).

General
Both design methods are not developed to determine the erosion of a clay layer, but to give some rough ideas for a certain expected needed thickness of a clay layer. Because design methods are concerned, a certain safety factor will be included. Both methods are based on very limited data and are therefore considered to be unsure.

2.4.6 Conclusions

- The concept of the clay dike has been tested and monitored for a few years at two test sections at Het Verdronken Land van Saeftinghe. During this period no changes of the dike geometry are observed but also no severe storms have taken place. The development of the vegetation in the clay is considered to be good and the clay dike is for the time being considered to be a satisfying alternative.
- A thick clay layer as a revetment on the outer slope of dikes is sometimes applied at other locations in Zeeuws Vlaanderen as well, at locations where a high foreland is present.
- A design method has been developed for clay layers. On basis of this method a design manual is made by Dienst Weg- en Waterbouwkunde which is used by Projectbureau Zeeweringen.
- The following remarks on the design method can be made:
    - The erosion velocity is only based on the Delta Flume experiments on dehydrated, structured clay.
    - No attention is paid to the influence of the slope. A decrease in erosion velocity could take place at more gentle slopes.
    - No attention is paid to the increase of erosion resistance of deeper layers in the clay.
    - The influence of the vegetation of the erodibility in not taken into account.
- According to the design manual a high foreland at a minimum level of MHW-0.5m needs to be present. This foreland has to stay stable the next 50 years with fixed salt marshes.
- The developed design method and design manual for clay dikes are considered to be unsure.

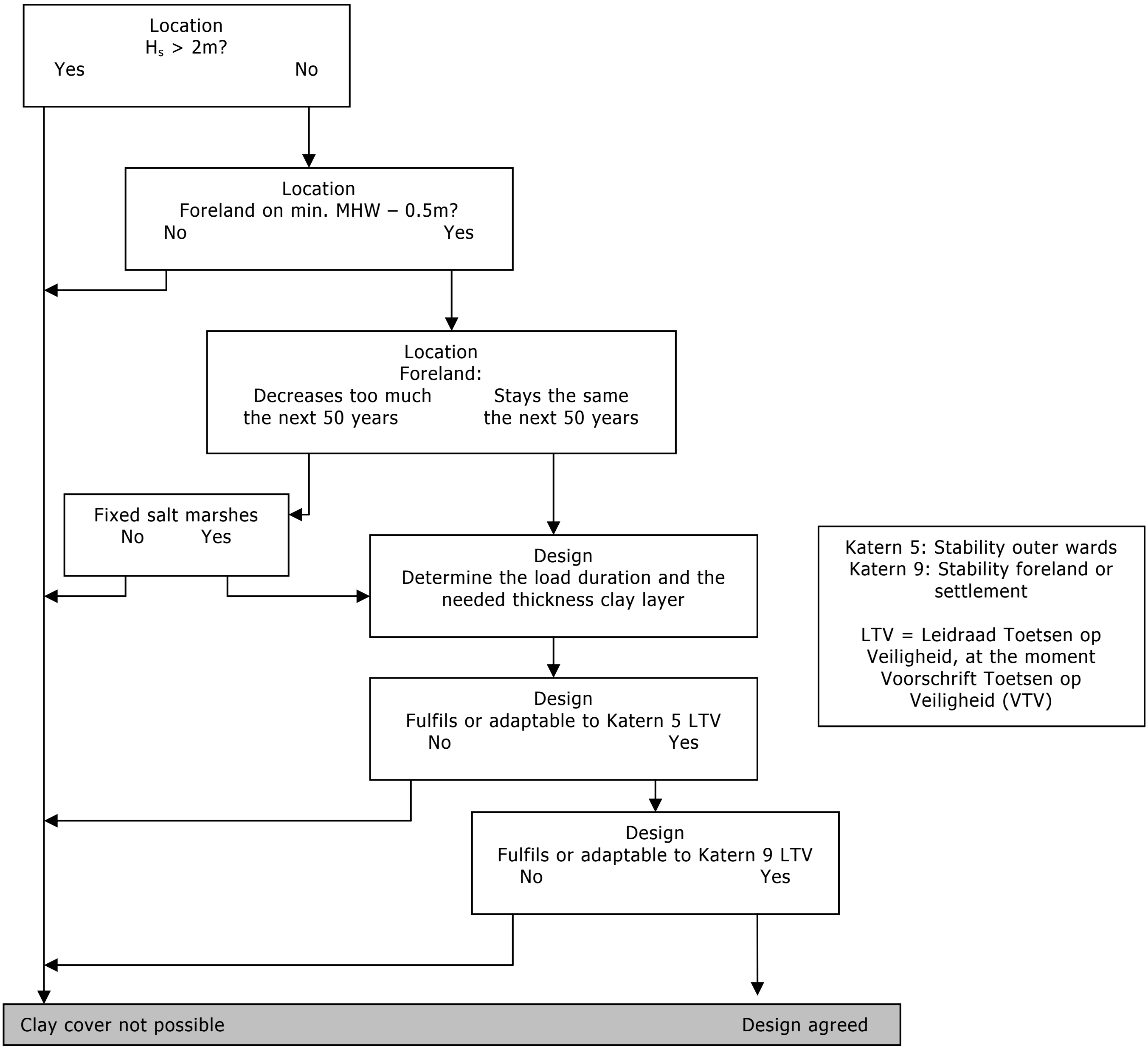


Figure 2.11 Flow schedule to determine if the application of a clay dike is possible.
[Handleiding Ontwerp]

2.5 Foreland and salt marshes

The foreland is not part of the dike, but it forms together with the dike a water retaining function. It is an area lying on the outer side of the dike which is regularly flooded. A foreland appears at certain locations and consists mostly of salt- or brackish water natural areas: mud or salt marshes. Because of its higher level in front of the dike it could reduce the wave load on the slope of the dike landwards of the foreland.

Salt marshes are natural areas laced with gullies and creeks which allow the salt water to penetrate deep into the marsh with every tide. They are biologically rich, owing to the rich supply of organic matter and the provision of protected habitats for many species. Over time the marshes accreted until they are only submerged during spring-tide. The vegetation consists of salt-loving plants and plants that are adapted to living with salt water, both considered to be rare at the moment. Many birds use the marshes for feeding, as roosts or as breeding grounds. For all these reasons salt marshes are considered to be important natural areas.

'Het Verdronken Land van Saeftinghe', Zeeuws Vlaanderen, 'De Schorren', Texel, in the Netherlands and 'Het Zwin', West-Vlaanderen, in Belgium are some examples of salt marsh areas.

Figure 2.12 Salt marshes De Schorren, at Texel, the Netherlands. [www.nioz.nl, February 2007]

3 Erosion and wave load

3.1 Erosion

3.1.1 Erosion due to currents

Erosion of non cohesive material
Erosion is the result of a gradient in sediment transport and starts if the load exceeds the critical value for the start of motion of the grains. This critical value is usually expressed in a critical flow velocity or critical shear stress, and is dependent on the soil properties. For non-cohesive soils the development of this erosion process is known, see Figure 3.1. Models and formulas describing the erosion process are available, such as Shields.

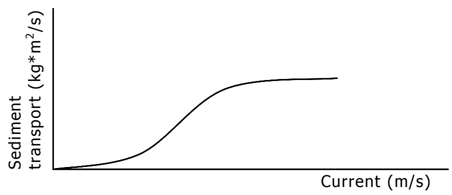


Figure 3.1 Schematization: Erosion of non-cohesive material for an increasing current. At a certain critical flow velocity the material starts to erode.

Erosion of cohesive material
For cohesive soils the erosion process is more complicated and little is known about the behaviour of cohesive material under hydraulic loading. The cohesive properties deliver extra physical-chemical forces and also the clay structure exerts influence on the erodibility, which is dependent on changes in the water content.

The erodibility of cohesive material is influenced by several factors [De Vroeg, 2002]:
- Soil structure development
- Density
- Clay-silt-sand fraction
- Initial salt content pore water
- Salt content of the eroding water
- Liquid content
- Type of clay minerals
- Sodium and lime content (natrium en kalk)
- Content of organic material
- Present metals (Fe, Al)
- Temperature (for fresh water)
- Presence of roots
- Presence of holes of animals
- In homogeneities (e.g. enclosed sand layers)

If is assumed that the erosion process develops in the same way as for non-cohesive material, with the critical shear stress, and thus the grain size, as measure for erosion, the critical shear stress for cohesive material will be larger. This corresponds with experiences, since clay has cohesive forces, but this method is nevertheless assumed to be too simplistic for cohesive material. [De Vroeg, 2002] Experiments by Mirtskhoulava in 1988 en 1991 show that the erosion process of saturated clay develops in different phases. First the bonds between the aggregates and lose particles breaks and small particles are washed away, which makes the surface rough. Enlargement of the drag forces and lift forces lead finally to the break of all coherence. In Figure 3.2 the erosion processes are schematized. In this erosion process a distinction is thus made between erosion of small pieces (grains), which wear out of the surface, and erosion of blocks (lumps). The total transport is the summation of those two. Little is known about the course of the block erosion.

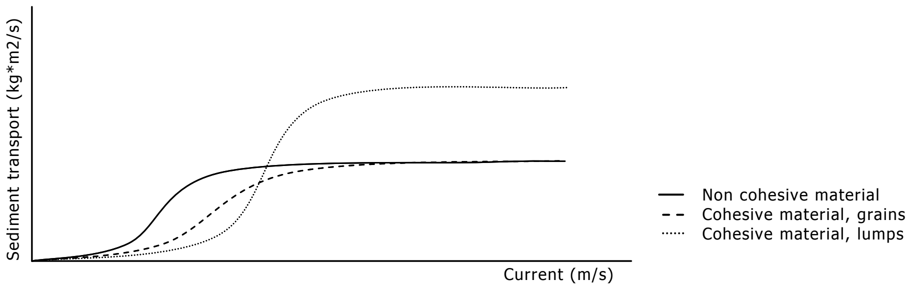


Figure 3.2 Schematization: Sediment transport of non cohesive material and cohesive material under an increasing flow velocity. For non cohesive material a distinction is made in transport of grains and transport of blocks of clay (lumps).

A determination by Mirtskhoulava of the critical depth averaged velocity for cohesive sediments is given in Table 3.1.

Table 3.1 Critical depth averaged velocity for cohesive sediments (by Mirtskhoulava). [Hoffmans, 1997]

Type of soil	h (m)	U_c (m/s)
Loamy sand, light loamy clay with low compactness	1	0.4
Heavy loamy clay with low density	3	0.5
Low density clay	10	0.6
Light loamy clay with medium compactness	1	0.8
Heavy loamy clay with medium density	3	1.0
Clay of medium density	10	1.3
Light loamy clay (dense)	1	1.2
Heavy loamy clay (dense)	3	1.5
Hard clay	10	1.9

Based on this work a simplified expression for the depth averaged velocity has been developed: [Hoffmans, 1997]

$$u_c = \log\left(\frac{8.8h}{d_a}\right)\sqrt{\frac{0.4}{\rho}\left(\left(\rho_s - \rho\right) g d_a + 0.6C_f\right)}$$

u_c the critical depth averaged velocity for cohesive sediments (m/s
h flow depth (m)
d_a size released aggregates = 0.004m
P_s density sediment (kg/m^3)
P density water (kg/m^3)
g fall velocity (9.81 m/s^2)
C_f 0.035C_0 (Pa), see Table 3.2

Table 3.2 C_0 values. [Hoffmans, 1997]

Type of soil and range of liquidity index	Soil property at voids ratio						
	0.45	0.45	0.65	0.75	0.85	0.95	1.05
Loamy sand							
0-0.25	14.7	10.8	7.85				
	(30)	(29)	(27)				
0.25-0.75	12.7	8.83	5.88	2.94			
	(28)	(26)	(24)	(21)			
Loamy clay							
0-0.25 (low plasticity)	46.1	36.3	30.4	24.5	21.6	18.6	
	(26)	(25)	(24)	(23)	(22)	(20)	
0.25-0.5 (medium plasticity)	38.2	33.3	27.5	22.6	17.7	14.7	
	(24)	(23)	(22)	(21)	(19)	(17)	
0.5-0.75 (high plasticity)			24.5	19.6	15.7	13.7	11.8
			(19)	(18)	(16)	(14)	(12)
Clay							
0-0.25		79.4	66.8	53.0	46.1	40.2	35.3
		(21)	(20)	(19)	(18)	(16)	(14)
0.25-0.5			55.9	49.0	42.2	36.3	31.4
			(18)	(17)	(16)	(14)	(11)
0.5-0.75			44.1	40.2	35.3	32.4	28.4
			(15)	(14)	(12)	(10)	(7)

An overview of research to the critical velocity for cohesive material is given in De Vroeg (2002).

3.1.2 Erosion of non cohesive material due to waves

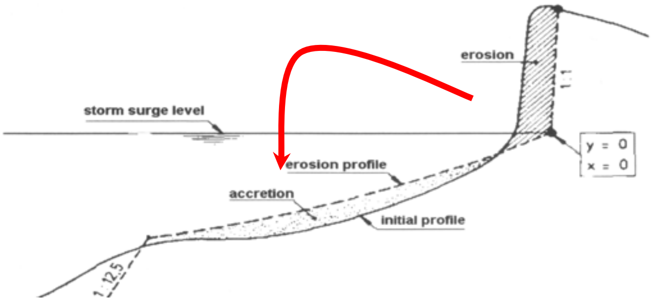


Figure 3.3 Beach profile after dune erosion. [Schiereck, 2001]

The erosion of slopes due to waves is often investigated for dunes. Waves cause a step profile as is shown in Figure 3.3. This is the natural equilibrium profile for slopes consisting of homogeneous non-cohesive soils of sufficient thickness. If no longshore transport takes place the eroded sand is deposited in front of the dune. The shape is determined from the position of the still water level. Above the still water line the angle is assumed to be equal to the angle of repose. (See also Figure 3.4)

Vellinga (1986) stated a formula or this equilibrium profile to be used as a first indication:

$$z = 0.39 w^{0.44} x^{0.78} = p x^{0.78}$$

z see Figure 3.4 [m]
w fall velocity of the particles [m/s]
x see Figure 3.4 [m]

From this formula the intrusion length of the wave in the profile is found:

$$L_e = p^{-1.28} H_s^{1.28}$$

L_e intrusion length [m]
H_s significant wave height [m]

[Schiereck, 2001]

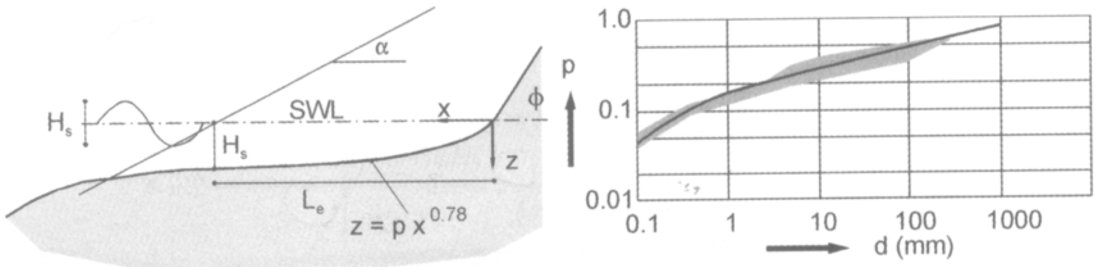


Figure 3.4 Erosion of a slope of non cohesive material by waves [Schiereck, 2001]

See Chapter 9 for more information.

3.2 Wave load

The slope of a sea dike is subjected to flow, tide and waves. The wave load can be divided in an up and down flow over the slope and the impact of breaking waves. Waves, characterized by a wave height and a wave period, can be translated to a flow velocity or shear stress close to the slope. The normative loads on a slope consisting of a clay cover are wave impacts during storms. During a storm the waves are high and are accompanied by high water levels.

3.2.1 Wave impact

A wave impact is caused by the mass of the water of the wave which attacks the slope with large rapidity. Not every wave results in a wave impact and not every wave impact has the same size. A wave impact is a pressure impact acting on a certain width. The shape of this impact pressure distribution is assumed to be triangular with H as the base length. See Figure 3.5. (See also Appendix VI.)

The quasi-static wave load is the pressure under a wave which increases and decreases with the wave cycle. When a wave collides with the surface a wave impact, a very short and very high pressure occurs. [Schiereck, 2001]

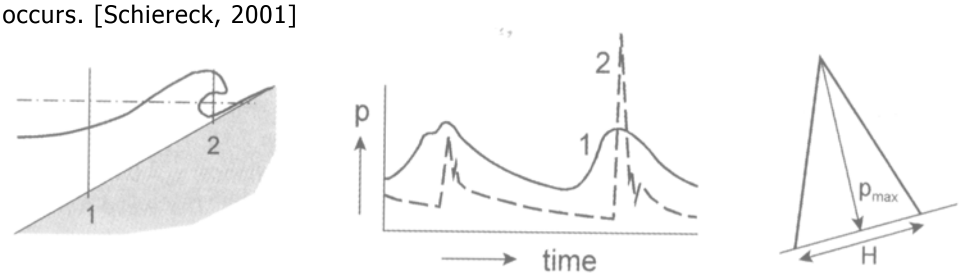


Figure 3.5 Wave impact with pressure distribution [Schiereck, 2001]

The maximum pressure during a wave impact is related to the wave height:

$$p_{\max} \therefore H_s$$

p_{max} maximum pressure during wave impact (N/m^2)
H_s significant wave height (m)

$$p_{\max} = \rho_w g q H$$

ρ_w density of the water (kg/m^3)
g acceleration of gravity (m/s^2)
q impact factor, dependant on the slope angle (-)

An approximation for the maximum pressure during wave impact is given in Schiereck (2001):

$$p_{\max 50\%} \approx 8 \rho_w g H_s \tan\alpha$$

$$p_{\max 0.1\%} \approx 16 \rho_w g H_s \tan\alpha$$

$p_{max\ 50\%}$ maximum pressure exceeded by 50% of the waves (N/m^2)
$p_{max\ 0.1\%}$ maximum pressure exceeded by 1 in 1000 waves (N/m^2)
ρ_w water density (kg/m^2)
H_s significant wave height (m)
α slope angle (°)

3.2.2 Breaker type

The load on a dike slope is for a large part determined by the way the waves break on the slope, the breaker type. See Figure 3.6. The breaker type depends on the surf similarity parameter or Iribarren parameter, which is dependent on the steepness of the slope (tan α) and the wave steepness (H/L_0):

$$\xi = \frac{\tan\alpha}{\sqrt{H/L_0}}$$

ξ Iribarren parameter (-)
α angle of the slope (°)
H wave height (m)
L_0 wave length at deep water (m)

$$L_0 = \frac{gT^2}{2\pi} \text{ (m)}$$

T wave period (s)

[Battjes, 2001]

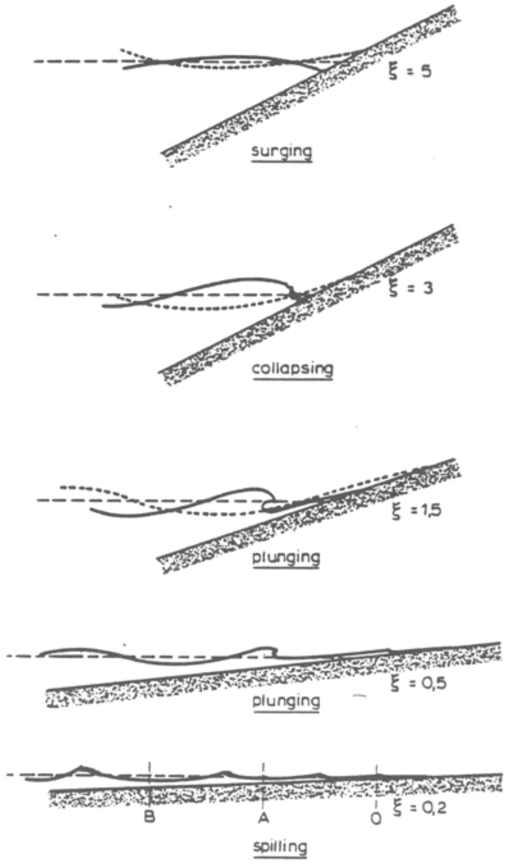


Figure 3.6 Breaker types. [Battjes, 2001]

Plunging breakers on a dry slope give the highest wave load on the slope. This wave impact gives fast, high pressure fluctuations and high flow velocities of a short duration.
Gentle slopes (e.g. 1:8) give lower values of the Iribarren parameter which thus reduce the load on the slope. However, a set-up by the wind during a storm will also have influence on the wave behaviour.

3.2.3 Storm surge

A storm surge is determined by the surge effect and the tidal effect, see Figure 3.7.

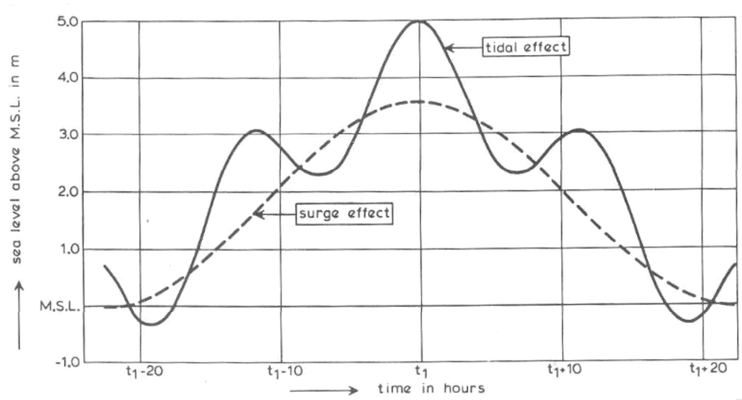


Figure 3.7 Storm surge. [Vellinga, 1986]

4 Experiments to the strength of clay under wave loading

Many experiments have been performed on clay: laboratory experiments, field experiments and large scale models tests. This chapter gives an overview of all large scale experiments performed on clay. These experiments were performed with the aim to obtain information on the residual strength of clay. First the experiments of the residual strength of clay under stone revetments are described, next the experiments of the residual strength of clay under grass sods.

4.1 Research of the residual strength of clay under stone revetments

First in 1984 and later, more thorough, in 1992, the residual strength of a clay sub layer was tested after experiments on the stone revetments itself. The residual strength of a dike is defined as the capacity of the sub layers to withstand wave and current loading after the top protection layer has failed. It is expressed in terms of the time it takes for erosion to progress through the entire clay layer and expose the sand of the core of the dike. The experiments were performed in the Delta Flume of Delft Hydraulics, where it is possible to make a 1:1 model of a dike. Waves were generated which break on the slope of the model.

First the Delta Flume experiments are described and relevant conclusions for an analysis in Part 3 are given.

4.1.1 Delta Flume 1984 (DF1984)

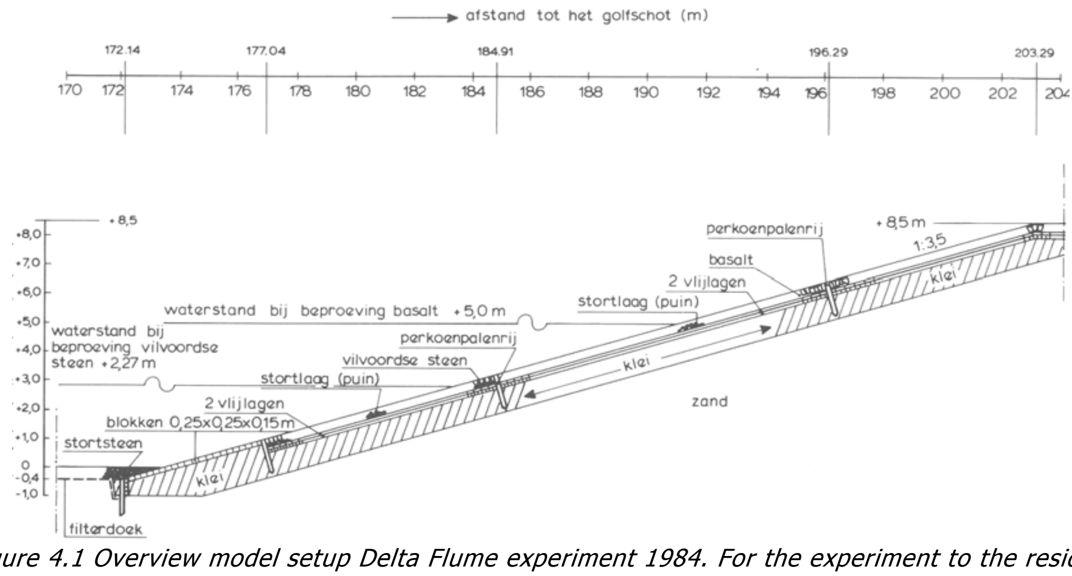


Figure 4.1 Overview model setup Delta Flume experiment 1984. For the experiment to the residual strength of the clay layer the stones were removed.

In 1984 research was performed on the strength of dikes along the Oosterschelde under concentrated wave loading. This was done with the aim to gain insight into the strength of the dikes in order to prescribe a closure procedure for the Oosterschelde storm surge barrier. Therefore the stability of several existing revetments was tested in a large scale experiment (1:1) in the Delta Flume under the influence of long-lasting wave loading. In the end, the erosion resistance of the clay sub layer was tested. The clay sub layer had a thickness of 0.80m and separate layers of 0.25m, which were applied each and then compacted. Two types of clay were used: clay with a low sand percentage and clay with a high sand percentage.

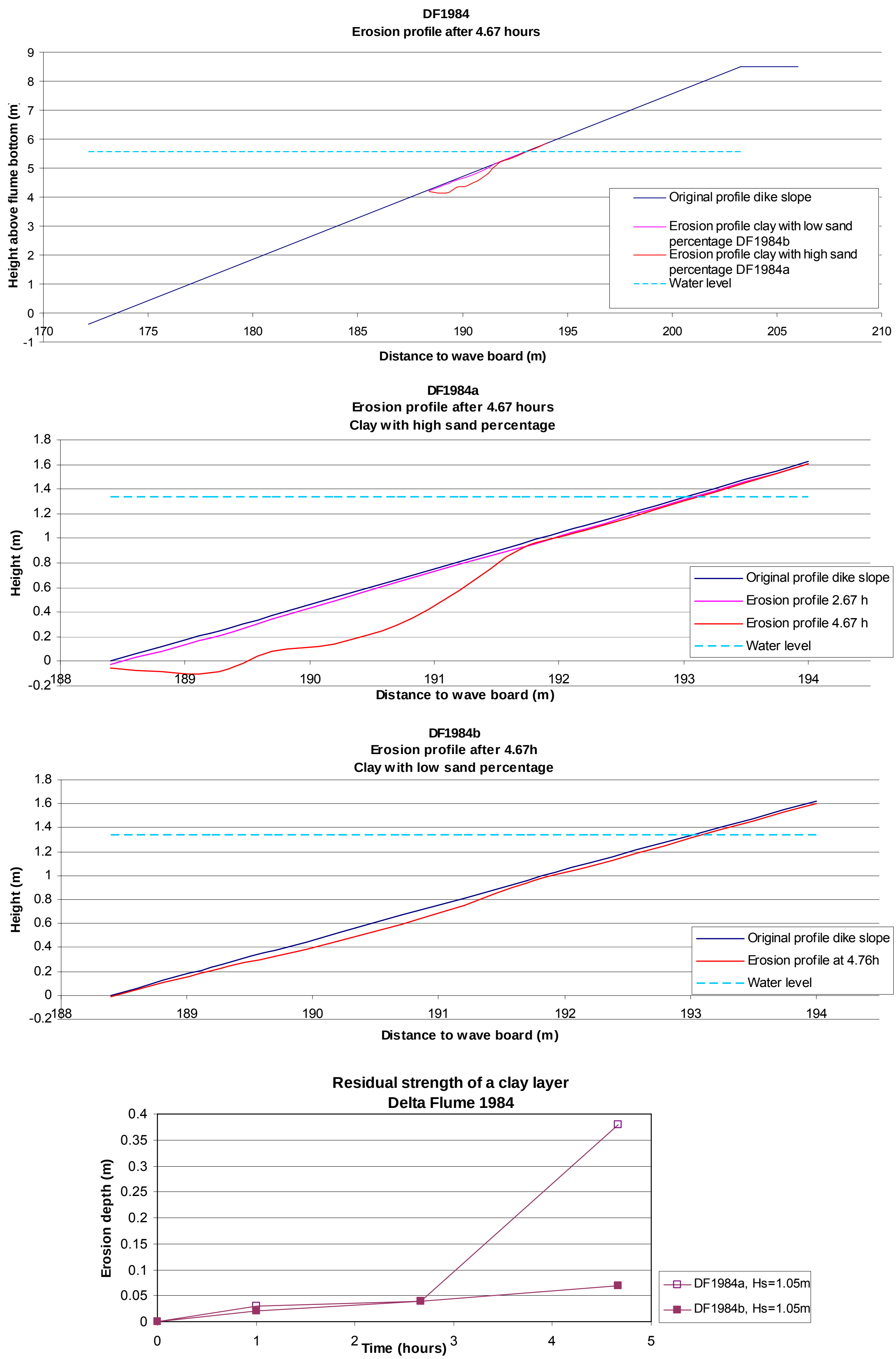


Figure 4.2 Development of erosion holes in the unstructured clay layer under a wave load with H_s=1.05m. Results obtained during the Delta Flume experiment of 1984 to the strength of Oosterschelde dikes.

The main difference between the two clay types is the grain-size distribution. In the clay with a low sand percentage more small particles were present than in the clay with a high sand percentage. Both clays had a high water content.

The first experiment was aimed at damage development of Vilvoortse stone and its sub layers. Holes were made in the revetment, but the brick layer was kept in place. After 15.35 hours the erosion of the clay with a low sand percentage was 0.08m and the clay with a high percentage 0.22m, but both damages were developed around poles at the borders of the flume. The wave heights were 0.77m (first 7.15h) and 1.02m (rest of time). The brick layers were washed away in around 1.30 hours, and therefore it can be concluded that the clay has a large resistance to erosion, of many hours. However, because the stones were spread over the holes during the test, the erosion development of the clay layer is influenced by the stones and therefore this test is not included in the comparison of Chapter 8.

The second experiment was performed on the bare clay layer with regular waves, after removal of the stones. The properties of the experiment are given in Table 4.1. The erosion of both clay types was restricted to the zone under the waver level. The wave breaking was of a surging type, so no real breaking occurred, but the largest erosion took place in the wave breaking zone. During the first 2.40 hours hardly any erosion occurred, only a small hole developed in clay with a high sand percentage. After 2.40 hours this hole extended quite quickly to 0.4m deep.
Also artificial notches were made in the clay after 2.40 hours to accelerate the erosion process. However the notches only grew a little under the water level in both clay types. The notches above water level were not affected. The development of this experiment is given in Figure 4.2. See also Appendix VII.

[Burger, 1985]

Table 4.1 Properties of the Delta Flume experiment 1984

Properties experiment DF1984		
Thickness clay layer	d_c	0.80m
Wave height	H_s	1.05m
Wave period	T	12s
Slope		1:3.5
Duration experiment	t	4.40h

Conclusions

- This experiment was performed on unstructured clay, because the clay was applied in thin layers (0.25m thick) and is compacted after the installation of each layer.
- Hardly any erosion occurred to clay with a low sand percentage.
- An erosion hole of 0.4m deep developed in clay with a high sand percentage after 4.40h.
- This developed hole could be a result of the clay type with a high sand percentage or of bad compaction, which made it possible for aggregates to be washed away.
- After 4.40h the experiment was stopped.

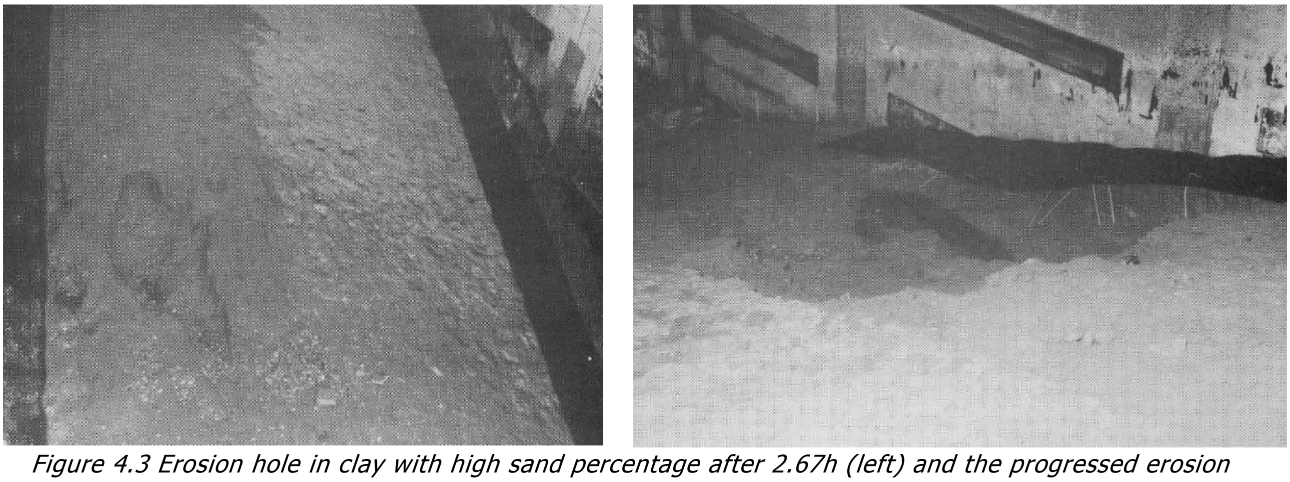


Figure 4.3 Erosion hole in clay with high sand percentage after 2.67h (left) and the progressed erosion below the water line (right) [Burger, 1985]

4.1.2 Delta Flume 1992 (DF1992S)

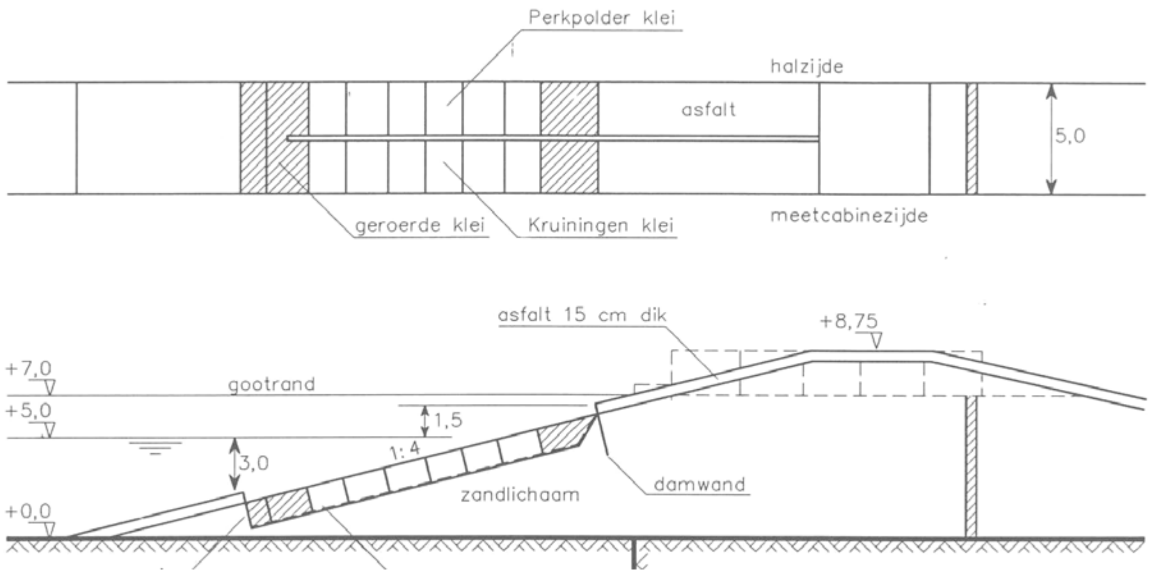


Figure 4.4 Overview model setup Delta Flume experiment 1992 [Wouters, 1993]

In 1992, the residual strength of clay under stones was tested in a 1:1 model in the Delta Flume of Delft Hydraulics, after the testing of the stone revetment itself. The experiment was executed with breaking waves on a dike slope of 1:4 with a 0.8m thick clay layer. Two types of clay were tested under two different wave conditions. The type of the clay, the storm duration and the wave height was taken into account.

Properties of the tested clay

The tested clay came from beneath stone cover layers from two locations in Zeeland; a sea dike in Perkpolder and the eastern dike in Kruiningen.

The clay from the dike in Kruiningen had an erosion category 1, except the 0.1-0.2m thick top layer, which had a somewhat lower resistance to erosion. In smaller scale tests undertaken before larger scale experiments in an erosion apparatus, there was hardly any erosion observed. The clay had a clear soil structure over the whole depth, with some sand and silt inclusions in the upper decimetres. A net of roots was present on top of the clay. During the extraction (September 1991) the clay was very dry and there was hardly any cohesion left.

The Perkpolder clay had a moderate resistance to erosion (category 2). In the erosion apparatus there was however hardly any erosion observed. The upper and the lowest 0.5 decimetres were sandy and in the clay several sand lenses were present. In these areas (the upper and lowest decimetres) a fine soil structure has developed, but this is not a strong development. During the extraction the clay was moist.

The clay samples were taken out of the dike in blocks, 0.8m thick by 2.45m wide. During extraction the samples were slightly dislocated on the edges and the underside was a compacted slightly as a result of shaking during transport. Thus as a result profile changes occurred in the upper and lowest 0.15m. For the application in the Delta Flume experiment the best samples were used and the edges were cut off, in order to obtain the most undistorted original clay layer. The small changes were not considered to influence the representiveness of the clay from these dikes.

Description of the experiment

Two test series were performed on both clay samples: one at high water conditions (test K) and one at low water conditions (test L). The representative extreme wave conditions for Zeeland were used with irregular waves (in a Pierson-Moskowitz spectrum). Details are given in Table 4.2.

Table 4.2 Properties Delta Flume experiment 1992, clay under a stone revetment

Properties experiment DF1992S		Test K (high water)	Test L (low water)
Thickness clay layer	d_c	0.8m	0.8m
Water level	h	5.0m	3.5m
Wave height	H_s	1.47m	1.0m
Wave period	T_p	4.9s	4.2s
Slope		1:4	1:4
Duration experiment	t	2h	15h

Damage started just under the water level in the zone of the breaking waves and extended slowly upwards. In the wave run-up zone no significant erosion occurred. Damage arose in the form of block erosion, but the erosion of the Perkpolder clay was more even. The blocks of eroded clay were aggregates coming loose from the clay and were rounded by the waves to elliptical shapes. The maximum dimensions of the blocks were around 0.15m. In many blocks cracks were visible along which the block was fallen apart. The developed erosion holes had very steep upper and side edges and sometimes also steep edges on the underside, which however were a bit lower. The bottom and sides of the holes followed the cracks of the soil structure of the clay, but this soil structure was sometimes more smooth due to the wearing by the waves.
The erosion hardly deepened, but moved higher on the slope. This was also seen at the water movement in the gulley, because the breaker point moved higher on the slope.

High water
In the high water experiment (K series) in the Kruiningen clay a hole formed at +5.2m within 10 minutes, which developed to 0.35m wide and deep in 15 minutes. In the other zones only shallow damage occurred to the top layer by peeling off of pieces of several centimetres/decimetres. Even after an hour (after approx. 700 waves), only in the zone under still water level gradual erosion occurred. Above the still water level, just a little erosion occurred on the sharp edges (from prints of shoes or equipment).
Also the upper decimetres of the Perkpolder clay eroded in less than 0.5h. After removal of 0.4m clay the damage moved upwards along the slope rather than downwards into the clay. This delay of downwards erosion could be related to the small extent of soil structure development beneath 0.3m in the Perkpolder clay.

The Kruiningen clay was completely eroded after 7190s (approximately 2 hours). At that time a hole of 0.6m deep had formed in the Perkpolder clay. At this point the experiment was stopped.

Figure 4.5 Damage of the clay layer with a soil structure under a stone revetment. The clay eroded in the form of lumps to a depth of 0.4m deep.

Kruiningen clay Perkpolder clay Perkpolder clay Kruiningen clay

Figure 4.6 Pictures of the developed erosion holes after the high water experiment (left side) and the low water experiment (right side).

Low water

The damage development during the low water experiment (L series) was comparable with the high water experiment, but occured slower. It was remarkable that after the removal of the upper 0.3m of the Perkpolder clay (the part with the soil structure) in the zone under still water level, the damage developed upwards along the slope and the erosion depth did not increase. Only after 15 hours of loading did the 0.8m thick clay layer erode. (It should be noted that the lowest 0.2m did not have a soil structure, because of the compaction during transport.)

Pictures of the developed erosion holes at the end of the experiments are given in Figure 4.6 and an overview of the development over time is given in Figure 4.7 and Figure 4.8. See also Appendix VIII.

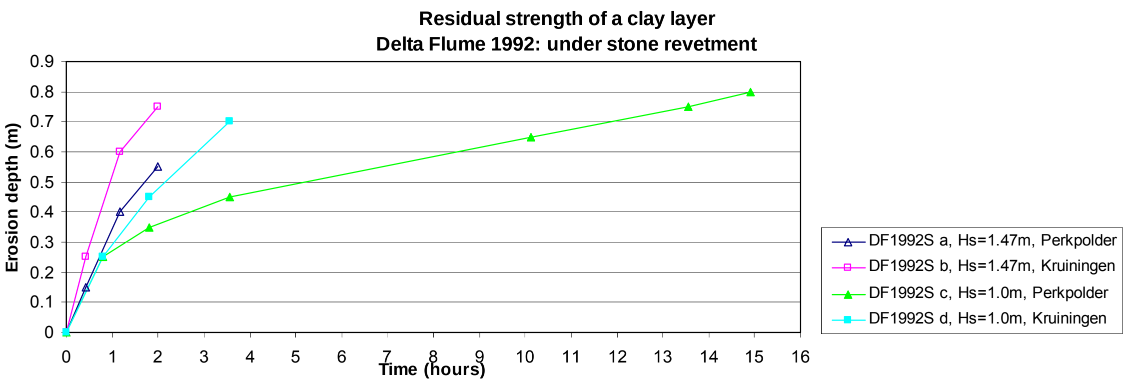


Figure 4.7 Development of the loss of thickness of a structured clay layer under wave loading. Results obtained during the Delta Flume experiment of 1992 of the residual strength a clay layer in stone revetments.

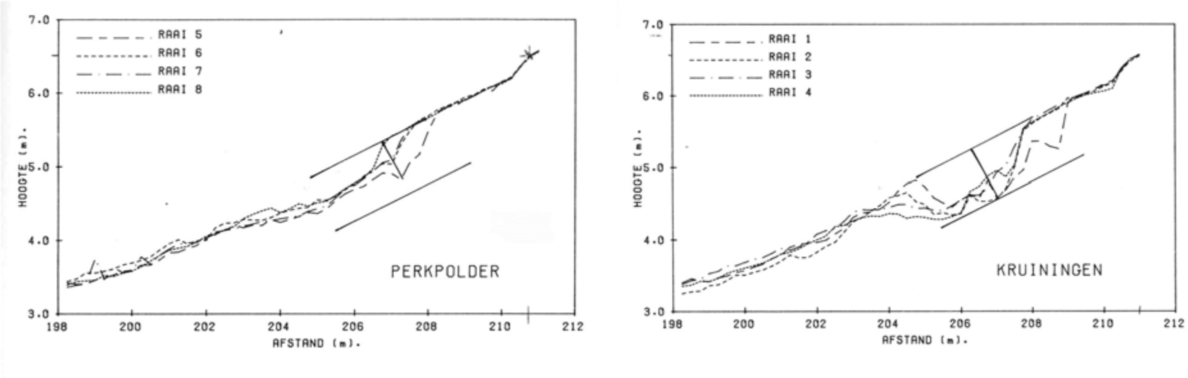
If the maximum erosion is determined to be 0.5m, the erosion velocities can be determined as given in Table 4.3. Kruiningen clay erodes approximately twice as fast as Perkpolder clay. Increasing the wave height by 50% results in doubling of the erosion velocity.

Table 4.3 Erosion velocities Delta Flume 1992 under stones

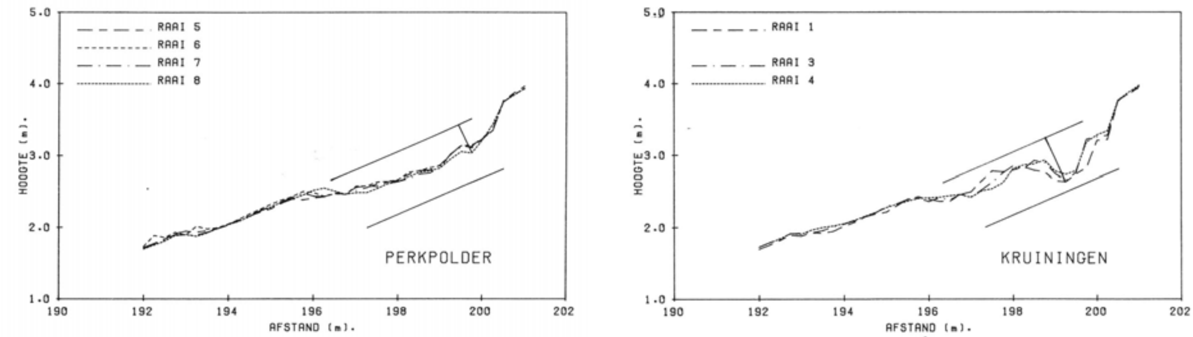
Test series	H_s (m)	T_p (s)	Duration erosion of 10mm (s)	
			Perkpolder clay	Kruiningen clay
K	1.47	5.0	120 (=0.30m/hour)	70 (=0.52m/hour)
L	1.00	4.2	288 (=0.13m/hour)	160 (=0.23m/hour)

[Wouters, 1993]

High water experiment: H_s=1.47m, end of test (at 2h):



Low water experiment H_s=1.05m
Failure of the Kruiningen clay (at 3.6h):



Failure of the Perkpolder clay (at 15h):

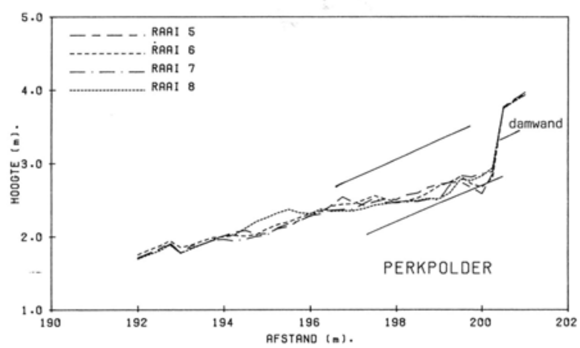


Figure 4.8 Overview of the developed erosion holes. [Wouters, 1993]

Analysis and conclusions

This experiment showed that the upper part of the clay layer under a stone revetment has a well developed and visible soil structure, which erodes easily under wave loading. The erosion, and thus the residual strength of the clay layer, depends strongly upon the amount of soil structure development. Clay with a more developed soil structure erodes faster. The under side of the Perkpolder clay had a less developed soil structure and appeared to have a significantly higher residual strength.

The erosion starts in the zone of the wave impact, develops further upwards on the slope and deepens quickly in the zone of the wave impact. The developed hole has steep edges.

A clay layer of 0.8m thick with a clear soil structure is determined to have a residual strength of several hours at a wave load of 1m and higher.

Under a stone cover some surface erosion in the clay always takes place, which leads to the development of gullies. This erosion is present near transition zones and not dependent on the position of the high water level. The average depth increases by 0.5-10mm/year. [Kruse, Aug 1995]

The response of the water pressure meters is strongly dependent on the local conditions of the clay. If a crack is widened, the wave pressure meter in the crack will give a higher result. There is relatively small damping in deeper lying water pressure meters and no significant difference between Kruiningen and Perkpolder clay, while the under side of the Perkpolder clay is less structured. The occurring water pressure gradients in the clay during wave loading are regularly high enough for lifting of the clay cover, but are of a short duration. The clay however is not quickly washed away, but there is also no gradual erosion. Therefore it can be stated that for the start of erosion not a certain critical water pressure is normative, but the influence of waves during a certain time period. Thus, the fatigue of cohesion and interlock forces of the aggregates is important. It can be stated based on results from the eroded holes and wave conditions that with every wave an aggregate is removed.
[Hofmann, 1995 p.10; 1993]

The erosion velocities (Table 4.3) show that the relationship between K and L of the time needed to significantly damage the clay layers is around the same (2.4 (288/120) and 2.3) This relationship is approximately similar to the relationship between the incoming energy flux (2.5) in both series. (The wave energy is dependent on the square of the wave height (H^2)). The damage caused can thus be comparable to the incoming energy, with which the local dissipated energy will be connected. Therefore the damage to the clay layers, in terms of number of waves (load duration) could be described in coherence with the height of the load, instead of just the exceeding of a critical load. So the inflicted erosion damage scales with the supplied energy.
[Hofmann, 1995, p.8,9; Hofmann, 1993 p.12]

Reanalysis including Delta Flume experiment on grass cover (DF1992G, see 4.2.2)
The results from the Delta Flume experiments on the residual strength of clay under stone covers have been reanalysed including the results of the Delta Flume tests on grass covers (1992, see 4.2.2). The erosion of clay in the zone of wave loading takes place mainly by the retreat of a cliff in the slope.

- Erosion by vertical deepening of the erosion hole in the zone of wave loading for waves of 0.75m is only effective for loose soil or soil with a fine soil structure. (This is present in the upper 0.35m of clay under stones.)
- Erosion by vertical deepening of an erosion hole for waves higher than 1m only occurs when a clearly developed soil structure is present.
- For waves of approximately 2m there is hardly any vertical erosion in a stiff soil without a soil structure, like is often present below MHW +1m. Therefore it can be concluded, that a stiff, closed clay layer strongly delays vertical erosion.

The build up of a water overpressure in the slope due to the waves can lead to instability problems if the surface becomes locally too steep. The upper side of the hole will retreat fast if the tensile strength of the soil of the cliff is less than the forces which are induced by the water pressure gradients. For closed clay layers without soil structure this will only occur at very high waves.
[Kruse, June 1995]

4.1.3 Conclusions

- A well developed and clear soil structure is present in the upper part of a clay layer under a stone revetment.
- The amount of soil structure development is an important normative factor for the resistance to erosion. Clay with a soil structure erodes easily under wave loading and unstructured clay erodes much slower. The amount of soil structure development appears to be more important than the clay type.
- The wave impact is the most important load on the slope.
- Damage development due to waves starts at the level where the breaking waves hit the dike.
- The erosion in the zone of the wave loading takes mainly place in by the retreat of a cliff in the slope.
- The damage develops further upwards on the slope and deepens in the zone of the wave impact.
- The water pressures in the clay are strongly dependent on the local conditions of the clay. (In a crack the pressures are high, in an aggregate low.)
- The occurring water pressure gradients during a wave loading are high enough to lift the clay, but the clay is not quickly washed away.
- It is assumed that fatigue of the cohesion and interlock forces of the aggregates could be an important factor in the erosion process.

4.2 Research into the residual strength of clay under grass

Several large scale experiments have been performed on grass slopes. Initially research was concentrated only on the strength and the erodibility of grass, but some experiments also focused on the erodibility of the clay layer. These experiments were mostly performed with flowing water, only the Delta Flume experiment of 1983, the Delta Flume experiment of 1992 on a Friesian sea dike and the Schelde Basin experiment of 1994 were performed with wave loading. These experiments are described in this section. See also Appendix III for an overview of the large scale experiment performed on grass covers.

Based upon the description of the experiments several conclusions, relevant to this research have been made.

4.2.1 Delta Flume 1983 (DF1983)

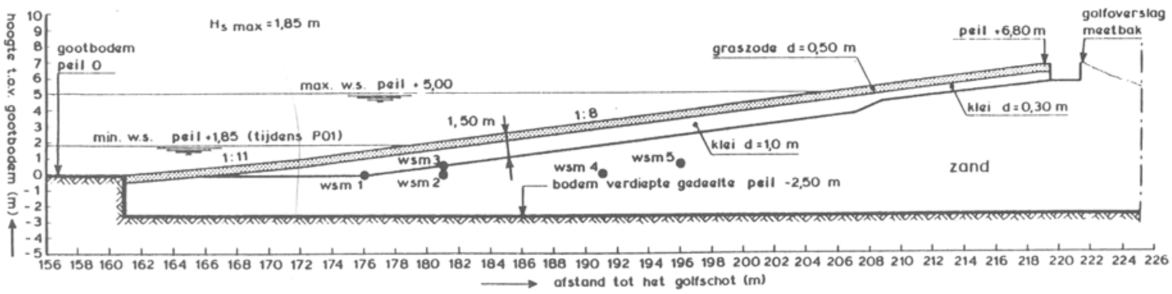


Figure 4.9 Model setup in Delta Flume experiment 1983. [Burger, 1984]

In the Delta Flume of Delft Hydraulics, experiments were performed in 1983 pertaining to the strength of the outer slope of a green dike during a storm flood (see Figure 4.9). This research was performed because the revetment of a dike in Friesland needed improvement, clay was amply available and a high foreland was present. Therefore the construction of a dike with clay and grass was suggested, with or without concrete stones that allowed vegetative growth.

Table 4.4 Properties of the Delta Flume experiment 1983.

Properties DF1983		
Thickness clay layer	d_c	1.0m
Thickness grass sod	d_g	0.5m
Wave height	H_s	1.57m
Wave period	T_p	5.26s
Slope		1:8
Duration experiment	t	8h
Water level	h	+5.00m
Location holes	h-1m	+4.00m
	h-0.5m	+4.50m

After two experiments on the grass slope, an experiment was performed on a grass slope with holes, which were made beforehand. The holes were 0.50m*0.20m and 0.07m deep. The position of the holes was such that the wave load would have the greatest impact on the holes as possible. (Two holes in the wave run-up zone did not suffer any erosion.) At constant water level and with a constant irregular wave load the extension of the holes was studied. After 8 hours the experiment was stopped. The first occurrence of erosion was noticed after 5.5h, in the holes just below the water level. It was observed that the damage developed because the grass cover around the holes was undermined due to clay erosion in the holes. Parts of the densely rooted top layer were subsequently torn down by the breaking waves. The erosion velocity increased approximately proportional to the circumference of the hole (progressive erosion). After 8 hours of loading no erosion of the sub layer of clay with a low sand percentage was observed and the root structure of the grass cover was still intact, only the grass itself had disappeared. See Table 4.4 for the properties of the experiment and Figure 4.10, Figure 4.11, Figure 4.12 and Figure 4.13 for the results.

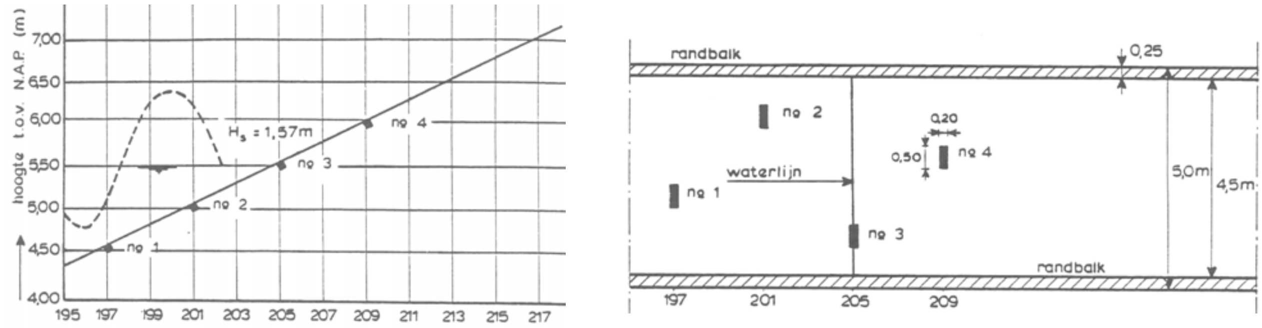


Figure 4.10 Location initial made holes: side-view (left) and view from above (right). [Burger, 1984]

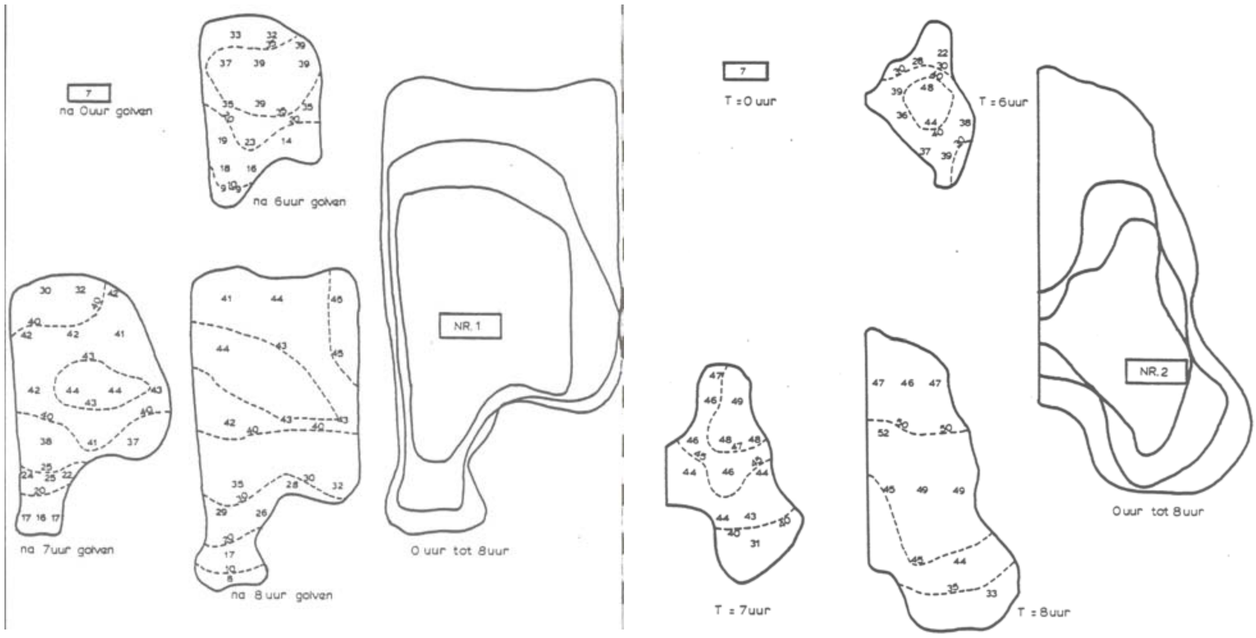


Figure 4.11 Damage development of hole 1, at 1m below water level NAP+4.50m, (left) and hole 2, at 0.5m below water level NAP+5.00m (right). [Burger, 1984]

The clay layer was applied and compacted in thin layers. The clay in the turf was very sandy (average 45% sand). The presence of the sand lenses was clearly visible and the grass was able to develop well in this layer. The thickness of the clay layer was 1.0m under the storm surge level (NAP+5.50m = +5.00m above the bottom of the flume) and 0.3m thick above this level. The substratum of clay, under the grass layer, consisted of a high amount of small particles and can be described as good erosion resistant clay. The consistency and the density are however low therefore it does not fulfil the current demands for clay in dikes.

The experiments have an Iribarren parameter of 0.6, due to the gentle slope. Thus the waves were plunging breakers, but the wave hits the water layer of the previous wave and not directly onto the slope. This reduces the load on the slope. The extreme form of this breaking manner is a spilling breaker.

Including grass cover
An experiment was performed on the intact grass cover with waves of (H_s) 1.03m for a duration of 18 hours. (Tp=5.20s, h=+2.80m) The top layer of grass and clay was eroded and tore away, but no erosion of the sublayer of clay with a low sand percentage occurred.
Another experiment was done with wave heights, wave periods and water levels which were expected to occur during a design storm for this dike in Friesland. With varying water heights of 0.65-1.85m for a duration of 29h, the maximum erosion was 0.005-0.01m. The clay erosion developed in the zone of long lasting moderate waves as well in the zone of heavy waves, occurring during a short period. In both cases the erosion was developed in the zone of 0.5-1.0m under the water level. This clay erosion occurred quite quickly and the roots of the grass cover were still present afterwards. The roots formed a kind of felt layer which prevent the clay from washing away. See also Appendix IX.

[Burger, 1984]

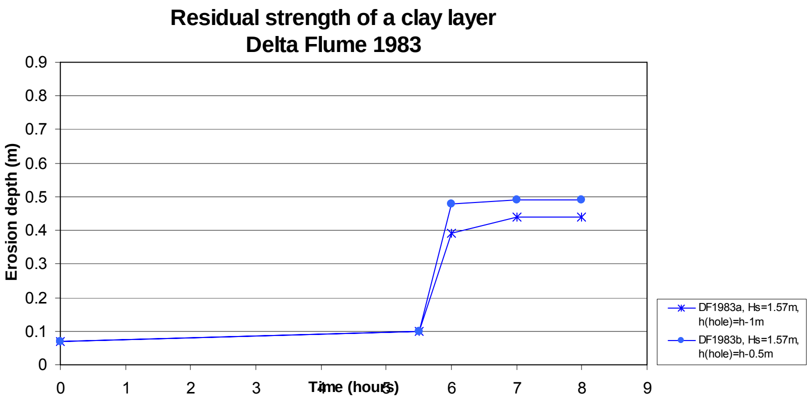


Figure 4.12 Development of erosion of the clay layer in two 0.07m deep holes in grass cover under a wave load of Hs=1.57m, obtained during the 1983 Delta Flume experiment to the strength of a Friesian sea dike.

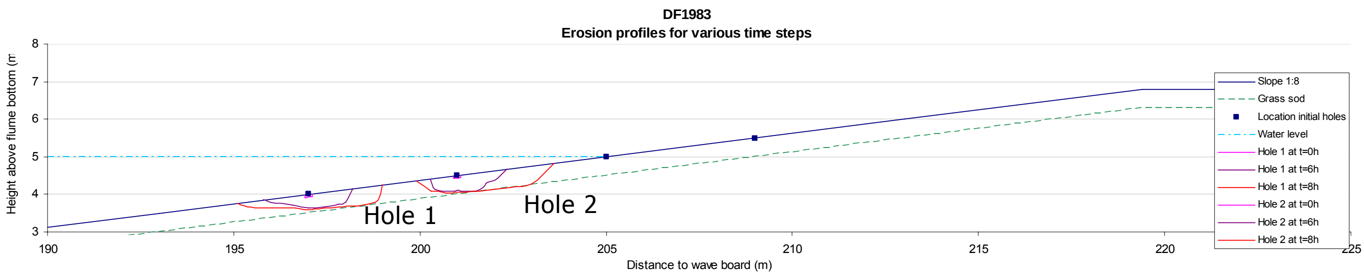


Figure 4.13 Erosion profile of the two holes of DF1983.

Conclusions

- The erosion developed in the wave breaking zone, just under the water level. In the wave run-up zone no erosion occurred. The erosion developed further upwards.
- The clay was not structured.
- Rapid erosion occurred between 5.5 and 6h.
- The gentle slope caused a smaller wave loading on the slope, because the waves broke on a water layer and not directly onto the slope.
- A hole of 0.07m deep in the grass cover is enough to decrease the strengthening of the dike that the grass provides and initiates erosion. This is just a small hole and can, in real dikes, be difficult to observe, so maintenance and inspection are very important.
- After removal of 0.07m of the top layer the erosion process starts. The whole top layer with grass was 0.5m thick and consisted of clay with a high sand percentage. This explains the deepening and widening of the holes during the wave loading with high waves. The clay type of the sublayer is clay with a low sand percentage, this could explain the slowing down of the erosion process around a depth of almost 0.5m. No erosion of the sublayer is observed.
- In this experiment, advanced damage developed due to points of application of the wave load in the clay layer under the roots.
- Holes of 0.07m deep were made in the grass cover. At this depth a root structure can still be present which can strengthen the clay layer.
- The grass cover around the holes can influence the erosion process. The erosion velocity in the holes can be reduced and this can improve the strength of the clay in the holes.

4.2.2 Delta Flume 1992 (DF1992G)

The grass cover from a dike in Blija, Friesland, was removed and built up in the Delta Flume, for experiments on a grass dike representative for the Netherlands. The research consisted of erosion tests, overflow tests and a test of the residual strength of the clay. The outer slope was constructed of a 0.9m thick clay layer with grass, a gradient of 1:4, and an asphalt toe construction to 2m above the flume bottom. At first short experiments with regular waves were performed which gave insight into the water pressure variations over time and space during less complicated circumstances. Next the model was loaded with regular irregular wave conditions for different water levels. The irregular wave conditions had a realistic loading situation and served therefore to provide insight into the real failure development.
[Smith, 1994a and b; Verheij, 1998]

Table 4.5 Properties of the Delta Flume experiment 1992, grass slopes

Properties DF1992G		
Thickness clay layer	d_c	0.8m
Thickness grass sod	d_g	0.05-0.1m
Water level	h	4.8m
Wave height	H_s	1.35m
Wave period	T_p	4.7s
Slope		1:4
Duration experiment residual strength	t	5h

Including grass cover
The experiment including the grass cover was done under wave conditions at the Waddenzee during high water and a storm, see Table 4.5. The first erosion was noticed after 9 hours. A hole of 0.12m deep (diameter 0.75m) had developed in the zone of the highest wave impact, at 1m below water level, which grew to 0.15m deep (diam 1.0m) after 11 hours. After this, the hole was repaired and protected from further erosion. The test was continued and after 17 hours another hole of 0.11m deep (diam 0.80m) had developed 0.5m under the water level. This is the location where most waves were breaking. See Figure 4.14 for the results.
A second experiment with the grass cover was done, with a low water level on the unloaded part of the slope. With irregular waves of 0.75m no erosion was noticed after 20 hours. (T_p=3.4s, h=3.5m) This experiment is not included in the graph.
On the basis of these experiments the three erosion zones were formulated, see 5.1, as well as calculation rules for the strength of grass covers, see Appendix II.
[Smith, 1994a and b]

The maximum erosion was observed on the underside of the wave impact zone. Thus it is apparent that not only the shear stresses induced by the flow velocities, but also the wave impact plays an important role as the maximum velocity is not experienced in this area. The high elasticity and the roots restrict the erosion. [Verheij, 1998]

Residual strength
The experiment of the residual strength of the clay layer was performed after the grass tests with the wave conditions given in Table 4.5 (same as the first grass experiment). After four hours a large hole of 0.40m deep had developed at the edge of the flume, just below the water level, which grew through the whole clay layer in one more hour. The development of the hole was probably distorted due to the edge of the flume. Therefore this result was stated to be representative only for transition areas, for example a hard cover layer of asphalt or a block revetment. The measured residual strength was (conservatively) estimated to be five hours.
Another place gave more reliable results for the resistance to erosion of the clay. Erosion of 0.25m was observed after five hours of wave loading at the location of 0.5m under the water level. (Erosion velocity of 0.05m/h) The erosion velocity and the location of the erosion are comparable to the results gained in the experiments including the grass cover. These values lead to an estimated residual strength of 16 hours. The experiment was however stopped after these five hours. See Figure 4.14.
Based on these two results the residual strength of a 0.80m thick clay layer under continuous loading with high waves was estimated to be 10 hours.
[Smith, 1994a and b]

The used clay in the cover layer was for the greater part well erosion resistant, it was determined to be little sandy clay, with a soil structure of angled blocks (dimensions of cm-dm) in the zone of 0.4-0.8m deep. Above this layer the top layer with root structure is present and the clay has a crumbly, porous structure, which is the finest in the upper 0.15m. [Kruse, 1998]

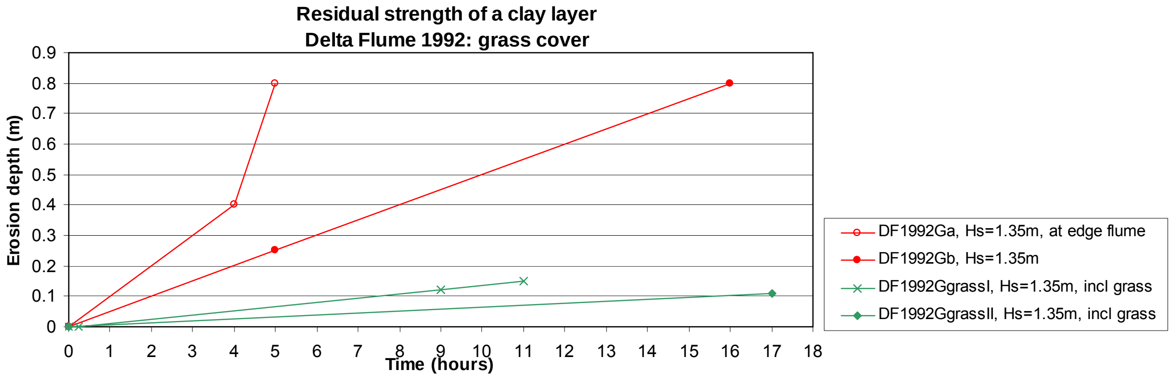


Figure 4.14 The erosion depth as a function of the time for two tests: the residual strength of a clay layer under a grass cover and on the grass cover itself.

Conclusions

- In the clay some soil structure was developed.
- The clay type was clay with a low sand percentage, only a little sand was present.
- The erosion developed just under the water level in the zone of the highest wave impact.
- The resistance to erosion with the presence of the grass cover was very high, with an erosion velocity of only a few millimetres per hour.
- The influence of the edge of the flume was large. The resistance to erosion of the rest of the cover was much higher. (Residual strength of 17h.)
- The start of the first erosion hole at the edge of the flume could maybe have been developed because of a weakness in the clay cover at this location. This erosion was later influenced by the edge of the flume.

4.2.3 Schelde Basin 1994

The experiments in the Schelde Basin of Delft Hydraulics in 1994 researched the strength of grass from river dikes under wave loading. Tests on the outer slope of dikes were executed for several types of grass with different management methods. See Table 4.6 for the properties of these experiments.

Table 4.6 Properties of the Schelde Basin experiment 1994

Properties Schelde Basin 1994		
Water level	h	0.8m
Wave height	H_s	0.31m
Wave period	T_p	2.5s
Slope		1:3
Duration experiment	t	60h

Erosion of the well rooted grass sods was just a few centimetres, of the moderate rooted sods a bit more than 0.1m and in places with hardly any roots, a hole of more than 0.2m developed very quickly. The maximum erosion velocity in the wave impact zone was 0.3mm/h for grass with a good sod quality and 2.3mm/h for a bad sod quality.
[TAW ErG, 1998]

Holes developed in the wave impact zone of 0.35m*0.35m and 0.10m deep. The erosion depth after loading was 0.06m.

Conclusions

- Hardly any erosion developed in the clay under the low wave loading.
- The wave height was too low to be representative for storm conditions at a sea dike. Therefore this experiment is not included in the comparison of Chapter 8.
- This experiment might be comparable with the other large scale experiments if it is possible to plot the erosion depth as function of the wave height.

4.3 Conclusions

- The erosion developed in the wave breaking zone, just under the water level in the zone of the highest wave impact. In the wave run-up zone no erosion developed.
- A gentle slope gives a smaller wave load on the slope, because the waves break on a water layer and not directly on the slope.
- A hole of 0.07m deep in the grass cover is enough to decrease the strengthening the grass provides the dike and to start erosion.
- The top layer with the grass erodes easily after removal of the grass. The clay in the sublayer is stronger and this slows down the erosion process around a depth of almost 0.5m. This was clearly observed in DF1983.
- Advanced erosion of the grass cover after the formation of a hole can develop due to the points of application of the wave load in the clay layer under the roots.
- The root structure can influence the residual strength of the clay layer under the grass layer, because roots can grow there strengthening the clay layer.
- Also the grass cover around the holes can influence the erosion process. This can reduce the erosion velocity in the holes and thus improve the strength of the clay in the holes.
- The resistance to erosion in the presence of the grass cover is very high, with an erosion velocity of only a few millimetres per hour.
- The influence of the edge of the flume is large. Resistance to erosion of the rest of the cover is much higher.
- Weak places in the clay cover can form a easy start of erosion.
- Good grass cover grows best in a not-so-cohesive clay layer, so if the strength of a grass cover is tested and holes are made, the clay below it will probably be of a high sand percentage. For a clay dike, the upper part of the clay cover has to be of clay with a high sand percentage in order to make it possible for vegetation to grow in it, however the lower part has to be of clay with a low sand percentage as this provides the strength against erosion.

5 Performed research of the strength of clay under wave loading

This Chapter provides a overview of research to the strength of clay under wave loading, performed in recent history. For further reading refer to the literature given in each paragraph. See also Appendix X, Appendix XI and Appendix XII.

5.1 Erosion zones along the slope

The results of the experiment in the Delta Flume in 1992 on grass slopes led to the definition of three erosion zones along the slope (see also Figure 5.1):

Zone 1 At a depth between $0.3H_s$ and $0.6H_s$ below still water level
The most erosion took place in this zone, a hole developed and the highest average erosion rate occurred.
Zone 2 Between still water level and a depth of $0.3H_s$
The average erosion rate was half of that in zone 1.
No hole developed.
Zone 3 Above still water level
Practically no erosion observed.

[Smith, 1994a]

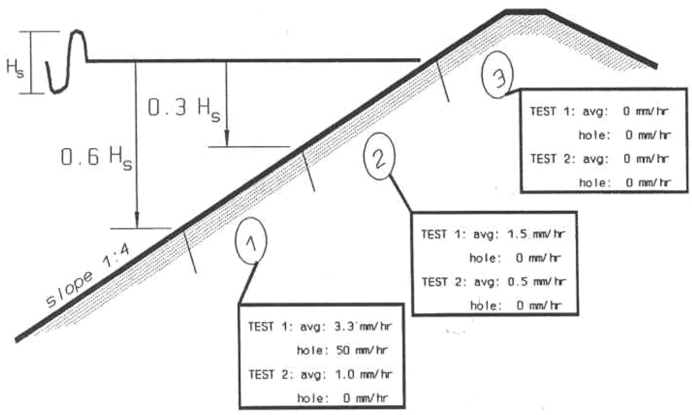


Figure 5.1 Erosion zones along the slope. [Smith, 1994a]

5.2 Modelling the erosion of clay under wave loading

On the basis of experiments of residual strength under stone revetments in 1992, Delta Flume 1992 DF1992S, it is tried to model the erosion of clay under wave loading, with the use of the results of the water pressure meters.

5.2.1 Water pressures

The water pressure gradients in the clay layer were expected to be an important factor for the erosion of clay under waves and soil structure development. Therefore the water pressures in the clay layer were followed during the experiments. Analysis showed that the water pressure gradients are most probably operational to the occurrence of damage to a clay layer with a soil structure. The water pressures in the clay are often high (sometimes during 1.5s an upward pressure was higher than 10 kN/m^3) and have a pattern that is similar to the water pressures of the wave load. However, there were no systematic trends observed in: the average water pressure over time, the

pressure differences in the water pressure meters, which were lying above each other, or in phase- and damping differences. The pressure distribution in the ground does not correspond with the distribution on the surface during wave impact. Sometimes the pressure deep in the clay is higher than just below the surface. It is suggested that the movements of the clay blocks are a reason for the bad correlation between the measurements in and on the slope. On the other hand, from the measurements it appears that regular upwards pressure gradients also occur which are higher than the dead weight of clay under water. The results of the water pressure meters are dependent on their direct location in the clay. Changes during the duration of the load lead to a better hydraulic connection with the surface of the clay. The erosion of clay however is not that fast. Cohesion and interlocking of the aggregates are suggested to be responsible for the delay of erosion and therefore it is stated that fatigue plays a role.
[Hofmann, 1993; Hofmann, 1995]

During a wave impact the water pressure meters give an increase in pressure readings. The increase propagates into the subsoil by a pressure wave, or the wave flows into the dike body through cracks in the clay. By a combination of these two factors the water pressure meters in the subsoil are expected to give a pressure increase which is correlated with the water pressure changes in the slope. If this correlation is complete it can be stated by the following formula:

$$f(t) = A * g(t+d) + B$$

f(t) *water pressure in clay (kN/m³)*
A *damping*
g(t) *water pressure on slope (kN/m³)*
d *time difference (s)*
B *constant*
[Hofmann, 1993]

5.2.2 Models

Several models were used in order to try to describe the residual strength of a clay layer. The efforts were concentrated on an analysis of the water pressures in the clay occurring during the Delta Flume experiments. The 1D models did not lead to satisfactory results, therefore a 2D model in combination with elastic storage and potential flow was suggested.

UDEC (Universal Distinct Element Code) –TU Delft (Faculty of Applied Earth Sciences)
Movements and stresses in a sample of discreet elements under the influence of internal stresses and external loads can be determined and followed using the numerical program UDEC. Also flow of a fluid between the elements can be modelled, which makes it useful when describing the water pressures in structured clay under wave loading. However, a dynamic calculation was not possible with the used version of UDEC and only a few simple calculations were possible. From this it can be concluded that structured clay can not be seen as a sample of loose elements but that there is a significant coherence and cracks can be assumed to retain their shape, because damping occurs in stiff cracks and damping was also visible in the Delta Flume experiments. The model shows how the forces develop around the clay elements. [Hofmann, 1995, p. 14]

UDEC is stated to be a very complicated, but good program to make a numeric model of the behaviour of structured clay under wave loading.

STEENZET/2 and PLUTO
Parallel to UDEC the programs STEENZET/2 and PLUTO were also used. In both programs it was explicitly assumed that structured clay can be represented by a continuous medium. The required parameters were determined by parameter tests which deliver bulk properties and thus an average behaviour of the clay.

STEENZET/2 is made for stone revetments, but was chosen because clay with a soil structure erodes in block shapes. With STEENZET/2 the water pressure distribution can be numerically solved in two dimensions with a consolidation equation. An overview can be made of the water pressure distribution in the slope under wave impact. The slope is schematized by several homogeneous layers lying above each other.

The 2-dimensional finite element program PLUTO can be used to determine deformations and plastic behaviour of soil under a load. This program is used to check if the gradients induced by the wave loading are sufficient enough to cause local failure of the clay. Some relevant water pressure distributions under wave impact calculated by STEENZET/2 were used to form the input of the PLUTO calculation. The results of PLUTO are elastic deformations due to this load.

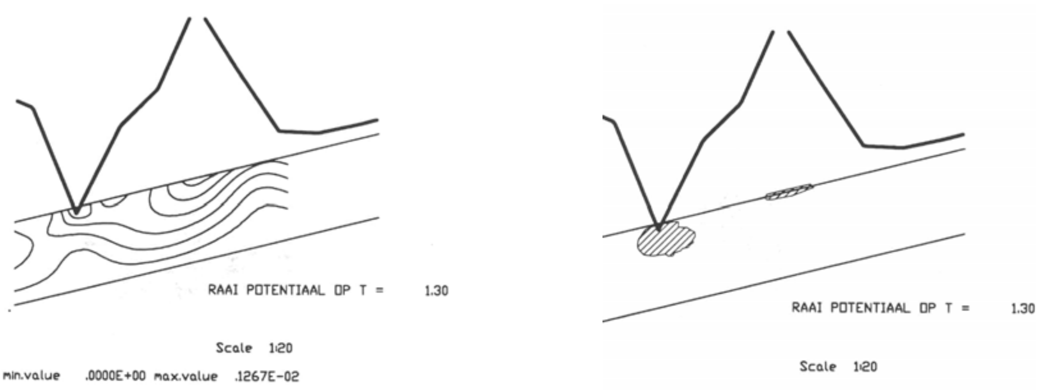


Figure 5.2 An example of the contour distribution of the elastic deformation during wave impact and the area where plastic deformation occurs during wave impact. Results of a PLUTO calculation.

See Figure 5.2 for an example of the contour distribution of the elastic deformation during a wave impact and the area where plastic deformation occurred during the wave impact.

Conclusions from the calculations

The failure mechanism of the clay layer was not supposed to be only the result of pushing out of the aggregates due to pressure differences over the depth, but as well due to sliding and moving out of the aggregates due to large gradients directed along the slope during wave impact. The water pressure at a certain time acting on the slope is directed perpendicular to the slope if the water pressures are hydrostatic. During the wave impact forces also arise due to the falling water mass on the slope. This load is directed vertically, which induces locally a load along the slope. To see if this assumption was right the calculated water gradients of STEENZET/2 were used as input for the program PLUTO. This failure mechanism was based on two assumptions:

- The water pressures in the clay are generated by a combination of potential flow and elastic storage.
- The structured clay cover can be represented by a homogeneous clay layer with modified parameters for the permeability, porosity, amount of air, elasticity modulus and Poisson ratio.

On the basis of the calculations it was established that the water pressure gradients during wave impact in combination with the measured soil properties are large enough to plastically deform the slope over a large area of the surface. This makes the clay locally unstable and therefore possible collapsible. (e.g. it can be assumed that these locations fail when the next wave hits the slope at the same location.) During the wave impact the acting gradients are the largest just after a wave impact, on a position just beneath the point of impact along the slope, and are directed out of the slope. These gradients are of a short duration and the outwards directed peaks are working on small areas just next to the pressure peak.
STEENZET/2 and PLUTO show that one wave can plastically deform a large area of the slope. However, the Delta Flume experiments show that one or several wave impacts are not enough to cause the slope to immediately fail over a large area. Removal of blocks takes place. Therefore it is expected that the material properties of the clay (cohesion and interlocking) are degraded under the influence of wave impacts and that at a certain time the coherence is not enough and individual blocks can be washed way. This is a process of fatigue.

These assumptions for the failure mechanisms are consistent with the observations of the Delta Flume experiments.

Hydraulic fracture test

With a hydraulic fracture test, the degradation of the clay properties on the basis of a continuing periodic loading was tested. Water is loaded under a pressure on one point in the clay. The influence of the water pressure gradients on the change in the structure of the clay was observed, especially on the pore system. The results were not satisfying and the fatigue of the structured clay could not be proved. It can however not be excluded that the set-up of these tests was responsible for this outcome, as a concentrated load in one point in the clay is largely different from wave impact.
[Hofmann, 1995]

5.2.3 Conclusions

The use of the models UDEC, STEENZET/2 and PLUTO have not led to satisfying and usable results. The model calculations however do give some conclusions which are comparable with the results from the Delta Flume experiment. These conclusions can therefore carefully be used for further research. These conclusions are:

- The wave impact has a large water pressure gradient directed parallel along the slope.
- This gradient is the greatest at a location just below the point of impact and is able to induce local failure of the clay layer.

5.3 Unprotected river banks

Research has been done to the erosion process of unprotected river banks. Unprotected river banks consist also, like clay layers on dike slopes, of a clay layer which delivers the resistance to erosion. Therefore this research to unprotected river banks could give good starting-points. From 1992 to 1996 research was done by CUR (Civieltechnisch Centrum Uitvoering Research and Regelgeving) to investigate erosion of unprotected river banks, in 2003 another literature study was done by GeoDelft. [CUR, 1993; Booster, 2003]

When considering this research it should be kept in mind that the waves from ships are different than waves at the coast. A wave coming form a ship is much lower and gives therefore a much lower load on the slope than wind waves which act on a dike during a storm. Present vegetation on the river banks can therefore contribute to the erosion resistance of the bank. A tidal range is not present. In addition to that wind waves imply an upper- and under flow, which are not present for ship waves because the wave duration is shorter and that mechanism has no time to develop.

The loads on a river bank are:

- Stationary and in-stationary flow
- Waves
- Water level variations (e.g. high water wave)

In CUR (1993) three main failure mechanisms are distinguished:

- Abrasion
- Pushing out of clay blocks
- Sliding

An overview of the different failure mechanisms occurring under different loading situations is given in Table 5.1. [Booster, 2003]

Table 5.1 Overview failure mechanisms for unprotected river banks related to the hydraulic load.

	Stationary flow	In-stationary flow + Waves	Water level variations
Abrasion ('slijperosie')	x	x	
Pushing out ('uitdrukken')		x	x
Sliding		(x)	x

Several attempts to model the erosion of river banks were performed, given in Appendix XI.

[CUR, 1993; Booster, 2004]

Conclusions

- Unprotected river banks and a clay layer on the slope of a sea dike obtain both their strength from the resistance against erosion during wave loading.
- Ship waves deliver however a much lower load on the slope than wind waves.
- For the erosion of clay under loading of ship waves three failure mechanisms can be distinguished: abrasion, pushing out of blocks and sliding.
- Up till now no good model is developed which determines the erosion of a river bank.
- There are no experiments performed to the erosion of river banks.

5.4 Determination of clay cover layer thickness by reduction of infiltration

The needed thickness of the clay layer is determined by the reduction of the infiltration. On the basis of serviceability the cover layer has to be a size Δd thicker than the crack depth d_c.

$d \geq d_c + \Delta d$ [m]

d needed thickness clay cover layer (m)
d_c crack depth (m)
Δd additional thickness (m)

The maximum crack depth is considered to be around the threefold of the shrinkage size, for the in Germany occurring climate circumstances. The shrinkage size V_s refers to the specific inclination of the soil due to shrinkage cracks. The shrinkage limits according to DIN 18122 enquire the difference between the soil volume at liquid limit water content and the soil volume after oven drying, related to the volume at liquid limit water content. On the basis of biotic macro pores a minimum root depth of 0.2m has to be assumed.

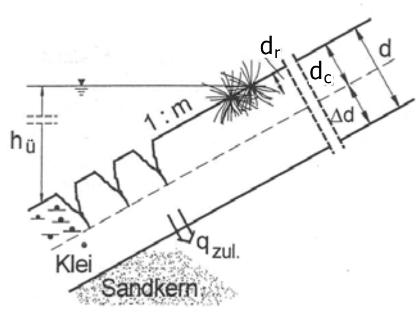


Figure 5.3 Structure of a clay layer

$d_c = 3V_s$

$d_c \geq d_r$ =0.2m

V_s shrinkage size
d_r minimum root depth

The additional thickness Δd is needed to reduce the pressure potential of the water level and the cover layer (d).
[Pohl, 2005; Pohl, 2006]

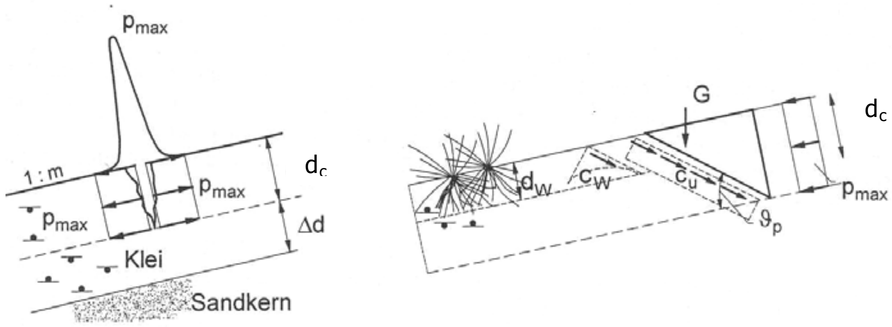


Figure 5.4 The additional thickness Δd is needed to reduce the pressure potential of the water level and the cover layer (d). [Pohl, 2005; Pohl, 2006]

See Appendix XII, Pohl (2005) and Pohl (2006) for more information.

Conclusions

- This method joins with the starting assumption for construction of the cover layer. The cracks depth is based on the shrinkage size v_s, which follows from German standards.
- The clay cover layer will be strong enough to withstand the pressure loads due to wave impact if the pore number does not exceed a certain value. This is depending on the installation of the clay.
- The thickness of the clay layer is not directly dependent on the wave height, because the wave height is a variable in the amount of compression needed of the clay, the pore number.
- This method assumes a significant influence of the cohesion due to the roots of vegetation. This joins with the theory that grass gives a significant contribution to the strength of the clay cover layer, but since it is still difficult to quantify this contribution, it will probably also be difficult to quantify the cohesion of the roots.

5.5 Empirical formula of the residual strength of clay

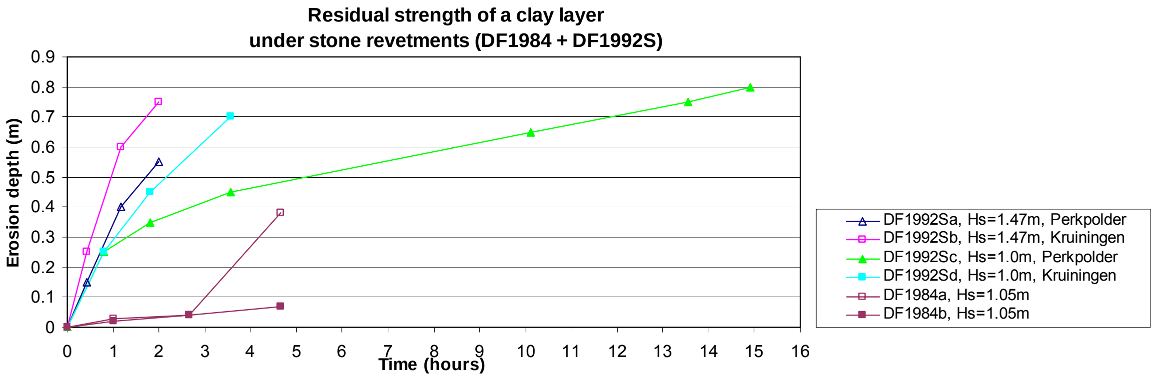


Figure 5.5 The combined results of experiments to the residual strength of clay under stone revetments from the Delfta Flume in 1984 and 1992; the erosion depth as a function of the time.

WL|Delft Hydraulics (2006b) determined an empirical formula for the erosion depth over time, based on reanalysis of the results of the experiments of the residual strength of clay under stone covers in 1984 and 1992. (See 4.1.1, 4.1.2 and Figure 5.5.) This was done because the residual strength was to be used to determine if the interlock forces of the stones could be included in the strength of a stone revetment. It was considered that the residual strength of the VTV was too conservative for this calculation and therefore a more realistic model was preferred. A distinction is made between structured and unstructured clay. DF1992S is structured clay, representative above the level 1m+MHW and DF1984 unstructured clay, representative below 1m+MHW. For the erosion of both types one formula was developed with different values for the clay type factor:

$$d_e = C_c * H_s \left(-6.9 + \ln(t)\right)$$

d_e erosion depth (m)
C_c clay type factor (-)
H_s wave height (m)
t time (s)

For the erosion depth a logarithmic relation of time was used. See Figure 5.6. On the basis of this graph it was stated that the erosion depth is approximately proportional with the wave height, because the erosion of the lower wave heights develops more slowly. In the past also a quadratic relation was suggested between the wave height and the erosion depth ($d - H^2$), in order to not overestimate the residual strength if the wave height is large. (H_s>2m) (Used in VNK calculations.)

The empirical parameter -6.9 fits with good results to the measurements; the other empirical parameter is dependent on the clay type, C_c, see Table 5.2.

The developed erosion hole in DF1984a is considered to be due to a lack of compaction. Therefore it is considered not to be of relevance for unstructured clay, because it is not representative for the coherence of clay after several years in a dike slope. This hole of 0.38m deep is not taken into account. If this hole would be included, the upper boundary for C_c of a clay type with a high sand percentage will be 0.13.

Table 5.2 Values for C_c for the different tests.

Experiment	Clay condition	Clay type	C_c (-)
DF1992S	Structured	Perkpolder clay	0.20
		Kruiningen clay	0.26
DF1984	Unstructured	Clay with high sand percentage	0.018-0.032
		Clay with low sand percentage	0.014-0.025

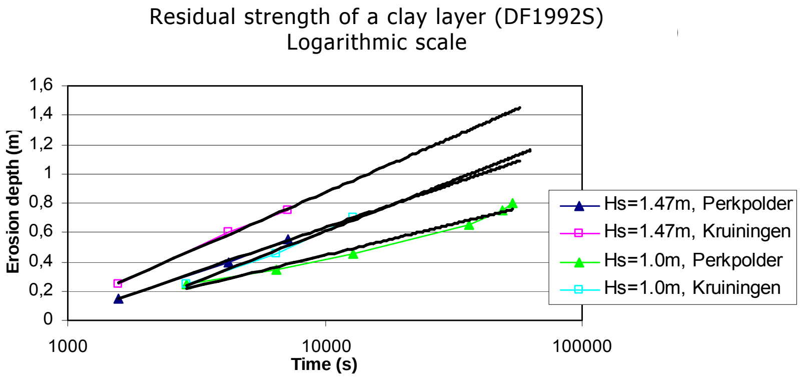


Figure 5.6 Erosion depth as a function of the time on a logarithmic scale.

To determine the residual strength the formula is rewritten into:

$$t_{rc} = e^{6.9 + c_{clay} \cdot \frac{d_c}{H_s \sin\alpha}}$$

t_{rc} residual strength clay layer (s)
c_{clay} constant, depending on the clay type = sinα/C_c, see Table 5.3
d_c thickness clay layer (m)
H_s significant wave height at toe of the dike (m)
α slope angle (°)

Table 5.3 Values for c_c for the different clay types.

Clay condition	Clay type	c_{clay} (-)
Structured	Perkpolder clay	1.22
	Kruiningen clay	0.94
Unstructured	Clay with high sand percentage	9-15
	Clay with low sand percentage	11-20

[WL|Delft Hydraulics, 2006b]

5.6 Voorschrift Toetsen op Veiligheid

The results of the Delta Flume experiments of 1992 of the residual strength of clay under stone revetments, DF1992S, are used in Leidraad Zee- en Meerdijken (TAW LZM) and Voorschrift Toetsen op Veiligheid (TAW VTV) to state a conservative testing method of residual strength of a clay layer. This method is used in the five yearly testing of the quality of the water defences. This is not a scientific method to determine the residual strength of the clay layer; it is a safe method used for determining the strength of the total revetment.

The residual strength of the clay layer (expressed over time) may be added to the strength of the stone revetment, if the stones and brick layers alone cannot deliver sufficient strength. This residual strength, expressed in a duration (in hours), is given in Table 10.4.

$$t_{r,top} + t_{r,clay} > t_{sm}$$

$t_{,r,top}$ residual strength of the top layer and brick layer (h)
$t_{,r,clay}$ residual strength of the clay layer (h)
$t_{,sm}$ normative duration of the load (h)

Table 5.4 Residual strength according to Voorschrift Toetsen op Veiligheid [TAW VTV]

Residual Strength of Clay Layer in hours

		below MHW +1m				above MHW +1m			
Resistance to erosion	Hs (m)	0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5
	Thickness clay (m):								
Low	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.7	2.0	1.5	1.5	1.0	2.0	1.5	1.5	1.0
	1.0	3.5	3.0	3.0	2.0	3.0	3.0	3.0	2.0
	1.2	5.0	4.5	4.5	3.0	4.5	4.5	4.5	3.0
High + Low	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.7	4.0	3.0	2.0	1.5	3.5	2.5	1.5	1.0
	1.0	7.5	6.0	4.0	3.0	6.5	5.0	3.0	2.0
	1.2	11.0	9.0	6.0	4.5	9.5	7.5	4.5	3.0

The residual strength of a wave height larger than 2m is nil.

6 Soil structure

The soil structure development is a very important factor in the erodibility of clay under wave loading. This appears from experiments to the residual strength of clay under stone and grass covers. A description of the development of a soil structure with the different influencing factors is given in this chapter. The knowledge given in this chapter is drawn up after an analysis of different reports of experiments on the residual strength of a clay layer under a stone cover in the Delta Flume in 1993 [Hofmann, 1993; Wouters, 1993], the residual strength of 11 dikes in Zeeland in 1995 [Kruse, June 1995, Aug 1995] and the Delta Flume experiment on grass dikes [Kruse, 2000]. This was combined with information given in the Technical Report Clay for Dikes [TAW Klei, 1996].

6.1 Development of a soil structure

Clay above the zone which is frequently under water has a soil structure as a result of regular changes in water content in this unsaturated zone and a result of environmental circumstances. If the clay is not kept in sufficient moist condition, atmosphere, flora and fauna affect the integrity of the clay and shrinkage and swelling results in a soil consisting of aggregates and blocks, a so called soil structure. This soil structure development usually decreases with depth. The aggregates still have a partly mutual coherence. The aggregates are divided by cracks in different sizes, sometimes too small to see with the naked eye. This soil structure in the unsaturated zone limits the resistance of clay soil against loading by high waves. A soil structure develops to a certain depth, depending on:

- Soil-water interactions
- Atmospheric conditions
- Time
- Degree of compaction
- Way of applying
- Type of cover layer on top of the clay
- Thermal stresses
- Animal behaviour
- Vegetation
- Composition of the soil

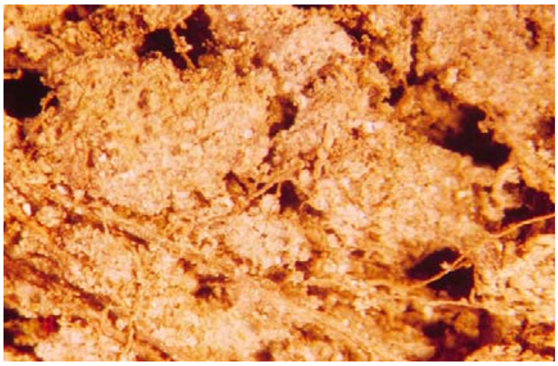


Figure 6.1 Soil structure in clay

The soil structure in the clay of a dike consists of cracks, animal tunnels and aggregates. The photograph in Figure 6.1 shows the fine soil structure in a grass sod. [TAW Klei, 1996]

Soil-water interactions and atmospheric conditions

The amount of soil structure development depends on the location in the clay and on the slope. Above high water level the soil structure the clay is clearly more developed and the amount of soil structure development reduces in depth. Cohesive soil in the unsaturated zone shrinks and swells, caused by suction stresses, which lead to compressive and tensional stresses, resulting in fractures. Tension cracks due to shrinkage and small slide planes due to swelling. The walls of these fractures are subjected to extreme wetting and drying conditions, resulting in further cracking. This process even addresses the fine microstructure of the clay. The fractures are directed vertically or at about 40° to the vertical, even under steep slopes. A strong, clear soil structure develops under constant change of shrinkage and swelling. The developed aggregates have little coherence. A fine structure of small cracks develops due to fast changes in water content, like rain. If the clay is wetted regularly by the tide, like clay under high water level, only in the upper centimetres to decimetres some soil structure develops.

Time

The soil structure develops to 0.5-3m above the ground water table. If deep roots of vegetation are present this develops deeper. In the upper 0.1-0.3m this soil structure has completely developed within 1-3 years in the unsaturated zone in a temperate climate to millimetre-sized blocks which are loosely piled up, and therefore it does not contribute to the residual strength. The permeability of a clay layer on a dike decreases to $3*10^{-5}$m/s within these few years. This layer can easily erode way by waves smaller than 0.5m. Well compacted clay is structured to 0.3m-0.5m depth after a few years and to 1.6m-1.8m depth after 50 years. For practical calculations INFRAM (2003) uses a growth in soil structure development of 10mm per year.

Degree of compaction
The influence of the degree of compaction became clear in research to soil structure development under stone covers which has been performed on 11 dike slopes in Zeeland in relation to the residual strength during wave loading and gulley formation under stones. Compacted clay has high resistance against erosion and a low permeability, if the material is kept in sufficient moist condition. The cohesion is large and the permeability is low. It can withstand waves of 2m for days sustaining only minor wear. In well compacted layers of cohesive soil at more than 0.3-0.4m depth hardly any cracks due to shrinkage and swelling have been developed, especially if the slope is directed to the North. These well compacted layers can, if they are not too thin, deliver an important contribution to the residual strength. Some old dike slopes were found as a part of the clay layer of a dike. These old dikes can consist of very compacted cohesive soils which deliver a very high contribution to the residual strength of the dike. (More than 24 hours for the upper 1-2m at waves of 1.2-2m. [Kruse, June 1995]) This is besides compacting, probably also because of mineralogical conversions in the layer.

At many locations however, the soil was not or just little compacted after installing and will therefore probably not contribute much to the residual strength. This soil consists of clumps with little coherence and large holes in between and the combination of high permeability and fine-grained soil intensifies the soil structure development. Around the high water line, the accessibility for compacting or dry processing is difficult. Therefore the applied clay around this level can have a lesser residual strength.

A 1.5m thick layer of well applied and suitable clay will probably deliver a very high residual strength during tens of years.

Way of applying
If clay with a higher water content, than is in accordance with the consistency limits, is applied above the phreatic plane, the water content will decrease till it is in accordance with the suction stress from the surroundings. This decrease leads to a shrinkage and volume decrease. Cracks develop in a pattern of prisms (0.1-0.2m) within at least 8 years. In the upper part these cracks can be homogenised by animals and roots, but in the lower zones, the cracks remain, see Figure 6.2.

Figure 6.2 Soil structure in a dike. The influence of animal burrowing and root penetration to 0.7m deep. (The structure underneath is a result of changes in moisture content.) [TAW Klei, 1996]

Type of cover layer
In the upper decimetre under a grass cover a dynamic development takes place of constant forming of new aggregates, which are constantly affected. This results in a strong soil structure of small aggregates (mm-cm), which are connected by roots and is developed within a few years.
At greater depth below grass, or under stone covers less visible aggregates develop with relative large dimensions (10cm), which have a certain mutual cohesion. Just under a stone revetment a soil structure is often present. After five years a recognizable soil structure is developed under stones, which is present in the whole, usually 0.8m thick, clay layer after 10 to 15 years.

Thermal effects
Thermal effects such as wind and temperature differences due to sunshine are of influence on the soil structure development. More temperature differences on the slope will result in more shrinkage and thus more soil structure development.

Animal behaviour and vegetation
Animal digging in the dike results in a network of holes and large pores where surface water can infiltrate in very quickly. The supply of oxygen and water increases the soil structure development significantly. Because of the oxygen supply vegetation growth is made easier, which loosens the soil and thus also increases the soil structure development. On the other hand can roots also bind aggregates, which increases the coherence and thus the strength of the soil, but it does not decrease soil structure development.

Composition of the soil
Sand inclusions in the clay enlarge the permeability and reduce the coherence of the clay. The supply of oxygen and water is increased and thus the soil structure develops more easily. If the cohesive layers are very closed below a level of 0.3-0.4m, a soil structure is often not present. The presence of an old dike body in the dike can increase the residual strength of a clay layer considerable, because this can be a very compacted body with hardly any soil structure. (If a sand layer is constructed between the old clay core and the clay cover layer sliding can occur if the clay layer is affected and a fast lowering of the outer water level takes place. [Kruse, June 1995]) A core of sand can increase the soil structure development by dehydration from beneath. If a thin clay layer (less than 0.7m) is present on sand, a soil structure develops fast over the whole depth.

The type of clay also is of influence on the soil structure development. Clay with a low sand percentage is more erosion resistant and is also more resistant against soil structure development. Due to the aggregate structure, the properties of the soil are different than the properties of the individual aggregates. Therefore the use of heavy clay with a low sand percentage and thorough compaction over the whole thickness of the clay layer is important for the residual strength of the layer.

[Hofmann, 1993; Kruse, June 1995, Aug 1995 and 2000; TAW Klei; Wouters, 1993]

6.2 Influence on the permeability and coherence

Due to the development of a soil structure the permeability of the whole clay layer (the bulk permeability) increases and the soil structure development reduces by compaction and settlement. The permeability of the individual aggregates is different, smaller, than the permeability of the whole clay layer. The bulk permeability of the clay is proportional to the number of cracks and holes, and increases with the third power of the width of the cracks. (The bulk permeability increases if the number of cracks increases.) Due to different layers and irregularities in a dike a variation of different soil structures, and thus permeabilities develop.

A well compacted clay layer has after application a permeability of 10^{-6} m/s, but this increases already within a year. The permeability of a clay layer in the Netherlands in the unsaturated zone of a dike revetment on a sand core is approximately $3*10^{-5}$ m/s. In the laboratory values of 100 to 1000 times lower are found. The permeability found by triaxial tests is much lower than in the field, because of large cracks and disturbances which are not measured in the laboratory.

The coherence of the clay mainly determines the strength of the clay. If the soil structure development is less clear, the coherence (cohesion and interlock) is higher. The effects of the water movements and pressure build-up in the cracks are not unambiguously related to the permeability. A higher permeability leads to less upwards pressure differences in the clay, while on the other hand a small permeability restricts the movement of soil particles from the clay layer.

[Hofmann, 1993; Kruse, June 1995, Aug 1995, 2000; TAW Klei]

6.3 Conclusions

The development of a soil structure is a very important factor for the strength of a clay layer under wave loading. The strength of clay under wave loading will reduce with the development of a soil structure and this clay will erode faster. The soil structure development has an influence on both the strength and the load. A soil structure develops as a result of regular changes in water content in the unsaturated zone and of environmental circumstances. Shrinkage and swelling of the soil induce cracks and the development aggregates and blocks of clay of different sizes.

A soil structure will always develop in time, but the amount of development depends on many factors and is difficult to describe or predict quantitatively. Important influences on restricting soil structure development are the way of compacting the clay and the composition of the soil. Compacted clay has a low permeability and therefore a high resistance against erosion, if the material is kept in sufficient moist condition. It can withstand waves of 2m for days sustaining only minor wear. This was witnessed by several experiments on revetments. A soil structure develops easily if sand is present. The use of heavy clay with a low sand percentage and thorough compaction over the whole thickness of the clay layer is important for the residual strength of the layer.

The permeability of a clay layer increases with a soil structure development, and therefore the properties of the whole clay layer are different form the properties of the aggregates. This makes structured clay an inhomogeneous material.

In general, a well developed soil structure is after time present in every clay sublayer, but there is a large variation in the amount of development and thus in residual strengths. Preventing the soil structure formation is thus a great contribution to the resistance of the clay against erosion. This can in theory be done by making sure the clay does not dehydrate, but is practically not feasible. Therefore insight into the development of a soil structure is of great importance in order to obtain insight in the behaviour of clay under wave loading.

Part 3

Analysis of the behaviour of clay under wave loading

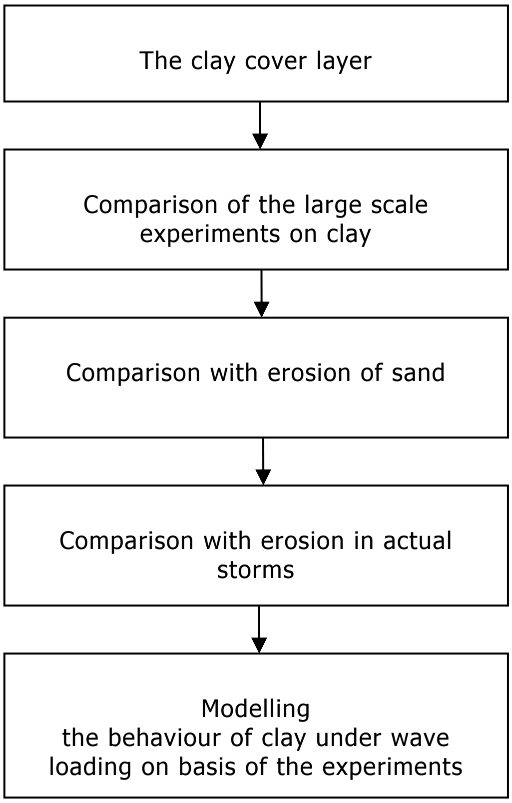
This third part of the report gives an analysis of the behaviour of clay under wave loading, based on a comparison of the large scale Delta Flume experiments on clay.

Furthermore, these Delta Flume experiments on clay are compared with the erosion behaviour of sand.

The experiments on clay and sand are compared with observation of outer slope erosion in actual storms.

A semi quantitative model to the behaviour of clay under wave loading is developed as part of this research, which is based on the conclusions of the analysis of the experiments on clay.

Part 3
Analysis of the behaviour of clay
under wave loading



7 The clay cover layer

Failure can be described by a reliability function: $Z = R - S$, in which R is the resistance to failure (the strength) and S is the load. If $Z < 0$ failure occurs. In this case, failure is the erosion of the clay revetment, layer 4.

7.1 Schematisation of the revetment

A schematisation of the clay revetment gives better insight in the structure of the revetment and the different layers. Each of the layers has different properties and will erode in a different way. The revetment is divided in five different layers. See Figure 7.1.

1. Vegetation on top of the soil
2. Vegetation layer in the soil
3. Structured clay
4. Unstructured clay
5. Sand

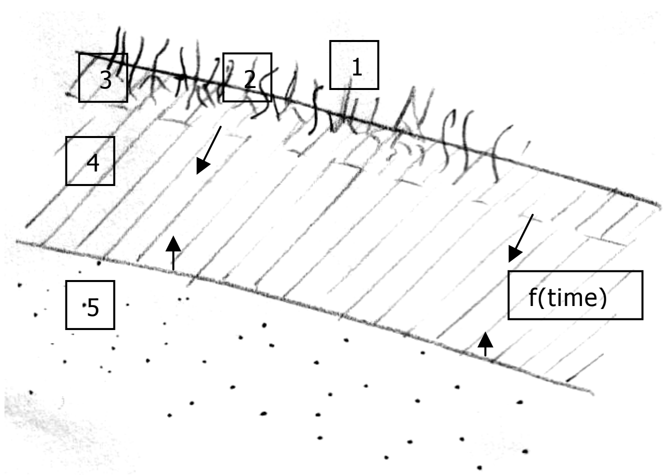


Figure 7.1 Schematization of a clay revetment.

1. Vegetation

The leaves of the vegetation (grass) can protect the clay cover from eroding by flowing water.

2. Vegetation layer in the soil

The vegetation layer in the soil forms the top layer of the revetment. A soil structure will be present in this layer, due to several factors (see chapter 11). The roots of the vegetation deliver the strength in this layer. The roots intensity decreases with depth and the strength depends on the quality of the grass cover, see Figure 7.2. The dimension in depth of this layer thus depends on the development of the roots in depth.

3. Structured clay

The structured clay layer develops further under the top layer of the vegetation. This layer will erode easily under wave loading and can therefore not deliver a large contribution to the strength of the clay revetment. The soil structure development increases as a function of time, so also the depth of this layer will develop as a function of time.

4. Unstructured clay
This thick clay layer with a low sand percentage delivers the strength of this revetment. Because of its thickness and the structured clay layer on top, this layer is supposed not to dehydrate. The thickness of this layer will be reduced in time, because of the soil structure development which develops from above and also, in smaller amount, from the sand below.

5. Sand
The core of the dike, which is placed under the protective clay layers, consists of sand. The sand will erode very fast under wave loading and this erosion will easily lead to breaching of the dike. Because the sand is very permeable it can induce dehydration of the clay layer above, and therefore weaken the unstructured clay from beneath.

The starting point of this type of shoreline protection is that the strength is delivered by the thick unstructured clay layer. The roots of the vegetation deliver additional strength. The structured clay has a small strength and can delay the erosion process a little, but is most important to cover the unstructured layer from soil structure development.

The soil structure development is dependent of the time. It can therefore be stated that layer 2 grows over time and thus layer 3 decreases in time. This could possibly reach a certain limit value.

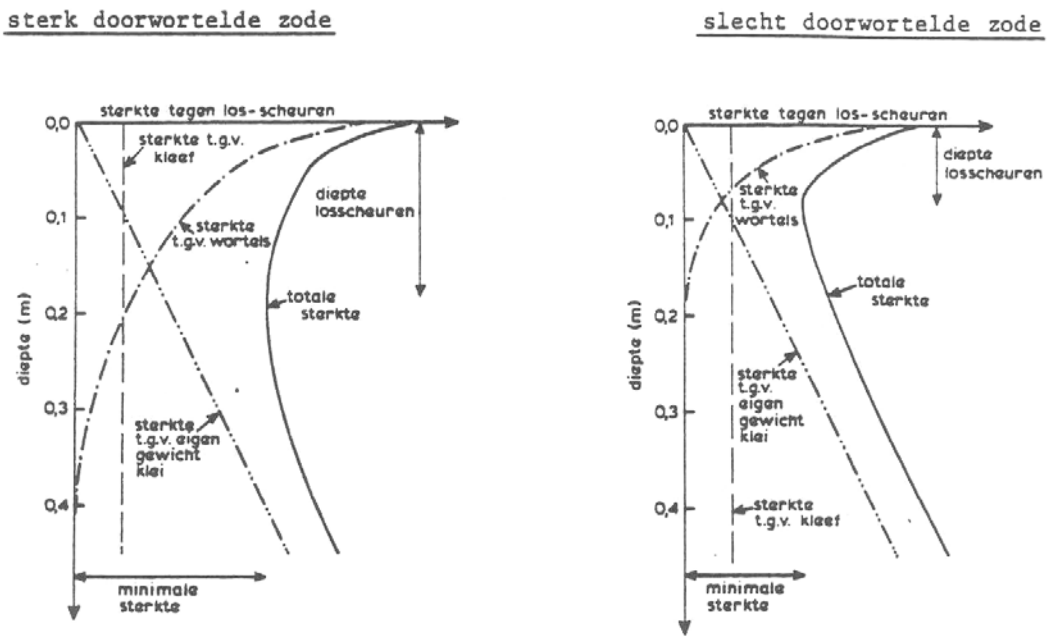


Figure 7.2 Strength of the roots of the vegetation top layer.

7.2 Erosion of clay

7.2.1 Load

Looking more closely to the forces occurring during a wave movement on a dike, also other forces than the large wave impact are present. These forces could also contribute to the erosion of the clay layer.
The oscillating wave movement induce water pressures on and from the slope, as well as a back and forth moving pressure along the slope.
If cracks are present the surface which are subjected to the wave movements becomes much larger, and these pressures will have more effect.

A wave, a falling water mass, can give a vertical load on the slope, which induces than locally a load directed along the slope.

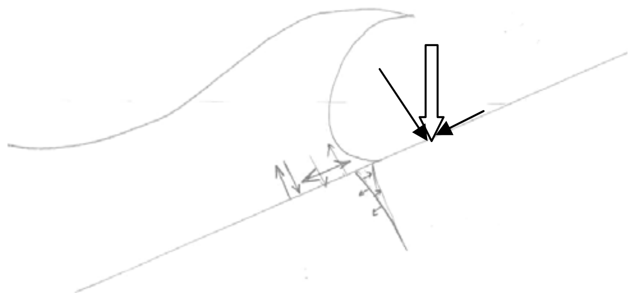


Figure 7.3 Forces on a slope of clay with cracks during wave loading

7.2.2 Erosion

Failure of cohesive soil is not only an erosion process of removal of individual grains, but aggregates of different sizes can be removed from the cover layer and even sliding of large parts can occur. These failure processes result from research and observations of erosion of cohesive material (studies to residual strength of clay under stones covers, strength of grass cover layers and erosion of unprotected river banks). The failure mechanisms of a clay cover are:

- Erosion of small pieces (pieces of $10^{-3} - 10^{-2}$m)
- Block erosion (pieces of $10^{-2} - 10^{-1}$m)
- Sliding (pieces of 10^{-1}–several meters)

These failure mechanisms influence each other, as is shown in Figure 7.4.

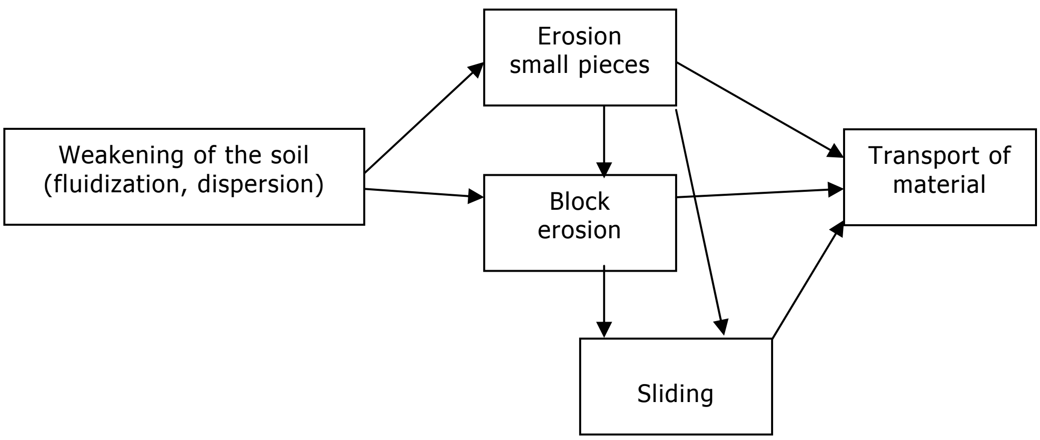


Figure 7.4 Overview and interactions of the failure mechanisms of a clay cover layer

Erosion of small pieces
Erosion of small soil pieces is a form of wear. If water flows along a surface a shear force acts on this surface and on its projections and irregularities. This results in a resistance, which is for non-cohesive material, like sand, a measure for the strength of the soil. Due to cohesion the strength of a cohesive soil is larger, but the aggregate structure weakens the soil and makes it possible for the flowing water to remove small particles. This process of wearing out of the soil can be divided in two mechanisms, dispersion and abrasion, which are relatively slow processes and therefore only relevant for erosion on the longer term.

Dispersion is the attraction of water by cohesive particles which leads to the release of these particles from the cover layer. The particles are so to say dissolved in the water. This process is only relevant for the erosion on the long term. This will only be of relevance in the parts of the slope which are permanently under water, because else this mechanism has no time to develop to significant erosion.

Abrasion is the removal of particles and small aggregates by drag and small lift forces, which are exerted by the flowing water on the surface. The sensitivity to erosion is determined by the fine scale soil structure, which is partly dependent on the composition and properties of the material. Turbulent running water can remove particles, small aggregates and sand, because the connection between particles will be affected due to constant change in direction of the moving water. (This relatively slow erosion process is also responsible for the gullies under concrete block revetments.)

Block erosion
Erosion in the form of blocks takes place in soils with a developed soil structure. Large aggregates can be lifted from the surface or fall out if the slope is steep. Blocks are removed due to water pressure gradients in- and on the slope induced by wave impact. Due to the wave impact, water not only flows over the surface, but also through the soil. This erosion develops often rapidly because of the small weight of the aggregates, but mainly because removal of one block undermines the stability of the surrounding blocks, which can lead to further removal and instability of the slope.

Sliding
Wave impact on a slope can lead to loss of stability, which induces movement and deformation in the soil and the loss of the internal coherence. This moved soil can easily be removed by the waves. During a wave load pressure gradients occur in the soil which decrease on one hand the strength of the soil (loss of effective stress) and on the other hand form an activating mechanism for movement of the soil. The impact of the wave tongue can form an extra urging force. Because of the initial damage steep slopes occur with little stability through which damage expands relatively fast and further sliding occurs.

wave impact -> pressure gradients -> decrease strength soil + induce movement
-> wave impact + steep slopes -> sliding

These failure mechanisms can be divided over the different locations on the slope:
- Downwards of the wave impact zone: Wearing erosion and sliding.
- In the wave impact zone: Wearing erosion, block erosion and sliding.
- Above the wave impact zone: Sliding.

A complicating factor is that there are many interaction possibilities between the failure mechanisms and the different zones. For example:
- Wearing erosion downwards of the wave impact zone can lead to sliding downwards, in and above the wave impact zone.
- Sliding lead to differences in the shape of the profile and thus different erosion behaviour. (Counts for every zone.)
- Also wearing erosion and block erosion can lead to a different profile and thus different erosion behaviour.

Besides these three erosion mechanisms, also other factors can induce erosion. Branches and garbage drifting along the shore can hit the clay cover layer and induce erosion of soil pieces. Especially during a storm an object can be thrown at the cover layer and induce significant damage.

[Kruse, 1998; Kruse, 2000; Booster, 2003]

7.3 Relevant parameters

Several parameters form relevant factors in the determination of the failure of a dike with a clay revetment. These factors can be divided into factors which are relevant for the load and factors which are relevant for the strength. These factors are listed below.

Load
- Wave height
- Wave period
- Wave steepness
- Breaker type
- Storm duration
- Water level (Water level development during storm)

Strength
- Clay type – Resistance to erosion of the clay
- Thickness clay layer
- Soil structure development: permeability of the layer, shape solidity f(time, weather conditions)
- Slope of the dike
- Foreshore (height and width)
- Roughness of the revetment
- Resistance against sliding
- Cohesion
- Consolidation (on the long term)
- Berm (height and width)
- Quality of the vegetation layer, if present.

7.4 Weak locations along the slope

For an accurate description of the erosion process it is important to know the influence of weak locations, because the erosion of a clay layer will start at the weakest location on a dike slope. A weak location in the clay slope of the dike is a location with a sand lens or a clearly present soil structure which reduce the strength of the cover layer.

A weak location has a lower erosion resistance and therefore the chance on erosion is larger. If the upper part of the clay layer is damaged, the wave has attraction points from where the erosion can continue. Erosion can easier start at irregular points on the surface, like holes, piles or vegetation lumps. The weakest location in the clay layer is therefore normative for the strength the dike. The properties of a weak location are however difficult to define and there will differ per weak location.

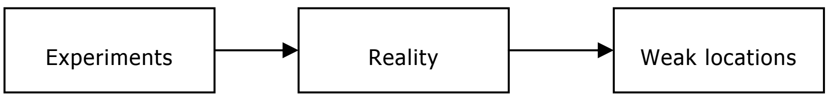
If the distribution of these weak locations along the dike slope is known, a probabilistic approach can be made for the strength of the cover layer. There is still not much known about the distribution of weak locations in a dike.

For the Delta Flume experiments it is not known if the relatively small width of the test section contains weak locations. If weak locations are not present in the Delta Flume, the erosion rate of a perfect clay layer is determined. If however weak locations are present, the test results will be more representative for the real situation on dikes. But if present, the actual conditions of these weak locations are still unknown.

The determined semi quantitative model is based on the Delta Flume experiments. If the experiments were performed on clay with weak locations, this presence of weak locations is thus also included in the model. If weak locations are not present in the experiments, the erosion rate of a real clay layer, which contains weak locations, will be higher than determined by the semi quantitative model. Even if the weak locations are included in this model, the properties of a weak location are not included, because they are unknown and the accuracy of the model is still unknown.

The erosion process of clay can accurately be described if:
- The distribution of the weak locations on the slope is known.
- The properties of weak locations are known.
- The influences on the erosion rate and erosion development of the clay layer are known.

For modelling purposes, the weak location can be seized in a parameter which influences the erosion rate. In this way the model determines the erosion development over time of the intact clay layer and the erosion rate increases depending on a certain distribution of weak locations.



Lange scale experiments in the Delta Flume to the strength of clay under wave loading.

What is the strength of clay on a real dike during normative loading conditions:

‘Real’ experiments →

Even if the real strength is known:

The weakest location is normative for the strength the dike, not the calculated strength at a certain location along the slope.

8 Comparison of the large scale experiments on clay

8.1 Reasons for a comparison of the large scale experiments on clay

In recent history several experiments to the residual strength of clay under wave loading have been performed and assumptions on the erosion behaviour and erosion rate of clay have been made. However, there is still little knowledge on the behaviour of clay under loading by currents and waves. Especially quantitative erosion descriptions are unavailable. In 'Klei voor Dijken' (TAW Klei, 1996) a subdivision of clay in three erosion categories has been made, with the sand percentage of the clay as a main influencing factor, but a quantification of the erosion rate of the three erosion categories is not given.

Several attempts have been made to describe the erosion of clay under wave loading, both qualitatively and quantitatively, based on experiments. However, these descriptions are all based on just a few experiments, for example:

- An empirical formula for the residual strength of a clay sublayer has been formulated by WL|Delft Hydraulics [WL|Delft Hydraulics, 2006b]. This formula is based on the results of two Delta Flume experiments on the residual strength of clay under stone revetments and had in total six results. (See 5.5).
- A description is made of erosion zones along the slope, based on the Delta Flume experiment on grass slopes (DF1992G) [Smith, 1994a] (See 5.1)
- Modelling attempts with the models UDEC, STEENZET/2 and PLUTO have been preformed based on the data obtained from the Delta Flume experiments of 1994 to the residual strength of clay under stone revetments. (See 5.2.2)
- The current design methods for clay revetments are based on results of the Delta Flume experiments on the residual strength of clay under wave loading of 1992 (DF1992S). [INFRAM, 2003; Handleiding Ontwerp]

For this research the choice is made to investigate and compare all performed large scale experiments which involve clay under wave loading. This is done in order to give a complete overview of all large scale and relevant experiments and not to make other assumptions on the behaviour of clay like has been done several times in recent history. This can be used to verify if a comparison of these experiments will result in new valuable conclusions and to validate existing conclusions and assumptions on the basis of more data.

In this chapter the results of all experiments about the behaviour of clay under wave loading are reanalysed and compared. Existing theories are validated with the data of all large scale experiments.

The experiments on the residual strength of clay under grass sods are included in the comparison, because on a clay dike vegetation growth will always develop. Therefore a little influence of some roots which still can be present in the clay after removal of the grass layer is not a problem. It can even be more representative for clay dikes.

Also the results of the experiments on grass covers themselves can lead to new insights, because than even more data is obtained to compare and the influence of the grass cover is on the strength of a slope in comparison with the bare clay layer can be determined.

8.2 Overview of all large scale experiments

Before making the comparisons, a short overview of all the experiments, as described in Chapter 5 are given. The tables with the properties of the experiments are repeated below and an overview of the clay properties with some experiment properties and remarks are given in Table 8.1. The results of all the experiments are combined in one graph in Figure 8.1.

Experiments on stone revetments:

Properties DF1984		
Thickness clay layer	d_c	0.80m
Wave height	H_s	1.05m
Wave period	T	12s
Slope		1:3.5
Duration experiment	t	4.40h

Properties DF1992S		High water level	Low water level
Thickness clay layer	d_c	0.8m	0.8m
Water level	h	5.0m	3.5m
Wave height	H_s	1.47m	1.0m
Wave period	T_p	4.9s	4.2s
Slope		1:4	1:4
Duration experiment	t	2h	15h

Experiments on grass:

Properties DF1983		
Thickness clay layer	d_c	1.0m
Thickness grass sod	d_g	0.5m
Water level	H	5.00m
Wave height	H_s	1.57m
Wave period	T_p	5.26s
Slope		1:8
Duration experiment	t	8h
Location holes	h-1m	4.00m
	h-0.5m	4.50m

Properties DF1992G		
Thickness clay layer	d_c	0.8m
Thickness grass sod	d_g	0.05-0.1m
Water level	H	4.8m
Wave height	H_s	1.35m
Wave period	T_p	4.7s
Slope		1:4
Duration experiment	t	5h

Remarks for Table 8.1:

- The values of the clay type are based on the reports of the different experiments. Because some experiments have been performed a long time ago, or because not all data is available, the subdivision in the three erosion categories is not made in every case. Clay with a low sand percentage is assumed to be similar to category 1 clay and clay with a high sand percentage to category 3 clay. See also the text box on clay types at the beginning of part 2.
- The clay condition is in these tables refers to the extent of soil structure development. This is based on information in reports of the different experiments. However, because all these reports have different authors and no quantitative values have been given, the comparison could be slightly different from reality.
- The Iribarren parameters have been calculated for each experiment in order to gain insight into the differences in wave conditions and breaker types of the experiments.

Table 8.1 Overview of properties of the large scale experiments on the strength of clay under wave loading, including two experiments on grass slopes.

<table>
<tr><th>Name experiment</th><th>H_s (m)</th><th>T (s)</th><th>Water level above flume bottom (m)</th><th>Slope</th><th>Duration experi ment, t (h)</th><th>Clay type</th><th>Clay condition</th><th>Influence vegetation</th><th>Other</th><th>Irribarren para meter</th></tr>
<tr><td>DF1984a</td><td>1.05</td><td>12</td><td>5.56</td><td>1:3.5</td><td>4.40</td><td>High sand percentage</td><td>Unstructured</td><td>No</td><td rowspan="2">Regular waves</td><td>4.2</td></tr>
<tr><td>DF1984b</td><td>1.05</td><td>12</td><td>5.56</td><td>1:3.5</td><td>4.40</td><td>Low sand percentage</td><td>Unstructured</td><td>No</td><td>4.2</td></tr>
<tr><td>DF1992Sa Perkpolder</td><td>1.47</td><td>4.9</td><td>5.0</td><td>1:4</td><td>2</td><td>Category 2 (Moderate)</td><td>Fine soil structure, some sand lenses</td><td>No</td><td></td><td>1.3</td></tr>
<tr><td>DF1992Sb Kruiningen</td><td>1.47</td><td>4.9</td><td>5.0</td><td>1:4</td><td>2</td><td>Category 1 (Low sand percentage)</td><td>Well developed soil structure</td><td>No</td><td></td><td>1.3</td></tr>
<tr><td>DF1992Sc Perkpolder</td><td>1.0</td><td>4.2</td><td>3.5</td><td>1:4</td><td>15</td><td>Category 2 (Moderate)</td><td>Fine soil structure, some sand lenses</td><td>No</td><td rowspan="2"></td><td>1.3</td></tr>
<tr><td>DF1992Sd Kruiningen</td><td>1.0</td><td>4.2</td><td>3.5</td><td>1:4</td><td>4</td><td>Category 1 (Low sand percentage)</td><td>Well developed soil structure</td><td>No</td><td>1.3</td></tr>
<tr><td>DF1983a</td><td>1.57</td><td>5.26</td><td>5.0</td><td>1:8</td><td>8</td><td>In grass layer: High sand percentage
In sublayer: Low sand percentage</td><td>Unstructured</td><td rowspan="2">Yes
Holes were made on beforehand in the grass</td><td rowspan="2">Only erosion in clay of grass layer(0.5m, high sand %), clay of the sublayer with low sand % was still intact</td><td>0.7</td></tr>
<tr><td>DF1983b</td><td>1.57</td><td>5.26</td><td>5.0</td><td>1:8</td><td>8</td><td>In grass layer: High sand percentage
In sublayer: Low sand percentage</td><td>Unstructured</td><td>0.7</td></tr>
<tr><td>DF1992Ga</td><td>1.35</td><td>4.7</td><td>4.8</td><td>1:4</td><td>5</td><td>Low sand percentage, little sandy</td><td>Soil structure</td><td rowspan="2">No, but some roots can still be present</td><td>Hole developed at side flume, influenced erosion process</td><td>1.3</td></tr>
<tr><td>DF1992Gb</td><td>1.35</td><td>4.7</td><td>4.8</td><td>1:4</td><td>5</td><td>Low sand percentage, little sandy</td><td>Soil structure</td><td></td><td>1.3</td></tr>
<tr><td>DF1992G grassI</td><td>1.35</td><td>4.7</td><td>4.8</td><td>1:4</td><td>11</td><td>Low sand percentage, little sandy</td><td>Soil structure</td><td>Yes</td><td rowspan="2">Test on grass cover itself</td><td>1.3</td></tr>
<tr><td>DF1992G grassII</td><td>1.35</td><td>4.7</td><td>4.8</td><td>1:4</td><td>17</td><td>Low sand percentage, little sandy</td><td>Soil structure</td><td>Yes</td><td>1.3</td></tr>
</table>

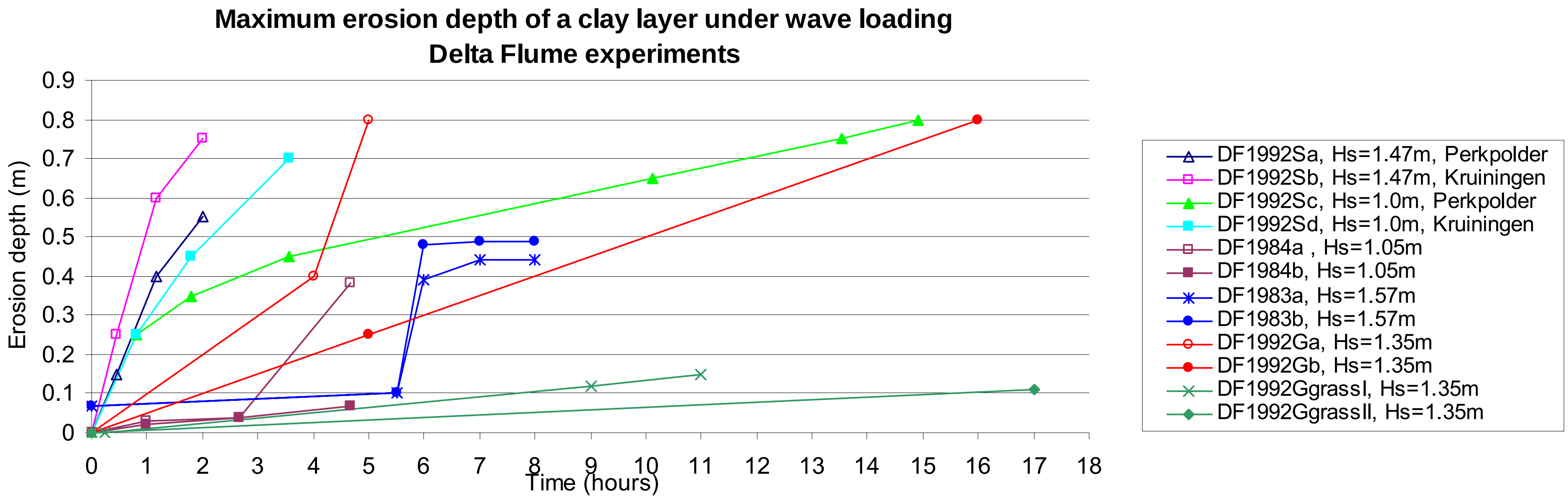


Figure 8.1 Overview of the results of the Delta Flume experiments on the strength of clay under wave loading, including two experiments on grass slopes.

8.3 Erosion development

In this paragraph the erosion development of the Delta Flume experiments on clay are compared. In 8.3.1 this comparison is based on the different clay types, which is the sand percentage in the clay. In 8.3.2 a comparison of the experiments is based on the clay condition. Within the subdivision of the clay conditions, the influence of the clay type is examined in 8.3.3 and the influence of the wave height in 8.3.4.

8.3.1 Comparison of the clay type

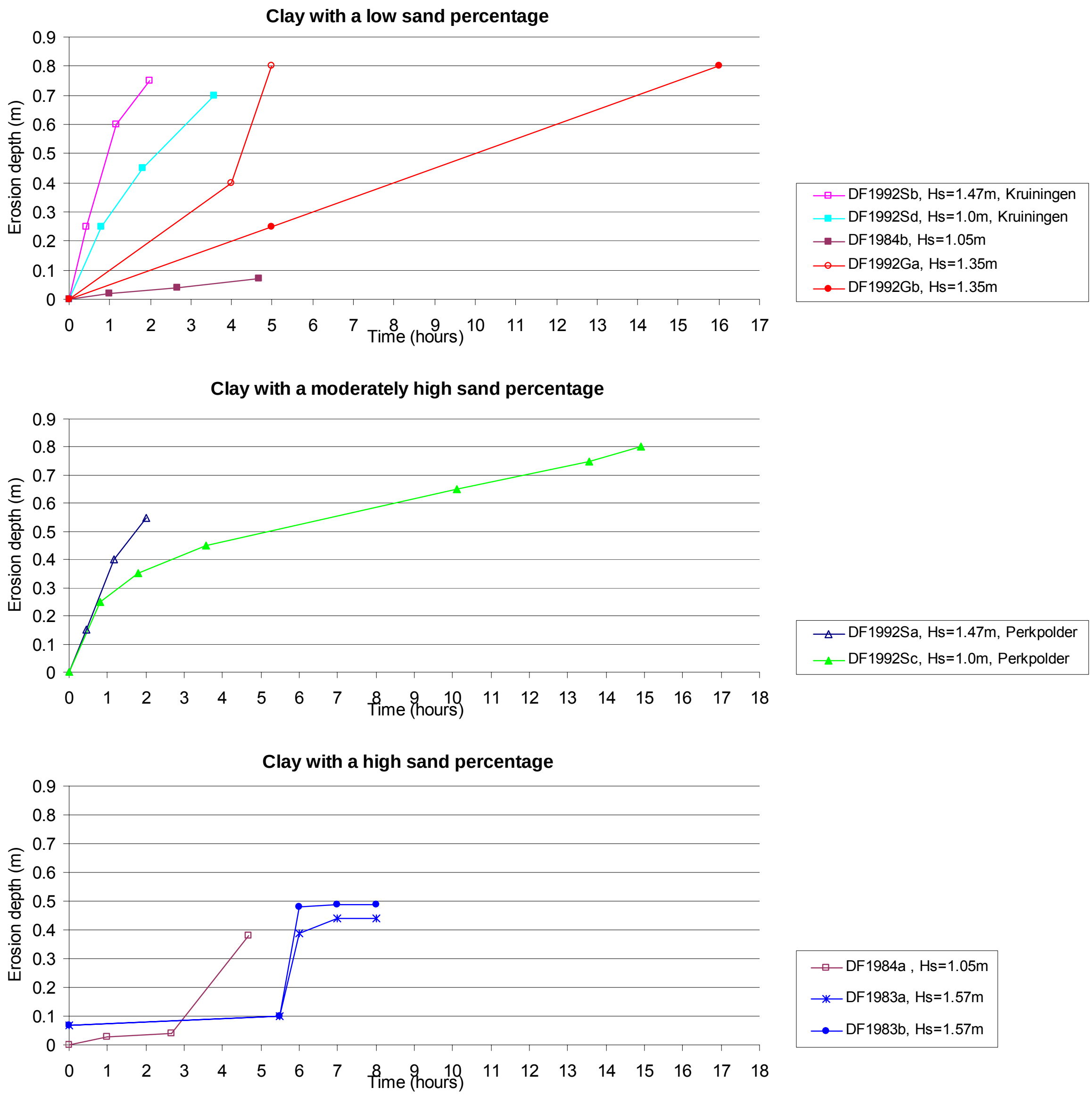


Figure 8.2 Overview of the experiments on clay. The comparison is based on the difference in sand percentage in the clay, the clay type.

An overview of the different experiments subdivided by the differences in the clay type, which is the sand percentage of the clay, is given in Figure 8.2.

Results
From this comparison it appears at a glance that a good subdivision based on the clay type is not possible, because the lines in the graphs are very scattered, especially for the clay with a low sand percentage. Clay with a low sand percentage is expected to have a high resistance against erosion,

but this can not be observed from the results of the experiments. The erosion duration of the clays with a high sand percentage is larger than that of clay with a low sand percentage.

When the wave conditions used in the tests on the clays with a moderately high sand percentage are observed more closely, it appears that a lower wave load results in a higher erosion resistance. This is what is to be expected. For the clay with a low sand percentage there is no relationship observed between the wave height and the resistance against wave loading. For the clay with a high sand percentage a lower wave height gives a lower strength. However, the Iribarren parameter of DF1983 is low (ξ=0.7), which results in a low wave impact despite the high wave height. The Iribarren parameter of DF1984 is high (ξ=4.2). The influence of the wave height is further elaborated in 8.3.4.

The erosion of the clay in experiment DF1983 was developed with a high erosion rate in clay with a high sand percentage (between 6-8h), but the erosion almost stopped when it reached the clay layer with a low sand percentage. In this experiment the clay type was apparently of large relevance to the erosion development, and thus to resistance of the soil against erosion. However, this clay is compacted after application and is therefore unstructured. The erosion rate of this clay outside the period of 6-8h is significantly lower than the erosion rates of the clays with a low sand percentage in the upper graph. In these clays a soil structure was developed, except for the clay of DF1984, and therefore it can be concluded that the clay condition will probably have an influence on the resistance to erosion. In 8.3.2 a comparison of the experiments is based on the clay condition.

Conclusion
A subdivision of clay based on the clay type only can not be made and therefore it can be concluded that the clay type is not the major influencing factor on the erosion rate of clay under wave loading.

8.3.2 Comparison of the clay condition

An overview of the different experiments subdivided by the extent of soil structure development is given in Figure 8.3. In 8.4 the experiments on the grass sods are included in the comparison.

Results
At first sight it is clear that structured clay has a lower strength than unstructured clay; the erosion rate of structured clay is higher. Moderately structured clay forms the transitional stage. Moderately structured clay is clearly stronger than structured clay, except test DF1992Sa (H_s =1.47m). This test results are similar to the structured clay.

The experiments on unstructured clay were stopped at an erosion depth of approximately 0.4m. The other tests were mostly continued to an erosion depth of 0.8m. If the lines of the unstructured clays in the graph are linearly extended, only test DF1984a results a deep erosion hole, the other clays give a high strength.

The erosion of the clay in experiment DF1983 developed with a sudden high velocity between 5.5h and 6h, in the clay layer with a high sand percentage. The erosion almost stopped when it reached the clay layer with a low sand percentage. This clay was compacted during application, so therefore it can be concluded that not just the clay condition is relevant but also the clay type. The top layer of the clay will probably have more soil structure development due to the high sand percentage of the clay and the vegetation development. This is further elaborated in 8.3.3.

Experiments DF1984 (H_s=1.05m, T=12s) contained surging breakers, so no real breaking occurred on the slope. DF1983 (H_s=1.57m, T_p=5.26s) were plunging breakers, but due to the gentle slope the waves were hitting the water layer of the previous waves and not directly on the slope. This was reducing the load on the slope. The wave loading of DF1984 was thus lower than of DF1983, but because the breaker types were different, also the erosion mechanisms could be different and a clear comparison is not possible to make. The influence of the wave height is treated in 8.3.4.

Conclusions
- From this comparison it can be concluded that the influence of the clay condition on the erosion resistance of the clay under wave loading is significant.
- A subdivision in three clay conditions gives a reasonably good result.
- Structured clay has a higher erosion rate than unstructured clay. Moderately structured clay forms a transition condition.

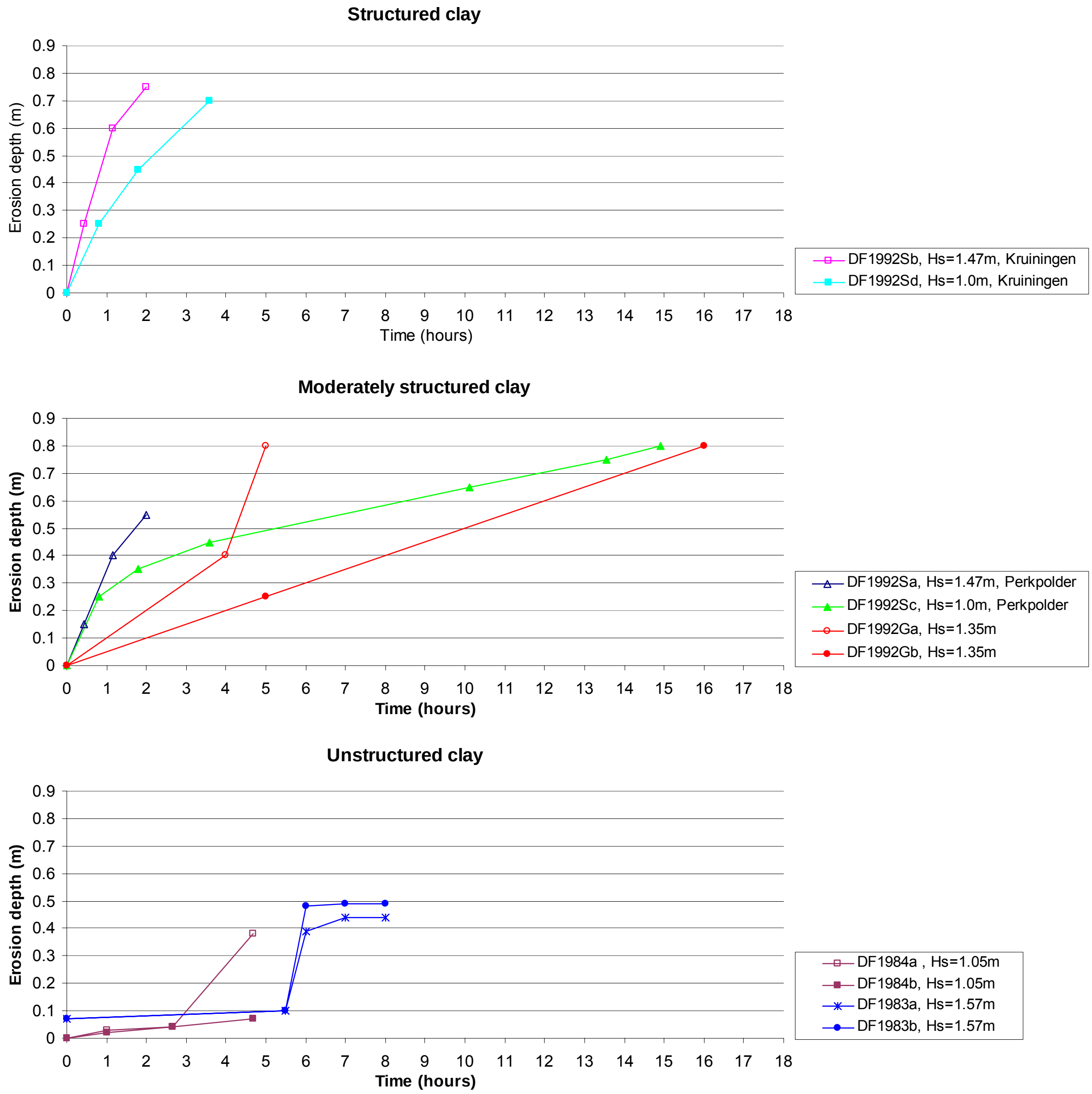


Figure 8.3 Overview of experiments on clay, based on the difference in clay condition. A subdivision is made between structured clay, moderately structured clay and unstructured clay.

8.3.3 Comparison of the clay condition and the clay type

It was expected that clay with a lower sand percentage would have a higher resistance against erosion by waves, and would have a lower erosion rate than clay with a high sand percentage. On the basis of the two comparisons of the clay type and the clay condition given in the former two paragraphs, it can however be concluded that the clay condition has more influence on the erosion rate than the clay type. In this paragraph the influence of the clay type on the erosion rate is investigated once again, now based on the differences in clay condition. This is given in Figure 8.4; within the subdivision per clay condition, the clay type is represented by three different colours for the three clay types: clay with a low sand percentage (in red), a moderately high sand percentage (in pink) and a high sand percentage (in yellow).

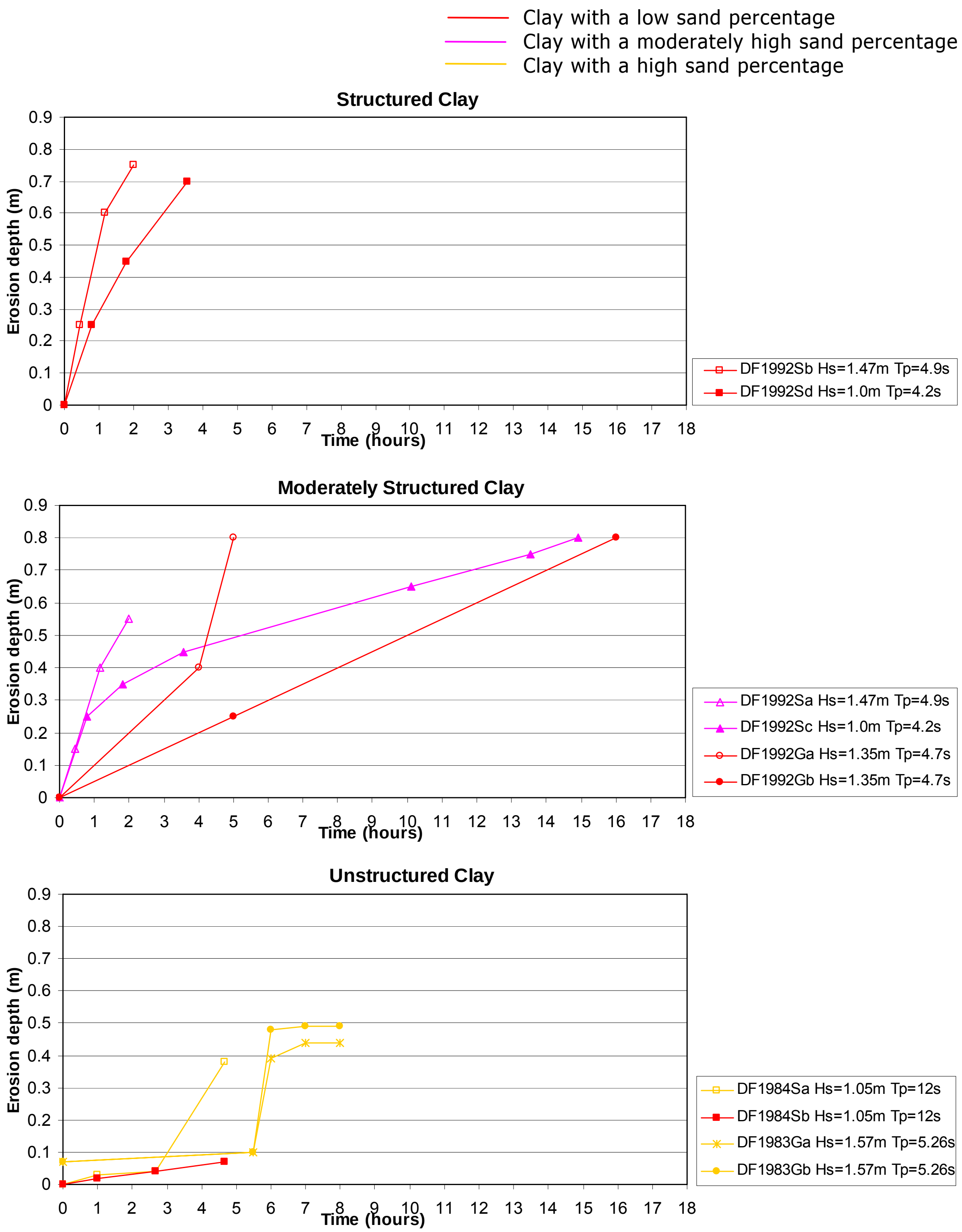


Figure 8.4 Overview of the experiments on clay. A comparison of the erosion development based on the differences in clay condition. Within the subdivision per clay condition, the clay type is represented by three different colours for the three clay types: clay with a low sand percentage (in red), a moderately high sand percentage (in pink) and a high sand percentage (in yellow).

Results
Structured clay
The two tests with structured clay are performed on the same clay and therefore no conclusions can be drawn.

Moderately structured clay
- Clay with a moderately high sand percentage had initially a higher erosion rate than clay with a low sand percentage, independent on the wave height. This is what would be expected.
- At a depth of more than 0.4m, the erosion rate of clay with a moderately high sand percentage decreased. The erosion rate of DF1992Sc became from a depth of 0.4m even lower than of the clay with a low sand percentage. This could however be due to the influence of the lower wave load of DF1992Sc.

Unstructured clay
- The final erosion of clay with a low sand percentage seems at first sight lower than of clay with a high sand percentage. However, if looked more closely, the erosion rates of DF1984a and b (high and low sand percentage) are initially the same, after some time a lump of was eroding of DF1984a, which resulted in the sudden higher erosion rate.
- The erosion rates of DF1984a and DF1984b are the first 2.8h similar, while the clay types are clay with a high sand percentage and clay with a low sand percentage. After 2.8h, the erosion rate of the clay with a high sand percentage was increasing significant, due to the sudden erosion of a lump of clay.
- Both clays with a high sand percentages show a sudden high erosion rate. The erosion rate of DF1983 became almost zero at a depth of 0.44-0.5m.
- The erosion of DF1983 took place in the top layer which consisted of a grass sod. A hole of 0.07m was made on beforehand in de grass. The erosion was quite suddenly developed into this layer, but was stopped when reached the lower clay layer without roots. In this layer, the sand percentage is low and hardly any erosion occurred.
- The erosion rate of DF1983, with a high sand percentage, is at the start and from a depth of 0.44-0.5m lower than of the clay with a low sand percentage. This could however at the start be due to the presence of roots in the clay which can strengthen the clay layer. The deeper lying clay layer consisted of clay with a low sand percentage.

Few examples of the erosion development over time:
- The developed maximum erosion depth of structured clay with a low sand percentage is after three hours approximately a factor of 12 (0.6m/0.05m) larger than of unstructured clay with a low sand percentage. Both clays have a wave height of 1.0m, but the wave period of the unstructured clay is larger.
- The developed maximum erosion depth of structured clay with a low sand percentage is after three hours approximately a factor of 1.5 (0.6m/0.4m) larger than of moderately structured clay with a moderately high sand percentage. Both clays have a wave height of 1.0m and a wave height of 4.2s.

Horizontal erosion
The horizontal measured erosion is analysed in Chapter 9 of a comparison to the erosion of sand.

Discussion
- The decrease in erosion rate from a depth of approximately 0.4m of moderately structured clay could be due to the compaction of the clay. Even though the clay condition is stated to be moderately structured, it is possible that the clay deeper in the layer is less structured than the upper part and thus results a lower erosion rate.
- The sudden high erosion rate of DF1983 could be a result of the high sand percentage in the clay.
- The low erosion rate at the beginning of experiment DF1983 could be due to the presence of roots in the clay which can strengthen the clay layer. The deeper lying clay layer consisted of clay with a low sand percentage, which can explain the low erosion rate.

Conclusions
- Within the subdivision of the clay condition, the clay type is an influencing factor on the erosion development of clay under wave loading.
- Based on this data, for structured clay no conclusions can be drawn.
- For moderately structured clay, clay with a low sand percentage gives a lower erosion rate than clay with a moderately high sand percentage.

- For unstructured clay can be concluded that a high sand percentage of the clay could result in a sudden higher erosion rate due to the erosion of lumps of clay.
- Unstructured clay with a low sand percentage has a high resistance against erosion. Especially in comparison to structured clay.

8.3.4 Comparison of the wave height

From a comparison of the different wave heights only no conclusions can be drawn. It is to be expected that a higher wave height will results in a higher erosion rate, but this can not be concluded based on the wave height only. (See also Appendix XIII.)

If a comparison is made again, but based on the different clay conditions, this hypothesis stands out more clearly as can be observed in Figure 8.4.

Results

Structured clay

- A higher wave height resulted in a higher erosion rate. The influence of the clay type can not be proved.

Moderately structured clay

- Till a depth of 0.4m the influence of the wave height is not clearly visible in the results. For the clay with a moderately high sand percentage, the test with a higher wave height has a higher erosion rate, as is expected. The clay with a low sand percentage has a wave load of H_s=1.35m which is approximately the average of the wave heights of the two experiments of the moderately high sand percentage and therefore it would be expected that it has a lower erosion rate than DF1992a (H_s=1.5m) and a higher erosion rate than DF1992Sc (H_s=1.0m). This is not the case, both erosion rates are lower than of the clay with a moderately high sand percentage. Apparently the clay type is a more important factor for the rate of erosion.
- From a depth of 0.4m, the erosion rate depends more clearly on the wave height. The lowest wave height (H_s=1.35m) gives the lowest erosion rate (DF1992Sc). The erosion rate of the highest wave load (H_s=1.5m), DF1992Sa, is however a little lower than of DF1992Ga with middle wave height. The other middle wave load has an erosion rate which lied between the highest and lowest as is expected.
- For the experiments of DF1992S the hypothesis that a higher wave height will result in a higher erosion rate is valid.

Unstructured clay

- The influence of the wave height on the erosion rate is not clearly to recognize due to the different wave characteristics and thus different breaker types. Experiments DF1984 (H_s=1.05m, T=12s, ξ=4.2) contained surging breakers, so no real breaking occurred on the slope. DF1983 (H_s=1.57m, T_p=5.26s, ξ=0.7) were plunging beakers but due to the gentle slope, the waves were hitting the water layer of the previous waves and not directly on the slope. This was reducing the load on the slope. The wave loading of DF1984 was thus lower than of DF1983, but because the breaker types were different, also the erosion mechanisms could be different and a clear comparison is not possible to make. A higher wave height did not result into a height erosion rate. The erosion rate of DF1983 was at the start and from a depth of 0.44-0.5m lower than of DF1984, which has a lower wave height. This is not what is expected. Only for the steep parts of the sudden high erosion rate, the erosion rate of the experiments with the highest wave height resulted into a higher erosion rate.

Discussion

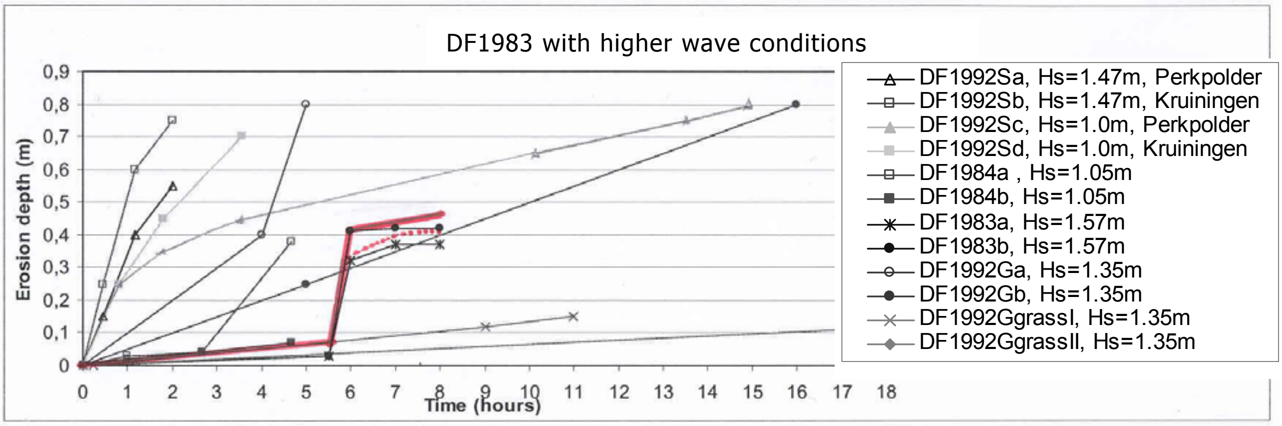


Figure 8.5 DF1983 with higher wave conditions and therefore a higher erosion rate.

If experiment DF1983 would be performed with higher wave conditions then a different development of the erosion over time would occur. If a higher Iribarren parameter would be obtained, like for example ξ=1.3 as is similar to most of the performed experiments, the wave load on the slope will be higher. Therefore the erosion rate could be higher as well. This makes the gradient of the graph steeper. In Figure 8.5 an example is given. In this case, the strength will be not as high as it seems at first sight in comparison with the other experiments.

If the line in the graph of Figure 8.5 is steeper, the erosion rate of DF1983 is approximately similar to DF1992GrassI, the erosion rate of the first experiment on the grass slope.

From this discussion can be concluded that for a higher wave load, not only the waver height and wave period are of influence, but also the slope angle. The Iribarren parameter will be of influence of the erosion development.

Conclusions

- Within the subdivision of the clay condition, the wave condition is an influencing factor on the erosion development of clay under wave loading.
- For structured clay a higher wave height results in a higher erosion rate.
- For moderately structured clay is observed that the sand percentage in the clay is initially of more influence on the erosion rate than the wave height. From a depth of 0.4m the wave height becomes of more influence than the clay type.
- Besides the wave condition also the slope angle could be of influence on the erosion development.

8.4 Comparison with the erosion development of a grass cover

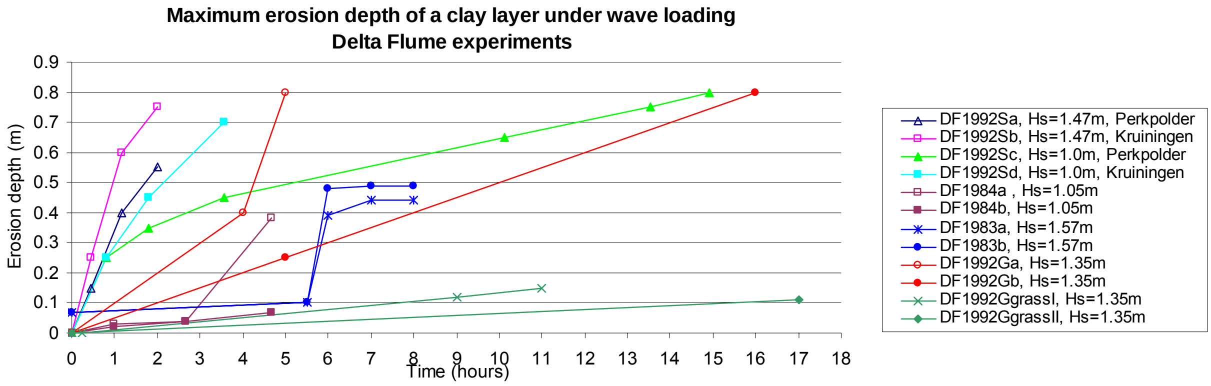


Figure 8.6 Overview of the comparison of experiments on clay and experiments on grass under wave loading

To gain insight into the behaviour of clay under waves compared to that of grass, the experiments on the grass cover (DF1992GrassI and DF1992GrassII) are included in the comparison of the clay experiments. The results of the experiments on the grass cover are compared the erosion development of the experiments on clay. (See Figure 8.6) Because the roots of the grass bind the clay aggregates and delay therefore the erosion process, the grass cover is considered to behave similar to unstructured clay. The actual condition of the clay between the roots of the grass will be structured, see chapter 6.

Results

- The erosion rate of grass is lower than of the clay experiments.
- The erosion rate of grass in these experiments is on average approximately $3*10^{-6}$ m/s (0.01m/h).
- The erosion rate of DF1992GrassI is approximately similar to DF1984b, which contained unstructured clay with a low sand percentage.
- The erosion development of grass is generally more constant than of clay and has a linear course over time.

Discussion

- The available data of the erosion of grass covers under waves is very limited, even more limited than the available data of clay. Therefore the conclusions are disputable and more experiments on the influence of waves on grass sods should be performed to verify these conclusions.
- It is unknown if the grass sod contained bare spots or other weak locations, which appear to be of major influence on the erosion resistance of grass. Due to the low erosion rate, it could be assumed that the grass layer, which was tested in the experiment, was intact.

Conclusions

- Grass has a higher erosion resistance against wave loading than clay.
- The erosion rate of unstructured clay with a low sand percentage is approximately similar to the erosion rate of grass.

8.5 Comparison of the empirical formula to the residual strength of clay for all the experiments

An empirical formula for the erosion depth over time for the residual strength of clay under stone revetments has been made by WL|Delft Hydraulics (2006b), see 10.5. This formula is based on the experiments to the residual strength of the clay layer of 1984 (DF1984) and 1992 (DF1992). In this paragraph this formula is compared to the results of the other large scale experiments.

8.5.1 The empirical formula fitted for all experiments

The empirical formula is repeated below, for more information is referred to 5.5 and [WL|Delft Hydraulics, 2006b]:

$$d_e = C_c * H_s\left(-6.9 + \ln\left(t\right)\right) \qquad (8\text{-}1)$$

d_e erosion depth (m)
C_c clay type factor (-), empirical parameter see Table 5.2 and Table 8.2
H_s wave height (m)
t time (s)
-6.9 empirical parameter which is fitted with good results to the measurements [WL|Delft Hydraulics, 2006b]

Based on the empirical parameter -6.9, the clay type parameter C_c is fitted for all experiments. This resulted to clay type parameters for each experiment, see Table 8.2. In the table a subdivision is made by the C_c given by Delft Hydraulics (2006b) and the values of C_c fitted for each experiment.

In this way the formula is obtained for each experiment, based on the parameter -6.9. This formula is plotted for each experiment in a graph including the tests results. A logarithmic trendline is plotted though the test results to compare this to the obtained empirical relation.

Table 8.2 Different values for C_c fitted to each experiment, compared to the values of C_c of WL|Delft Hydraulics (2006b)

Experiment	Clay type	Clay condition	C_c DH (2006) (-)	C_c (-)
DF1992Sa	Moderate sand percentage	Moderately structured	0.20	0.20
DF1992Sb	Low sand percentage	Structured	0.26	0.28
DF1992Sc	Moderate sand percentage	Moderately structured	0.20	0.20
DF1992Sd	Low sand percentage	Structured	0.26	0.25
DF1984a	High sand percentage	Unstructured	0.018-0.032 (excl. hole)*	0.018 (excl. hole)*
			0.018-0.13 (incl. hole)*	0.1 (incl. hole)*
DF1984b	Low sand percentage	Unstructured	0.014-0.025	0.02
DF1983a	In grass layer: High sand percentage In sublayer: Low sand percentage	Unstructured	-	0.08
DF1983b	In grass layer: High sand percentage In sublayer: Low sand percentage	Unstructured	-	0.08
DF1992Ga	Low sand percentage	Moderately structured	-	0.17
DF1992Gb	Low sand percentage	Moderately structured	-	0.12

* The developed erosion hole in DF1984a was considered to be a result of a lack of compaction. Therefore it was stated not to be of relevance for unstructured clay, because it is not representative for the coherence which clay obtains after several years in a dike slope. The developed hole of 0.38m deep was not taken into account in obtaining the formula. If this hole would be included in the data set, the upper boundary for C_c of clay with a high sand percentage will be 0.13.

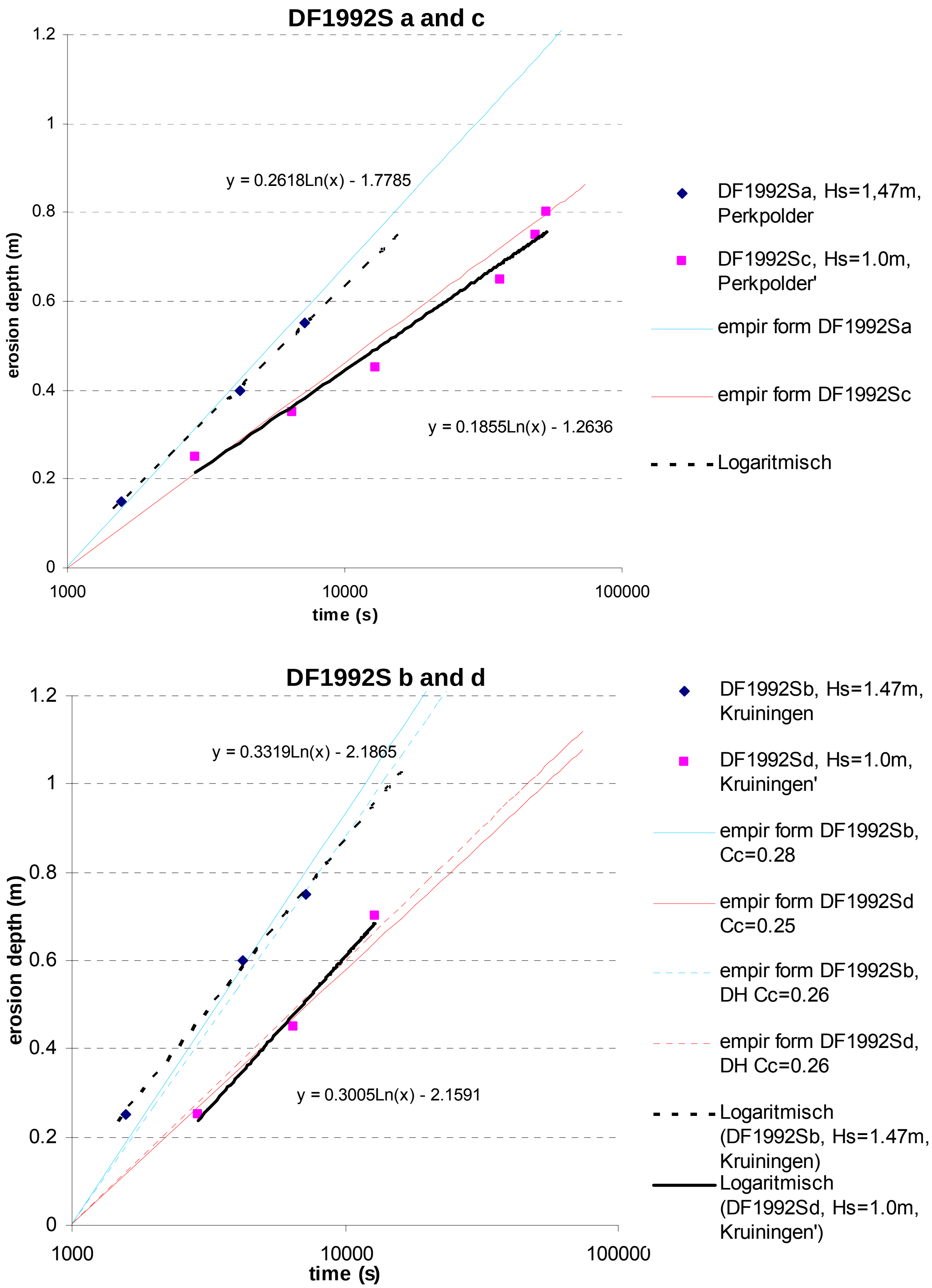
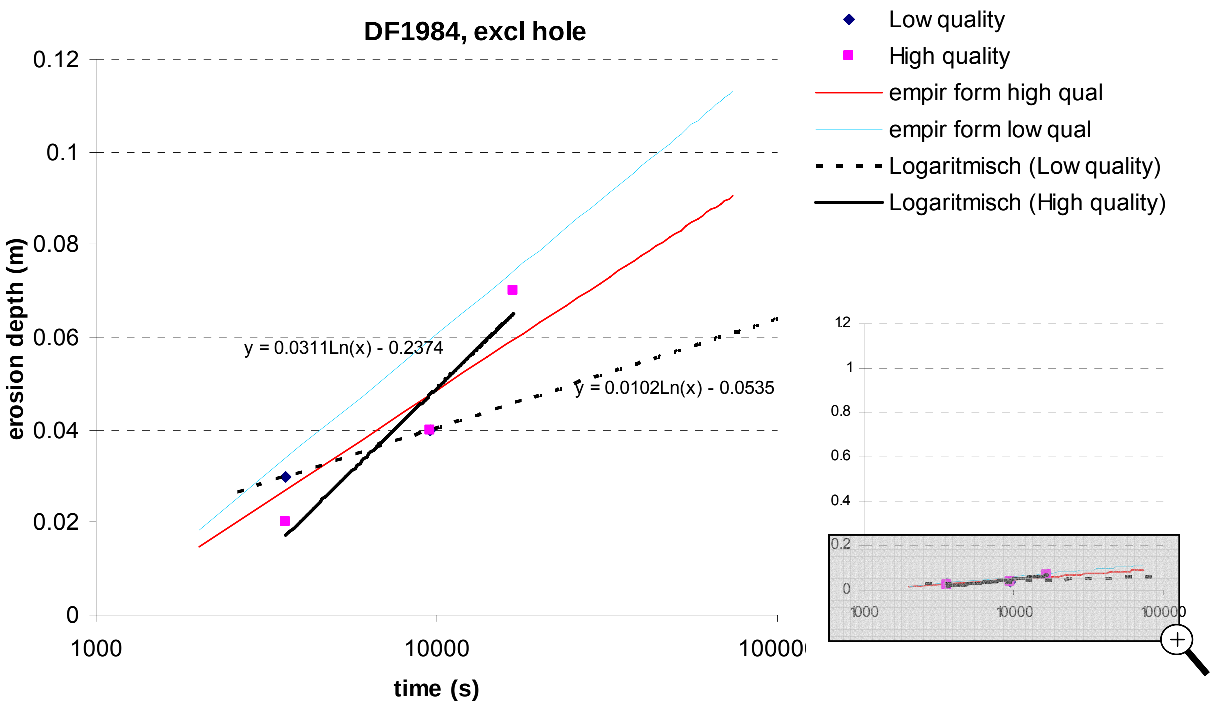


Figure 8.7 and Figure 8.8 The empirical formula fitted for the Delta Flume experiments of 1992 to the residual strength under stone revetments (DF1992S). The empirical relation (coloured line) with the fitted C_c value is plotted on a logarithmic time scale, together with the data points of the results of the experiments. The logarithmic trend lines of the data sets are added to the data sets, including the formula.
Above: DF1992Sa and DF1992Sc, moderately structured clay with a moderately high sand percentage. Below: DF1992Sb and DF1992Sd, structured clay with a low sand percentage.



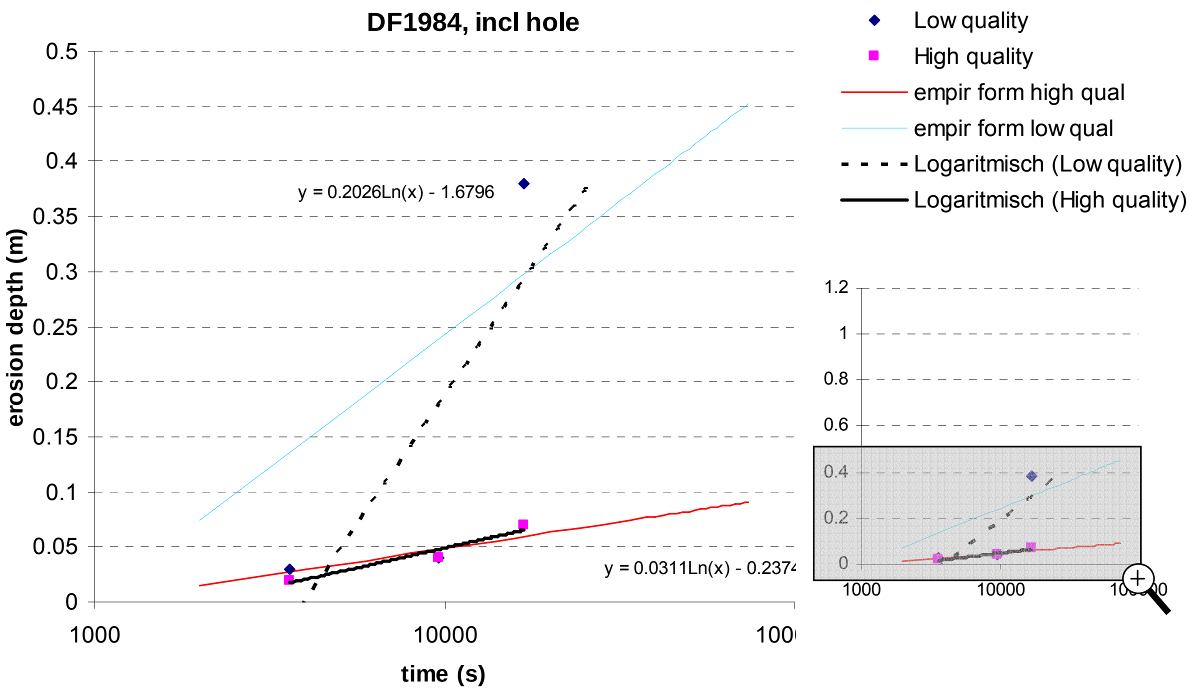


Figure 8.9 and Figure 8.10 The empirical formula fitted for the Delta Flume experiment of 1984 to the residual strength under stone revetments (DF1984). The empirical relation (coloured line) with the fitted C_c value is plotted on a logarithmic time scale, together with the data points of the results of the experiments. The logarithmic trend lines of the data sets are added to the data sets, including the formula.
Above: Excluding the data point of the developed hole, the approach of WL|Delft Hydraulics (2006b).
Below: Including the data point of the developed hole.

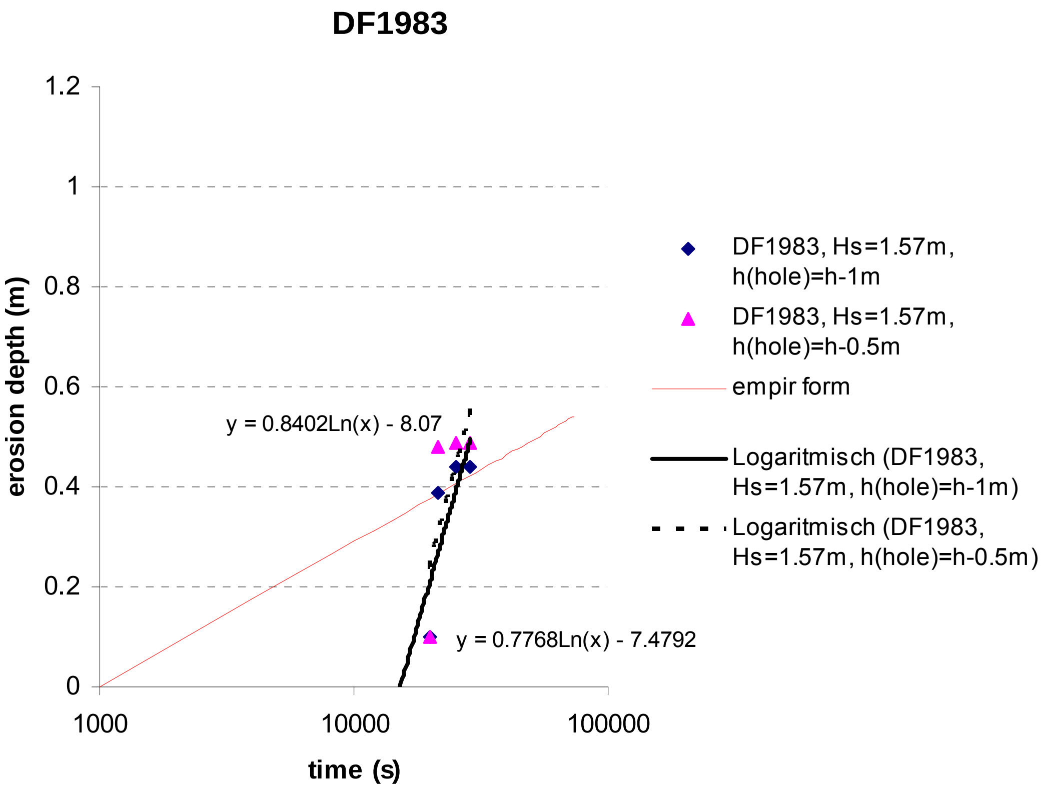


Figure 8.11 The empirical formula fitted for the Delta Flume experiments of 1983 to the residual strength under grass covers (DF1983). The empirical relation (coloured line) with the fitted C_c value is plotted on a logarithmic time scale, together with the data points of the results of the experiments. The logarithmic trend lines of the data sets are added to the data sets, including the formula.

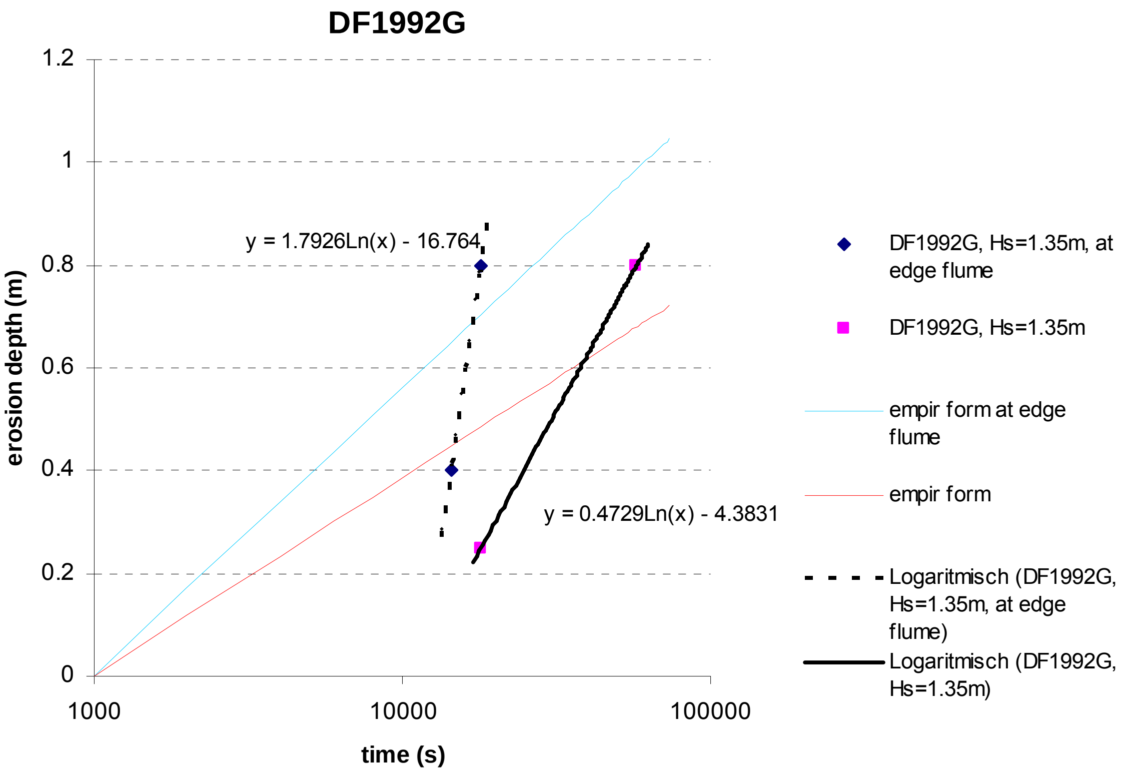


Figure 8.12 The empirical formula fitted for the Delta Flume experiments of 1992 to the residual strength under grass covers, DF1992G. The empirical relation (coloured line) with the fitted C_c value is plotted on a logarithmic time scale, together with the data points of the results of the experiments. The logarithmic trend lines of the data sets are added to the data sets, including the formula.

Results

- The obtained empirical formula is clearly fitted to the experiments of DF1992S. For these experiments the empirical formula is closest to the experiment results. This is however expected, because the empirical formula is based on these experiments.
- The obtained value of 0.26 for DF1992b and DF1992d (for Kruiningen clay) by WL|Delft Hydraulics can be obtained by the average values of the calculated values for C_c of the two tests.
- The empirical formula for DF1984 without the developed hole, like used by WL|Delft Hydraulics fits very well to the data set. If looked more closely to the data, there is however some difference with the logarithmic trendline of the data set.
- The empirical formula for DF1984 including the developed hole does not develop similarly to the trendline of the dataset. The difference is not very large, but the empirical formula gives a lower development of the erosion depth in time compared to the logarithmic trendline.
- The calculated empirical formulas of the experiments of DF1983 do not fit the data results. The empirical formula gives a slower erosion development over time.
- The calculated empirical formulas of the experiments of DF1992G do not fit the data results. Also here the empirical formula results in a lower erosion rate.
- At a normal time scale, not logarithmic, the results seem to be slightly better fitted.

8.5.2 Discussion and conclusions

The empirical formula fits well to the data it is based on, however it delivers no useful results when compared to the data of the other large scale experiments.

In the empirical formula as formulated by WL|Delft Hydraulics, the erosion depth is dependent on the wave height and the clay type. However, from the different large scale tests is concluded that the clay condition and the slope angle can also be influencing factors in the erosion process.

8.6 Location of erosion along the slope

For obtaining more insight into the erosion process of clay under wave loading, also the location of erosion is of importance. In this paragraph the location of erosion along the slope is investigated for each experiment and compared to each other.

By Smith (1994a) three erosion zones along the slope were defined, based on the results of the experiment in the Delta Flume in 1992 on grass slopes (DF1992G). These zones all have different erosion intensities. In this paragraph this theory is compared to the results of the location of erosion development along the slope of all other large scale experiments. At first the theory of the erosion zones is repeated, after which the location of erosion per experiment are defined and compared to the theory. Conclusions are provided at the end of this section.

8.6.1 Erosion zones

8.6.1.1 Repetition of the theory of the erosion zones

The theory of the three different erosion zones is given in chapter 5, and in short repeated below. Three erosion zones along the slope were defined, based on the results of the experiment in the Delta Flume in 1992 on grass slopes (DF1992G). These zones all have different erosion intensities.

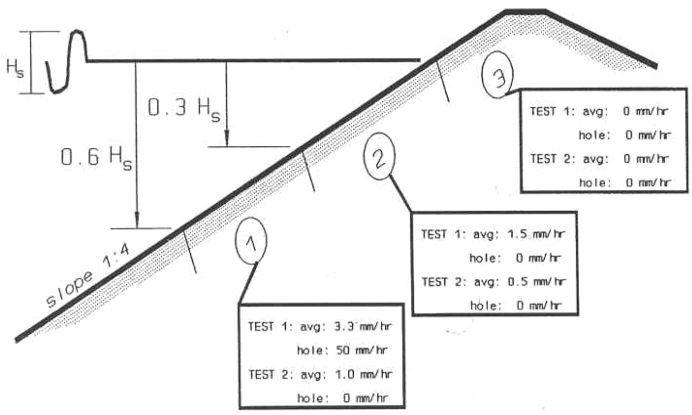


Figure 8.13 Erosion zones along the slope. [Smith, 1994a]

Zone 1 At a depth between $0.3H_s$ and $0.6H_s$ below SWL
The majority of erosion took place in this zone, a hole developed and the highest average erosion rate occurred.
Zone 2 Between still water level and a depth of $0.3H_s$
The average erosion rate was half of that in zone 1.
No hole developed.
Zone 3 Above still water level
Practically no erosion observed.
[Smith, 1994a]

8.6.1.2 Erosion zones per experiment

The boundaries of the erosion zones are dependent on the wave height and the water level and are defined by $0.3H_s$ and $0.6H_s$. The boundaries for each experiment are given in Table 8.3. The location of the boundaries is formed by the height of the water level minus the boundary value of the table.

Table 8.3 The boundaries of the erosion zones per experiment

Erosion zone	Stone			Grass	
	DF1984	DF1992Sa,b	DF1992Sc,d	DF1983	DF1992G
H_s (m)	1.05	1.47	1.0	1.57	1.35
$0.3H_s$ (m)	0.3	0.4	0.3	0.5	0.4
$0.6H_s$ (m)	0.6	0.9	0.6	0.9	0.8

8.6.2 Location of erosion for each experiment

For the determination of the location of erosion the same method is used as Smith (1994a) has done, by making the location of erosion dependent of the wave height.

DF1984

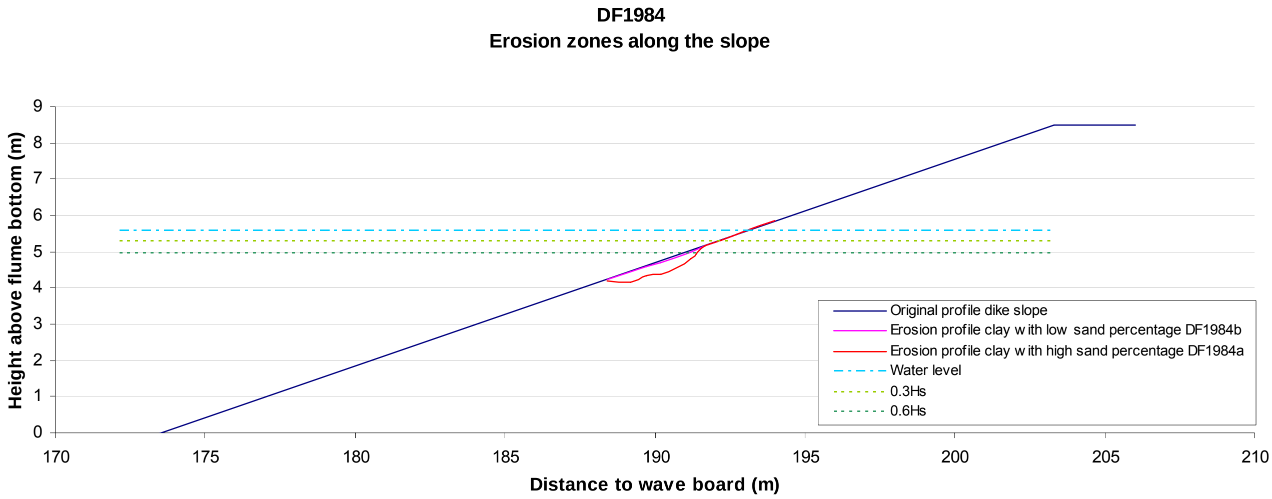


Figure 8.14 Erosion profiles of DF1983 including the theoretical erosion zones given by green lines.

The erosion of clay with a low sand percentage was taking place at a level between $0.4H_s$ and $1.2H_s$ below the water level. (0.4-1.2m below the water level)
The erosion of clay with a high sand percentage was taking place at a level between $0.4H_s$ and $1.3H_s$ below the water level. (0.4-1.3m below the water level)

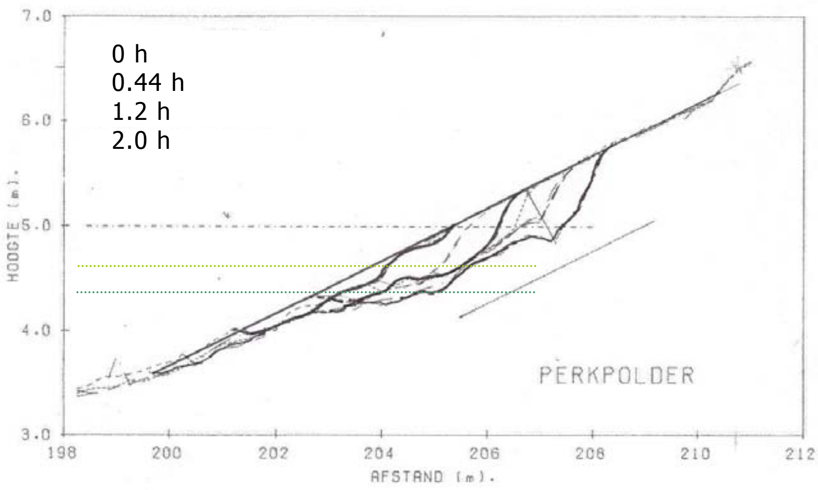
For DF1984 the greatest erosion took place below $0.6H_s$, as can be seen in Figure 8.14 where the erosion zones are given by the dotted light and dark green lines. According to the theory of the erosion zones the greatest erosion takes place between $0.3H_s$ and $0.6H_s$ below SWL. This theory is thus not valid for this experiment.

However the wave characteristics were of the surging breaker type, which causes a different kind of load on the slope than plunging breakers. This could explain the different location of erosion than the theory. Another remark is that the developed erosion hole was at maximum 0.4m deep, thus maybe zone 1 would be eroded if the experiment would have been continued. This could have been a development like the low water experiment of DF1992S.

DF1992S
High water level
For DF1992Sa (moderately structured clay with moderately high sand percentage) the erosion starts at approximately $0.3H_s$ below the water level, and continues to $0.6H_s$ below water level and $0.3H_s$ above water level (SWL-$0.6H_s$ till SWL+$0.3H_s$). (0.9m below – 0.4m above the water level)
For DF1992Sb (structured clay with low sand percentage) the erosion starts between $0.3H_s$ and $0.6H_s$ below the water level, and continues to $0.5H_s$ below water level and $0.3H_s$ above water level (SWL-$0.5H_s$ till SWL+$0.3H_s$). (0.8m below – 0.4m above the water level)
See Figure 8.15 and Appendix VIII.

The erosion of both clays started thus approximately in the heaviest erosion zone, zone 1, but developed till above the water line, in zone 3. At this point the erosion in zones 2 and 3 was the greatest.

High water level



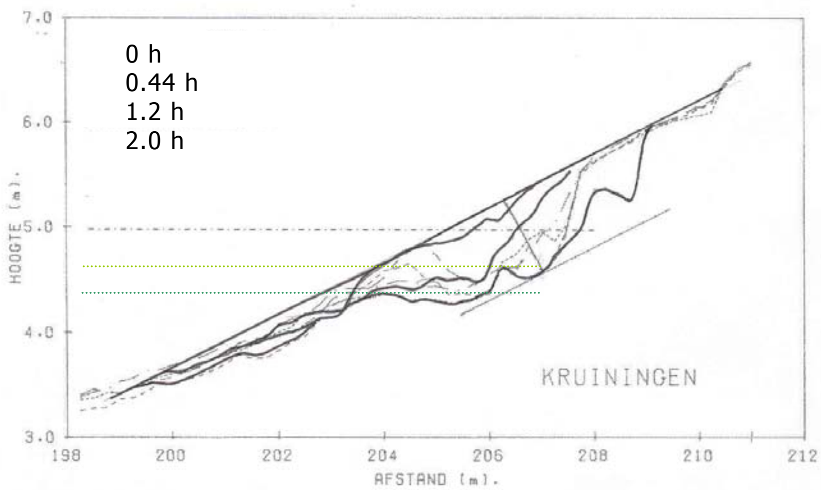


Figure 8.15 Erosion profiles of the high water experiment of DF1992Sa and DF1992Sb. The theoretical erosion zones are given by the green lines.

Low water level
For DF1992Sc (moderately structured clay with moderately high sand percentage) the erosion starts between $0.4H_s$ and $0.9H_s$ below the water level, and continues till above the water level. (0.9m below – 0.4m above the water level)
For DF1992Sd (structured clay with low sand percentage) the erosion starts between $0.4H_s$ and $1.0H_s$ below the water level, and continues as well till above the water level. (1.0m below – 0.5m above the water level)

For both clays an erosion hole started to develop below, zone 1, thus below the assumed zone of greatest erosion.

The hole in DF1992Sc was enlarged at this location below zone 1 and due to this development zone 1 and zone 2 were eroded as well. The erosion was limited by a sheet pile and therefore the erosion was limited to zone 2.

After 2.0h the initial developed hole in the DF1992Sd was not enlarging, but another erosion hole developed in zone 1 above the initial hole and developed also in zone 2. The highest erosion took place in zone 1.

See Figure 8.16 and Appendix VIII.

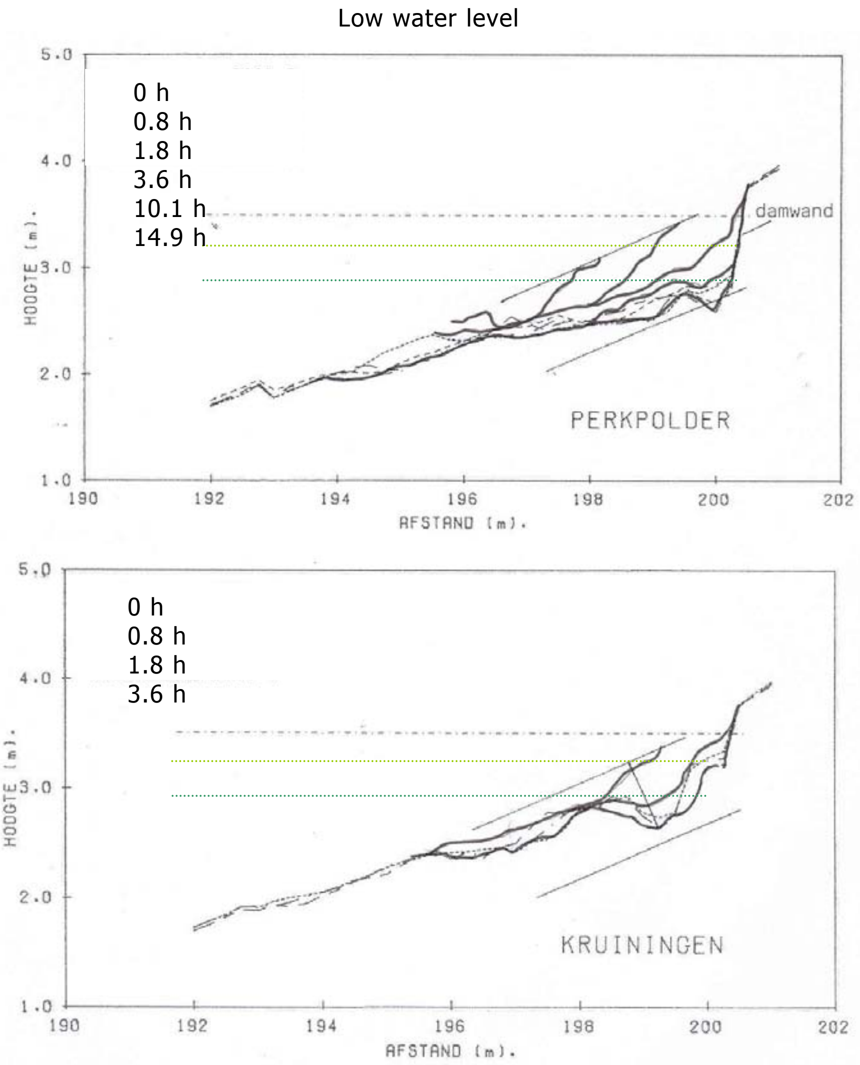


Figure 8.16 Erosion profiles of the low water experiment of DF1992Sc and DF1992Sd. The theoretical erosion zones are given by the green lines.

DF1983

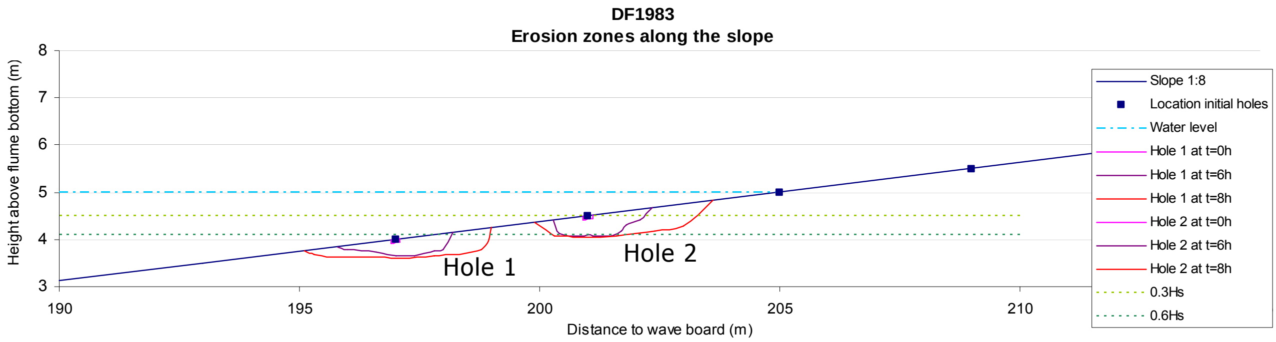


Figure 8.17 Erosion profiles of DF1983 including the theoretical erosion zones given by the green lines.

Four holes of 0.50m*0.20m and 0.07m deep were made beforehand in the clay of the slope at 1.0m and 0.5m below the water level. The location of initial hole 1 was just below zone 1 and the erosion of this hole also developed just below zone 1 and for a small part in zone 1. Hole 2 was located at the level of $0.3H_s$ and most of the erosion developed in zone 1 and for a small part in zone 2. The development of hole 2 was influenced by the side of the flume. See also Paragraph 9.2.1.

The erosion of both holes developed around the initial made hole.
For DF1983a (hole 1), the erosion developed from $0.5H_s$ till $0.9H_s$ below water level. (0.8-1.4m below water level)
For DF1983b (hole2), the erosion developed from $0.1H_s$ till $0.6H_s$ below water level. (0.2-0.9m below water level)
There was no significant difference in erosion development of the two holes. Therefore it can not be concluded from these results that the highest erosion will take place in the zone between $0.3H_s$ and $0.6H_s$, zone 1. The results could however prove that the existence of a weak location in the slope is responsible for the start of erosion and that this is a more important criterion than the location of the highest wave load on the slope.

8.6.3 Comparison

An overview of the location of erosion for each experiment with the experiment properties are given in Table 8.4. Only the wave characteristics and the slope angle are given as properties. The clay properties are assumed change only in depth and be approximately similar along the slope.

Table 8.4 Erosion locations for each experiment including experiment properties
-below water level, +above water level

Location (m)	Zone	DF1984a and DF1984b	DF1992S High water level		DF1992S Low water level		DF1983		DF1992Ga and DF1992Gb
			Perk Polder DF1992Sa	Kruiningen DF1992Sb	Perk Polder DF1992Sc	Kruiningen DF1992Sd	Hole 1 DF1983a	Hole 2 DF1983b	
Initial		-1.3 – -0.4	-0.4	-0.9 – -0.4	-0.9 – -0.4	-1.0 – -0.4	-1.0	-0.5	-0.8 – -0.4
Final		-1.3 – -0.4	-0.9 – +0.4	-0.8 – +0.4	-0.9 – +0.14	-1.0 – +0.5	-1.4 – -0.8	-0.9 – -0.2	-0.8 – 0
Initial $*H_s$		-1.3 – -0.4	-0.3	-0.6 – -0.3	-0.9 – -0.4	-1.0 – -0.4	-0.5	-0.9	-0.6 – -0.3
Final $*H_s$		-1.3 – -0.4	-0.6 – +0.3	-0.6 – +0.3	-0.9 – +0.14	-1.0 – +0.5	-0.9 – -0.5	-0.6 – -0.1	-0.6 – 0
>SWL	3		*	*					*
SWL-$0.3H_s$	2		**	**	*	*		*	*
$0.3H_s$ - $0.6H_s$	1		*	*	*	**		**	**
<$0.6H_s$		**			**	*	**		
Properties									
Water level(m)		5.6	5.0		3.5		5.0		4.8
Wave height(m)		1.05	1.47		1.0		1.57		1.35
Wave period(s)		12	4.9		4.2		5.3		4.7
Slope		1:3.5	1:4		1:4		1:8		1:4

* Location of erosion
** Location of highest amount of erosion

- For DF1992S high water level and DF1992G which have almost the same wave characteristics and slope angle, the erosion was taking place at approximately the same locations. Only the largest erosion was taking place in different locations.
- DF1992S low water level and DF1983b (hole 2) had the same location of erosion, but the wave period and slope angle are very different. The location of hole 2 was determined on beforehand.
- DF1984 and DF1983a (hole 1) have also approximately the same locations of erosion. Hole 1 was however made before the start of the experiment and the slope is more gentle. Also the wave period of DF1983.

Maybe DF1983 should be excluded from the comparison, because the holes were made before the start of the experiment, which has thus influenced the locations of erosion development. It is also the only experiment with a gentle slope.

8.6.4 Discussion and conclusions

The erosion of the experiments on clay was starting in all cases below the water level. The start is at a level of $1.3H_s$ and $0.3H_s$ below water level. For all experiments it can be seen that after the initial development of a hole, the hole tends to extend and develop further in an upward direction.

By Smith (1994a) was on the basis of results of the Delta Flume experiment on grass (DF1992G) the theory of three erosion zones along the slope was determined. According to this theory the highest erosion takes place between $0.3H_s$ and $0.6H_s$ below the water level (zone 1), after that between SWL and $0.3H_s$ (zone 2) and the lowest erosion takes place above the water level (zone 3). This theory is thus dependent on the water level and the wave height, not on other parameters.

However, other wave characteristics also could be of importance, because the theory is not in accordance with the results of the experiments. It is assumed that the clay properties of the experiments are the same along the slope and only change in depth and that they are therefore not of the influence on the location of erosion. In real situations the soil properties are of course of great importance as erosion will start at a weak location in the slope. This comparison gives thus an overview of other influence of other properties than the soil on the location of erosion.

For DF1984 the heaviest erosion took place below $0.3H_s$, thus below zone 1 of the highest erosion. However, the developed erosion hole was maximum 0.4m deep and thus maybe zone 1 would have been eroded if the experiment had a longer duration. Also Hole 1 of DF1983 developed below zone 1 and only developed a little into zone 1 as it deepened.

For the experiment with the low water level of DF1992S the erosion holes developed below zone 1. The hole in the Perkpolder clay developed into zone 1 and zone 2 as the hole deepened. In Kruiningen clay a second hole developed in zone 1, which extended and deepened over time into zone 2 and a little in zone 3.

Only for the experiment with the high water level of DF1992S and Hole 2 of DF1983, did the erosion developed initially in zone 1 and develop further into zone 2, and also zone 3 for DF1992S, as the holes were deepening. The greatest erosion of DF1992S however did not take place in zone 1 as would be according to the theory, but in zone 2 and zone 3. For DF1983 the greatest erosion took place in zone 1, however the hole was made beforehand of the experiment which thus influences the outcome.

On basis of these results it cannot be proved that wave characteristics are of influence on the location of erosion. These data are too limited for drawing conclusions on the subject. A different kind of load on the slope due to a different breaker type could be of influence on the location of erosion. This supposition is made on the basis of the erosion locations of DF1984 and DF1983 hole 1, which had erosion below the level of $0.6H_s$-SWL. However DF1983 should possibly be excluded from the comparison, because the holes were made before the start of the experiment, which has thus influenced the location of erosion development. On the other hand, DF1983 could prove that the existence of a weak location in the slope is responsible for the start of erosion and that this is a more important criterion than the location of the highest wave load on the slope. The results could however prove that the existence of a weak location in the slope is responsible for the start of erosion and that this is a more important criterion than the location of the highest wave load on the slope.

8.7 Shape erosion profiles

Table 8.5 Maximum slope angles of the clay experiments

Experiment	Angle (°)	Clay condition	Clay type
DF1984a	30	Unstructured	High sand percentage
DF1984b	15-20	Unstructured	Low sand percentage
DF1992Sa	65	Moderately structured	Moderately high sand percentage
DF1992Sb	75	Structured	Low sand percentage
DF1992Sc	-	Moderately structured	Moderately high sand percentage
DF1992Sd	70	Structured	Low sand percentage
DF1983a	80	Unstructured	High sand percentage
DF1983b	45	Unstructured	High sand percentage

The differences between the profile gradients of the different experiments are large. They differ from 20°-70°, as it listed in Table 8.5. If a hole was developed and was getting time to develop further into the clay a steep erosion profile with a cliff shape was developed. This is for example the case for DF1992. If only a small hole was developed, like for DF1984, a cliff shape is not formed (slope angle 15°-20°). This could however still have developed in time if the experiment was continued. See the former paragraph for the erosion profiles of the different experiments.

Clay condition
If the properties of the clay experiments are investigated more closely, the structured (DF1992S and DF1992Sd) and moderately structured clays (DF1992Sa and DF1992Sc) appear to have steep profiles. The unstructured clay of DF1984a and b have more gentle erosion profiles (30° and 15°-20°), which could thus be a result of the unstructured clay condition. DF1983 and b also contain unstructured clay, but these developed erosion profiles differ significantly (80° and 45°). The erosion took place in the grass sod, which has a high sand percentage. This can possibly be due to some roots in the clay which could have influenced the erosion process and the steepness and shape of the developed cliff profile. Remarkable is however that the clay types of DF1984a and DF1984b differed and of DF1983a and DF1983b were similar.

Clay type
Within the subdivision between clay conditions (moderately structured and structured clay versus unstructured clay), only for unstructured clay a subdivision between clay with a high sand percentage and a low sand percentage can be made. Clay with a higher sand percentage has a steeper erosion profile, however the differences in steepnessess of the clay with a low sand percentage are large.
For the moderately structured and structured clay the influence of the clay type cannot be proved on the basis of the available data sets.

8.8 Discussion and conclusions

8.8.1 Discussion

The discussion relating to the differences between experiments and actual storms is given in Chapter 10, Paragraph 10.6.

- There is not much data available on experiments on the behaviour of clay under wave loading, especially if the subdivision per clay condition is made. For each clay condition there are only a few experiment results available.
  The clay layers which were tested with the Delta Flume experiments had a maximum thickness of 0.8m. The erosion developed thus to a maximum depth of 0.8m, but for most experiments the erosion process was even stopped before the whole clay layer was eroded. DF1984b was even stopped at a depth of less than 0.1m. If these experiments were continued and were performed on thicker clay layers, more information on the erosion process of clay in depth would be known.

- The subdivision in the different clay types and clay conditions is made according to descriptions given in the reports of the different experiments. All these reports have different authors, which all have a different way of describing the sand content and the degree of structuration of the clay and not everywhere quantitative values have been given on this. Therefore the categorization in clay conditions could be slightly different from the real clay properties.

- The edges of the flume form boundaries which could have disrupted the erosion process or have given another erosion development than that which would occur in reality when the whole width of the dike is loaded.

- On real dikes a soil structure will always develop over time, which decreases in depth. This reduces the strength of the clay layer, but also some settlement will occur which will compact and thus strengthen the clay.

- A very important factor in the erosion process is that the erosion of a clay layer will always start at the weakest location on a dike slope. In the Delta Flume it is unknown if the relatively small width contains such a weak location or not. This influences the outcome of the experiment. If a weak location is present in the Delta Flume, the test results will be representative for the real situation. If a weak location is not present in the Delta Flume, the erosion rate of a perfect clay layer is determined. The actual condition and presence of these weak locations of a clay layer in the Delta Flume is unknown. Also the properties of a weak location are difficult to define.

- The four experiments on unstructured clay (DF1983a, DF1983b, DF1984a and DF1984b) reflect not realistic conditions for the upper part of clay layer on a dike, because the clay was constructed in the Delta Flume by applying and compacting of thin layers just before the experiment was started. Therefore the unstructured condition was obtained. In time however, a soil structure will develop, see Chapter 6. However these experiments give useful results for the behaviour of unstructured clay which will be present deeper in a clay layer.

- The experiments on unstructured clay contained different Iribarren parameters than the other experiments (ξ=1.3). DF1984 (H_s=1.05m, T=12s, ξ=4.2) contained surging breakers, so no real breaking occurred on the slope. DF1983 ((H_s=1.57m, T_p=5.26s, ξ=0.7) were plunging breakers, but due to the gentle slope the waves were hitting the water layer of the previous waves and not directly onto the slope. This reduced the load on the slope. Because the breaker types were different, the wave loading of these experiments were different and therefore also the erosion mechanisms could be different than of the other experiments.

  A different breaker type, thus a different Iribarren parameter could result in a different erosion mechanism. If the waves are of a plunging breaker type, the breaker type for which most experiments are performed and which results into the highest wave load on the slope, the wave impact, a cliff shape develops. It is possible that for other breaker types for which a different load type on the slope is present, a different erosion mechanism could occur. If the load is not an impact, but more run-up and run- down on the slope, then the erosion mechanism could be more like scour. For a certain water level and wave height the erosion process starts and a hole starts to develop. This could probably have a certain initial strength, a find of fatigue. The proceeding erosion mechanism is depends on the breaker type of the waves.

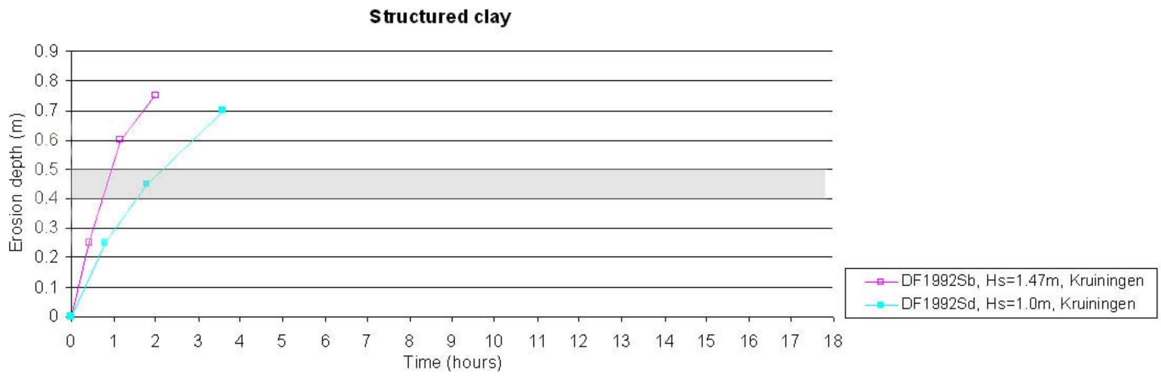
This makes a comparison of experiments with different breaker types not appropriate, however because this is the only data available, the results are still applicable.

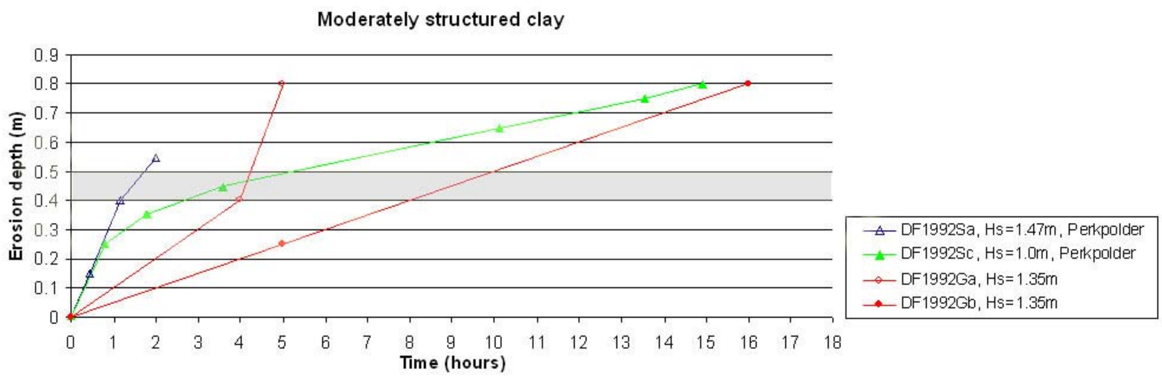
- In clay on a dike vegetation growth always will develop and therefore roots will be present in the clay decreasing in depth. These roots can influence the erosion process. In the experiments on the residual strength of a clay layer under stone revetment (DF1984 and DF1992S) roots were not present. In the experiments on the residual strength of clay under grass sods (DF1983 and DF1992G), there could have been some roots, which could have had some influence on the erosion development over time. An intact grass sod has a high resistance against erosion as can be concluded form the experiment on the grass sod, see 8.4. However more experiments on the behaviour of grass under wave loading should be performed because at the moment only two tests are available (DF1992Grass I and DF1992GrassII).

- In the erosion development of the moderate structured clay of DF1992Sc a transition to observe at a depth of approximately 0.4m where the erosion process is decreasing. It is possible that the clay deeper in the layer is less structured than the upper part and therefore the erosion rate is lower, deeper in to the layer. The can be a result of settling of the clay. It can be assumed that the clay in this area is a transition between structured and unstructured clay. The unstructured clay erodes more slowly than the structured clay.

  This level of 0.4-0.5m is also given in the graphs of the other clay experiments in Figure 8.18. Also for DF1992Sb of structured clay and DF1983 of unstructured clay a decrease in erosion rate is observed at approximately 0.4-0.5m. It is therefore assumed that a transition in clay conditions could take place at approximately this depth: the transition between structured and unstructured clay.

  The development of a soil structure is a function of time, see chapter 6, so the transition at a depth of 0.4-0.5m is not a fixed line, but will move deeper in depth in time of years. This level will therefore not be the same for all experiments and a more gradual transition could develop as well.





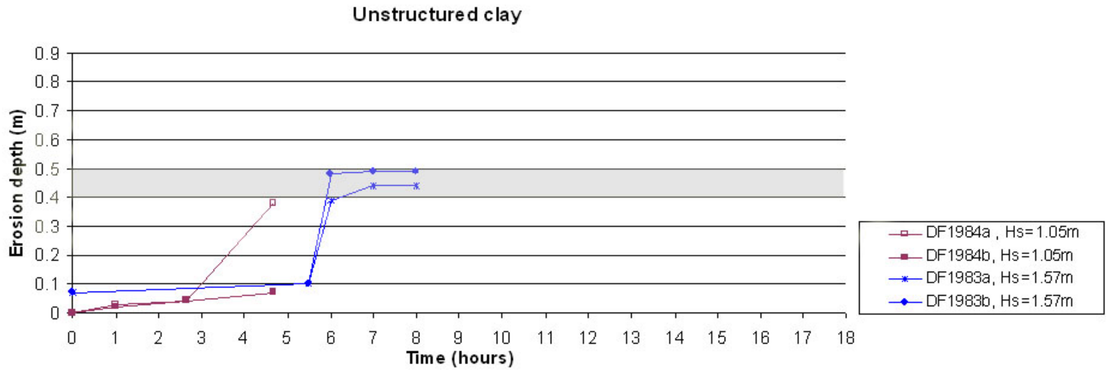


Figure 8.18 Transition between structured and unstructured clay at depth of 0.45m.

Based on this transition, the development of the erosion depth over time could be described roughly as given in the graph of Figure 8.19. The grass on the cover layer is included here for the overall picture. This grass here is assumed to have a high resistance to wave loading. The erosion of the structured clay (II) will develop with a certain high erosion rate, if the amount of structure decreases (III), the erosion rate also decreases. Unstructured clay (IV) has a low erosion rate.

A soil structure will develop over time, therefore the upper part of the clay layer will always be structured after a few years, see also chapter 6. An older clay layer will for a larger part be structured. This is also given in the graph.

This is further elaborated in Chapter 11.

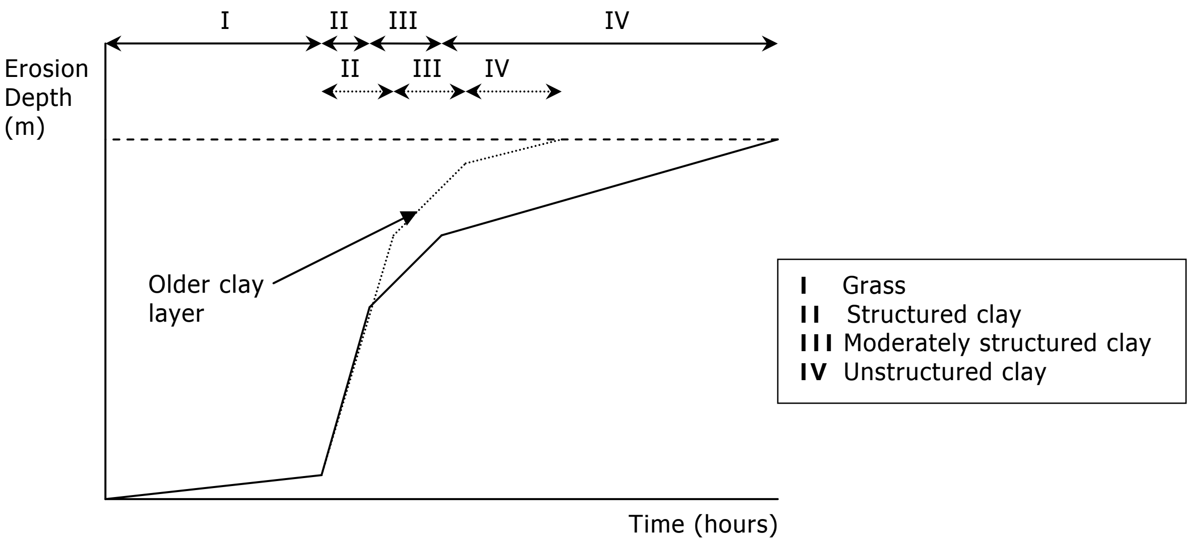


Figure 8.19 Schematized development of the erosion development of a revetment consisting of a clay layer in depth over time.

8.8.2 Conclusions

For this research the choice has been made to investigate and make a comparison of all performed large scale experiments which involve clay under wave loading. This is done in order not to make other assumptions on the behaviour of clay like already have been made several times in recent history, but to give a complete overview of all large scale and relevant experiments. This can be used to verify if a comparison of these experiments will result in new valuable conclusions and to validate existing conclusions and assumptions on the basis of more data. This chapter has shown that the overview and comparison of all large scale experiments has indeed resulted into a clearer view of the behaviour of clay under wave loading and this has resulted in several useful conclusions.

Erosion development

On basis of a comparison of all large scale experiments it can be concluded that the clay type is not the major influencing factor on the erosion rate of clay under wave loading, because a subdivision of the experiments based on the clay type only does not give coherent results. The clay condition, which is the extent of soil structure development, gives a reasonably good subdivision of the experiment results. It can therefore be concluded that the influence of the clay condition on the strength of the clay under a wave load is significant. The clay condition can be divided in three conditions: structured clay, moderately structured clay and unstructured clay. Structured clay has a significant higher erosion rate than unstructured clay. Moderately structured clay forms a transitional condition.

Based on this subdivision in clay conditions, the clay type is indeed of influence. For structured clay no conclusions can be drawn based on the available data, but for moderately structured clay, clay with a low sand percentage has a lower erosion rate than clay with a moderately high sand percentage. In unstructured clay with a high sand percentage a sudden high erosion rate can develop. Unstructured clay with a low sand percentage has a very high resistance against erosion, especially in comparison to structured clay.

The wave conditions and the slope angle are of influence on the erosion development as well. For structured clay a higher wave load results in a higher erosion rate. For moderately structured clay is to observe that the sand percentage in the clay is initially of more influence on the erosion rate than the wave height. From a depth of 0.4m the wave height has more influence on the erosion development than the clay type.

The clay which is compacted after applying appears to give a very high resistance to erosion in comparison with the other experiments. However, on real dikes a soil structure will develop over time and therefore the strength will be lower, but also some settlement will occur which can strengthen the clay layer. Despite these facts, it is clear that a smaller amount of soil structure development gives a higher resistance of clay against wave loading.

A comparison of the erosion development under wave loading of clay to the erosion development of a grass cover results in the conclusion that grass has a higher erosion resistance against wave loading than clay. The erosion rate of unstructured clay with a low sand percentage is approximately similar to the erosion rate of grass.

An empirical formula to the residual strength of clay under wave loading, which determines the erosion depth over time, was formulated by WL|Delft Hydraulics (2006b). This formula fits well to the data it is based on, but it does not deliver useful results when compared to the data of the other larger scale experiments. In the empirical formula as formulated by WL|Delft Hydraulics, the erosion depth is dependent on the wave height and the clay type. From the comparison of all large scale experiments is however concluded that the especially the clay condition is an influencing factor in the erosion process of clay and the slope angle could also be of influence.

Location of erosion

The erosion of clay starts below the water level by the development of a hole, approximately at a level of $0.3H_s$ and $1.2H_s$ below the water level. The hole can extend and develop until above the water level if the erosion process has sufficient time to continue. For some experiments the whole width of the flume was eroded, for some experiments only a hole was developed. The start of erosion was for all experiments a hole.

The defined theory of the different erosion zones along the slope by Smith (1994a) is not completely corresponding to all experiments.

Shape erosion profile
After an initial erosion hole is developed, the erosion profile of the clay has a steep slope with the shape of a cliff. The differences between the angles of the steep slopes of the different experiments are large, it differs from 20°-70°. If a hole was developed and was getting time to develop further into the clay a steep erosion profile with a cliff shape was developed.

General conclusion
On basis of this research, it can be deduced that conclusions based on one or just a few experiments should be interpreted carefully. The empirical formula [WL|Delft Hydraulics, 2006b] as well as the theory of the erosion zones [Smith, 1994a] could not completely correspond to all other experiments.

9 Comparison with erosion of sand

In this chapter the behaviour of clay and the behaviour of sand under wave loading are compared. The reasons for this comparison are given in the first paragraph. Based on the results of the former chapter, it is supposed that structured clay could erode quite similar to sand. Also clay with a high sand percentage is expected to erode more similarly to sand than clay with a low sand percentage.

In the first paragraph arguments for including a comparison with the erosion of sand are given, after which a short overview of the knowledge on the erosion process of dunes is given. Results and properties of the Delta Flume experiments on sand, which are used for the comparison with clay, are described in the third paragraph. The experiments are compared on the erosion rate, the location of erosion along the slope and the shape of the erosion profile. Finally conclusions are provided.

9.1 Reasons for including research to erosion of sand

The availability of the test data of the large scale experiments on clay in the Delta Flume is limited. The only other erosion data of clay available are the observations made after the storm surge disaster of 1953, these measurements are however also limited. To obtain more insight into the behaviour of clay in respect to that of sand, a comparison with the erosion rate of sand is made. The cross-shore transport during dune erosion has been studied extensively over the past years and therefore much more information about the behaviour of sand under wave loading is known. A rough comparison of the erosion of sand with the erosion of clay gives an idea of the differences in the order of magnitude of the erosion rate, the location of erosion along the slope and the shape of the erosion profile. Clay can only be considered to be a safe dike cover material if it erodes clearly slower than sand.

9.2 Erosion process of dunes

9.2.1 Description of the erosion process of dunes

In this paragraph a brief outline of the erosion process of dunes is presented. For a more extensive description is referred to Vellinga (1986), van de Graaff (2004) and WL|Delft Hydraulics (2006a).

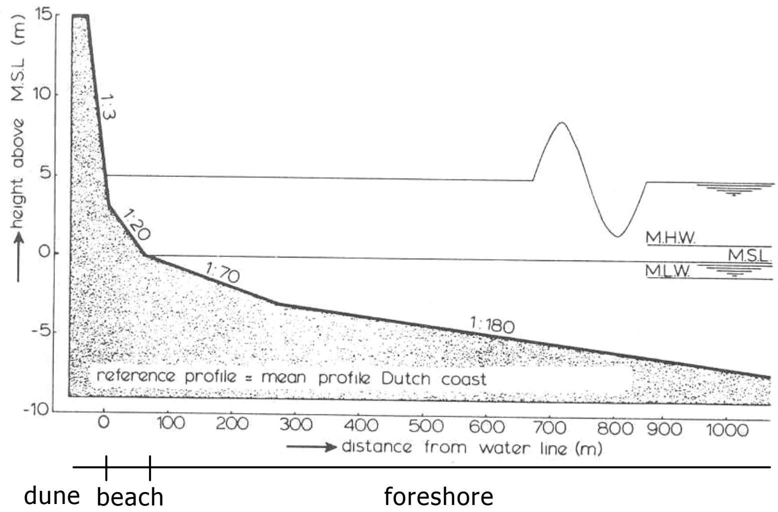


Figure 9.1 Characteristic dune profile of the Dutch coast, with a water level at storm surge level. The dune face has a slope of 1:3 or 1:4 which ends at approximately NAP+3m. Till a level of NAP the slope is 1:20, 1:70 from NAP to NAP-3m and from NAP-3m 1:180. An average grain size of the dune sand is (D_{50}) 200µm. [Vellinga, 1986 and WL|Delft Hydraulics, 2006a]

A characteristic dune profile of the Dutch coast is given in Figure 9.1. During normal conditions the dune profile is considered to be in a more or less dynamic equilibrium. During a storm surge the water level and wave attack are higher than usual and this equilibrium state is disturbed. Large amounts of sediment erode from the dune face and are deposited in front of the dune, on the foreshore. See Figure 9.2. In this way the dune profile is forced into a new equilibrium that fits better to the changing conditions. Because of the limited duration of a storm it is however not expected that a real equilibrium profile will develop during storm conditions.

Dune erosion during a storm surge leads to a fast decreasing width of the dune in a relatively small period of time. After the storm however, the profile will gradually return to the pre-storm equilibrium situation. During these normal conditions the wind and smaller waves transport the eroded sand back to the upper part of the beach profile. Dune erosion is thus a reversible process and therefore the hinterland is only threatened by a breakthrough when the width of the dunes becomes too small.

Initially the slope of the dune face becomes steeper and the dune face will retreat thereafter. Delta Flume experiments on sand of 2006 show that this retreat is non-linear in time. The eroded sediment is deposited in front of the dune and in this way a closed sediment balance is present if no longshore transport takes place. Due to the accretion, the depth of the beach becomes smaller and the wave breaking process will start earlier. In this way the wave attack on the dune becomes less severe, the wave energy is reduced, which results into less suspended sand and thus a decrease of the erosion rate in time.

The current calculation method is developed to describe equilibrium profiles for closed sediment balances. The model DUROSTA is a time dependent, physical-based cross-shore transport model which determines dune erosion in time based on sediment transport.

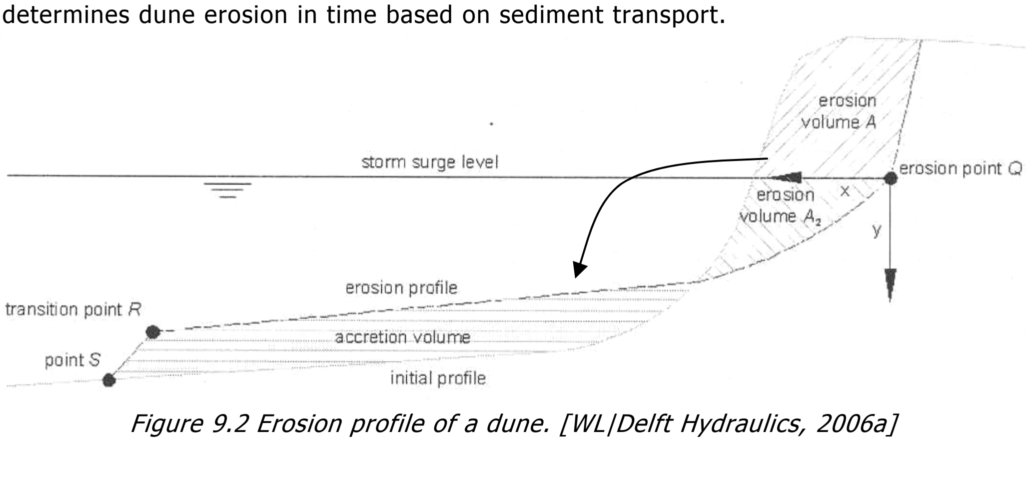


Figure 9.2 Erosion profile of a dune. [WL|Delft Hydraulics, 2006a]

9.2.2 Mechanisms of dune erosion

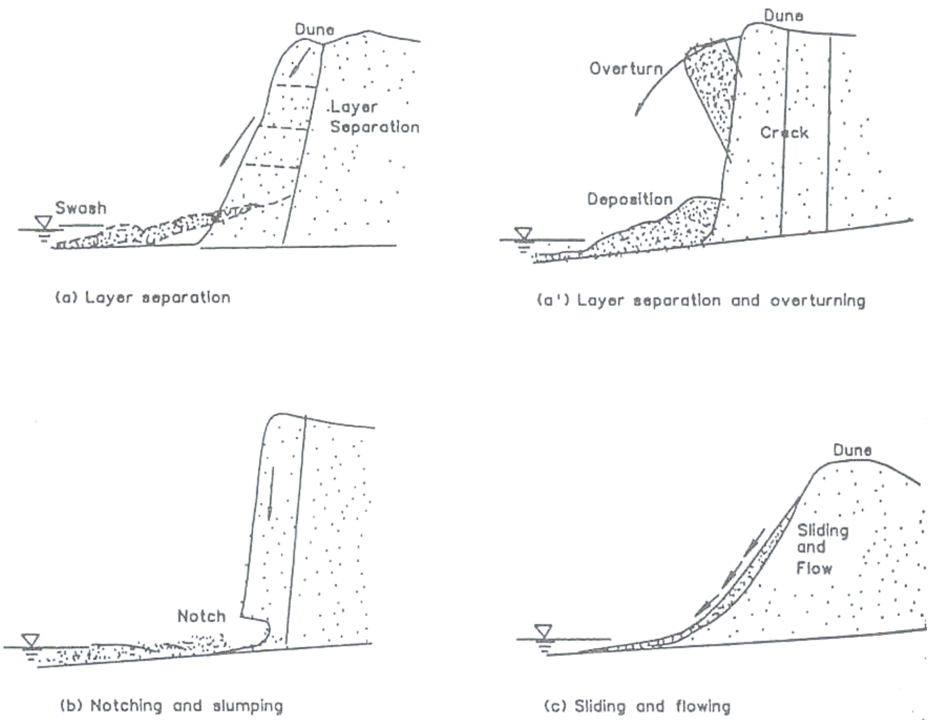


Figure 9.3 Dune erosion mechanisms by cross-shore processes. [Nishi, 1996]

Nishi and Kraus (1996) have formulated three types of cross-shore erosion mechanisms of sand dunes by wave impacts during storms or strong wave action, which are identified though field observations: See Figure 9.3.

Layer separation
Layer separation typically occurs if a near-vertical dune face is subjected to wave impact. Over the duration of a certain number of impacts a vertical crack develops. The developed outer layer, which is typically 0.3-0.5m thick, gradually separates from the dune. It either collapses suddenly (Figure 9.3a) or tilts forward and overturns (Figure 9.3b).

Notching and slumping
Notching occurs if a dune slope is nearly vertical and permeated by roots or highly compacted. It is limited to the elevation of wave attack and after the notch is sufficiently deep, the overlying sand column collapses. The width of the sand deposition at the foot of the dune face is less than that of layer separation which involves overturning and sliding of the sand layer.

Sliding and flowing
Sliding and flowing occurs on uncompacted gently sloping dunes which have a dune face close to the angle of repose of the sediments forming the dune. Modest wave impact at the base of the dune or even pelting by rain or exposure to strong winds can cause a thin layer of sand to run down the slope. This erosion mechanism will not result in severe dune erosion in a short period of time, but will make the slope steeper which can induce the erosion mechanisms of layer separation or notching and slumping under storm conditions.

Inner surf and swash zone processes redistribute the sand over the foreshore.

The degree of compaction was found to be a significant parameter that increases the erosion resistance of the dune. The eroded volume of a compacted dune was less than that of an uncompacted dune with a gentler slope.

9.3 Delta Flume experiments on sand

Delta Flume experiments on sand are used to compare the erosion behaviour of sand to the erosion behaviour of clay. These large scale experiments are used, because the erosion development of sand is determined for different time steps during the experiments and in the experiments on sand comparable hydraulic conditions are used as in the experiments on clay. Two Delta Flume experiments on sand, performed in 1981 and in 2006, are used.

The Delta Flume experiments on sand were performed to investigate the behaviour of dunes under wave loading. A characteristic dune profile of the Dutch coast was built in the Delta Flume on scale, using scaling relations developed by Vellinga (1986) [WL|Delft Hydraulics, 2006a]. Due to this scaling, the wave characteristics are smaller and the initial slope is steeper. These scaling relations are complicated and because it is only possible to make a rough comparison of the erosion behaviour, the unscaled experiment results will be used for the comparison. The scaled wave characteristics used in the experiments are approximately the same as the wave characteristics used in the experiments on clay. Therefore the experiment properties will be used and not the prototype properties. In this way uncertainties in the results due to scaling are also avoided.

The erosion of clay is measured during the experiments by a maximum erosion depth. For sand the erosion is measured in volumes of eroded material. To make a comparison, for and the horizontal erosion depth is measured. See also 9.4.2.

9.3.1 Delta Flume 1981

In 1981 Delta Flume experiments to investigate the behaviour of dunes under storm conditions were performed. The properties of the experiments are listed in Table 9.1 and an overview of the results is given in Table 9.2 and Table 9.3. Examples of the erosion profiles are given in Figure 9.4, the erosion profiles of the other tests are given in Appendix XIV.

Table 9.1 Properties of the Delta Flume experiments on sand 1981.

Properties DFSand1981			
	T1	T2	T5
Wave height	1.52m	1.52m	2.0m
Wave period	5.4s	5.4s	7.6s
Water depth	4.2m	4.2m	5m

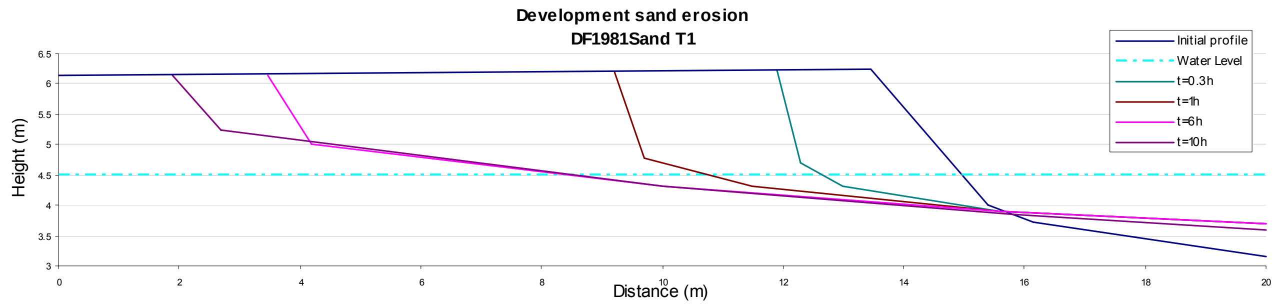
D_{50}=225μm
Profile steepness factor S_0=2
Depth scale factor n_d=5
Estimated fall velocity sand w=0.0268m/s

Table 9.2 Erosion development of the Delta Flume experiments on sand 1981.

Time (hours)	Horizontal erosion (m)			Erosion volume (m^3/m) [de Rijke, 1983]		
	T1	T2	T5	T1	T2	T5
0	0	0	0	0	0	0
0.1	1.5	1.5	3.0	2.64	1.76	10.82
0.3	2.6	2.2	4.4	4.71	3.92	18.03
1	5.2	4.4	7.0	9.42	7.08	27.73
3	8.5	7.0	10.7	15.76	11.07	40.59
6	10.4	8.5	12.6	17.77	13.25	49.41
10	11.5	9.6	-	19.22	14.77	-

Table 9.3 Slope angles of the Delta Flume experiments on sand 1981.

Slope	T1	T2
Initial profile	1:0.8 (50°)	1:1.2 (40°)
During the experiment	1:0.4 (70°)	1:0.4 (70°)
Final profile	1:0.8 (50°)	1:0.8 (50°)



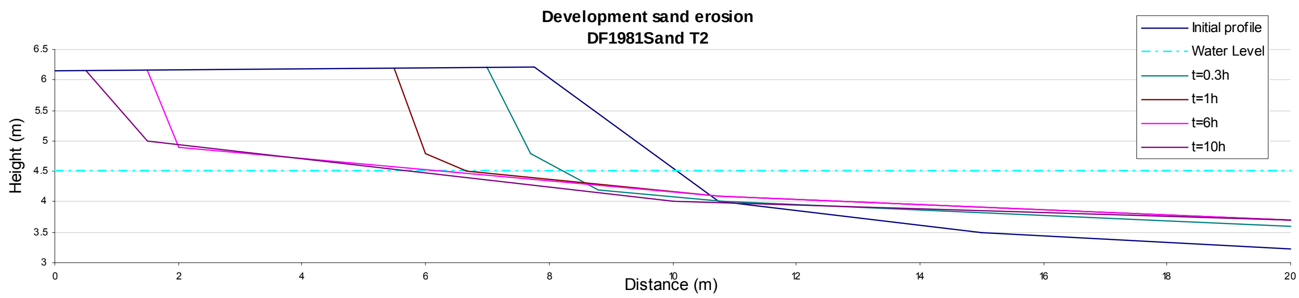


Figure 9.4 Erosion development in the sand profiles of test T1 and T2, Delta Flume 1981.

9.3.2 Delta Flume 2006

In 2006 Delta Flume experiments on sand were performed to verify the effects of the wave period on dune erosion. It was expected that longer wave periods increase dune erosion. In the Delta Flume of WL|Delft Hydraulics a characteristic profile of the Dutch coast was loaded with a wave load with three different wave periods. The same profile and hydraulic conditions as the 1981 Delta Flume experiment T2 (DF1981Sand T2) were applied. An overview of the prototype properties are given in Table 9.4 and the experiment properties in Table 9.5.

Examples of the erosion profiles are given in the figures 14.5 and 14.6. A total overview of all erosion profiles is given in Appendix XIV. The results of the experiments are given in Table 9.6 and Table 9.7.

Table 9.4 Properties of prototypes of the Delta Flume experiments on sand 2006.

Prototype properties			
	T01	T02	T03
Wave height	9m	9m	9m
Wave period	12s	15s	18s
D_{50}	200μm	200μm	200μm

Table 9.5 Properties of the Delta Flume experiments on sand of 2006.

Properties DFSand2006			
	T01	T02	T03
Wave height	1.41m	1.49m	1.51m
Wave period	4.9s	6.1s	7.2s
Water depth	4.5m	4.5m	4.5m

D_{50}=210μm
Depth scale factor n_d=6
Fall velocity sand
w=0.0268m/s

Table 9.6 Erosion development of the Delta Flume experiment on sand 2006.

Time (hours)	Horizontal erosion (m)			Erosion volume (m^3/m) [WL|Delft Hydraulics, 2006a]		
	T01	T02	T03	T01	T02	T03
0	0	0	0	0	0	0
0.1	1.4	1.0	1.4	0.90	1.01	1.15
0.3	2.2	1.9	2.2	2.13	2.29	2.48
1	3.6	3.2	4.1	4.23	4.58	5.31
2.04	4.8	4.6	5.5	5.88	6.32	7.10
6	6.6	7.0	7.5	8.60	9.57	9.85

Table 9.7 Slope angles of T01, Delta Flume experiment on sand 2006.

Slope	T01
Initial profile	1:1.4 (35°)
During the experiment	1:0.4 (70°)
Final profile	1:0.2 (80°)

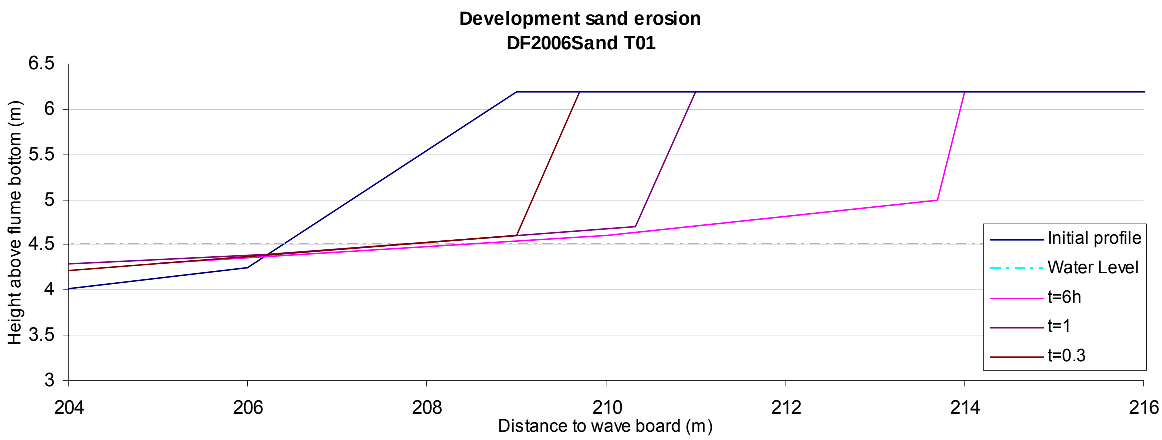


Figure 9.5 Erosion development in the sand profile of test T01, Delta Flume 2006.

9.3.3 Conclusions on the erosion of sand

Erosion development

- The initial erosion rate is high, but the decrease in erosion rate due to the accretion of sand is visible in the development of the erosion profiles.

Location of erosion

- The erosion development of all experiments was located above the water level.

Shape erosion profile

- The initial slopes of sand in the Delta Flume are very steep due to the scaling of the prototype dunes.
- The erosion of sand under wave loading develops in two phases. At first the slope gets steeper until it reaches its erosion profile steepness, which has the shape of a steep cliff. This cliff retreats in horizontal direction.
- The steepnesses of the developed cliffs were 50° to 70°.

9.4 Comparison of the experiments of clay and sand

For a comparison of the erosion of clay and the erosion of sand, based on the Delta Flume experiments, three factors are examined. Firstly the differences in erosion rates are considered. This comparison is based on the clay condition which appeared in the former chapter to form the basis of a good subdivision of the different experiments. The clay type, the wave height, the wave period and the slope angle are also mentioned briefly. Finally the location of erosion along the slope and the shape of erosion profile are investigated.

Due to limited available data it is not possible to make a careful comparison. This comparison will just give a general idea of the differences in erosion behaviour of clay and sand.

9.4.1 Assumptions

This comparison is only used to get a rough insight into the behaviour of clay compared to that of sand. Therefore only the experiments which have approximately the same properties (for example H_s and T_p) are used to be compared to each other. In this way the influence of these properties on the comparison is avoided.

In this respect DF1981Sand T5 is excluded from the experiments, because the wave height (H_s=2.0m) is higher than the wave heights used in the experiments on clay. A wave height of approximately 1.5m is used in the experiments sand and also in some of the experiments on clay. The water levels and wave periods, except for DF2006Sand T02 and T03 differ not much from each other in these cases. The experiments form therefore a good dataset for a comparison. DF1992Sc and d and DF1984 are not used in the comparison of the erosion rate, because the wave height is 1.0m. In Table 9.11 an overview of the properties of the Delta Flume experiments on clay and sand which are used in these comparisons is given.

The clay properties differ for each experiment. For this comparison it is assumed that the properties of sand do not differ as much from each other as the properties of the clay, and it is therefore assumed that the sand properties are the same for all experiments.

9.4.2 Erosion development

Direction measured erosion

Before making a comparison, first it should be determined which measured erosion rate will be compared to each other. In the experiments on clay a maximum erosion depth was given perpendicular to the slope. During the Delta Flume experiments on sand the erosion was measured in volumes of eroded sand and from the erosion profiles the horizontal erosion development is measurable.

At first glance the erosion rate of sand is seems to be larger than that of clay. Due to the development of a steep slope, it is not possible to measure the sand erosion perpendicular to the slope during the whole erosion process. The perpendicular erosion in sand can be approached by two methods. Method A, by taking the maximum perpendicular erosion depth, and using that point on the slope for further measurements, see Figure 9.6 A. In this way the erosion will finally become almost horizontal. This is prevented by constantly using the perpendicular erosion depth, method B. In time this depth will however be almost constant if the crest is reached, see Figure 9.6 B. Both types of measuring are not similar to the way the erosion of clay is measured. Therefore the erosion will be compared in both ways, because it is impossible to measure the erosion development in the same way. Because only a rough comparison of orders of magnitude of the difference in erosion rate is necessary, a this comparison is justifiable. By using both methods, the differences in erosion development of both methods can be observed.

Besides the perpendicular erosion, also a comparison of the horizontal erosion development is made (method C). This is the most accurate way to measure the erosion of sand besides measuring the erosion volumes, see Figure 9.6 C. The horizontal erosion of clay is determined on basis of the erosion profiles and the measured erosion during the experiments.

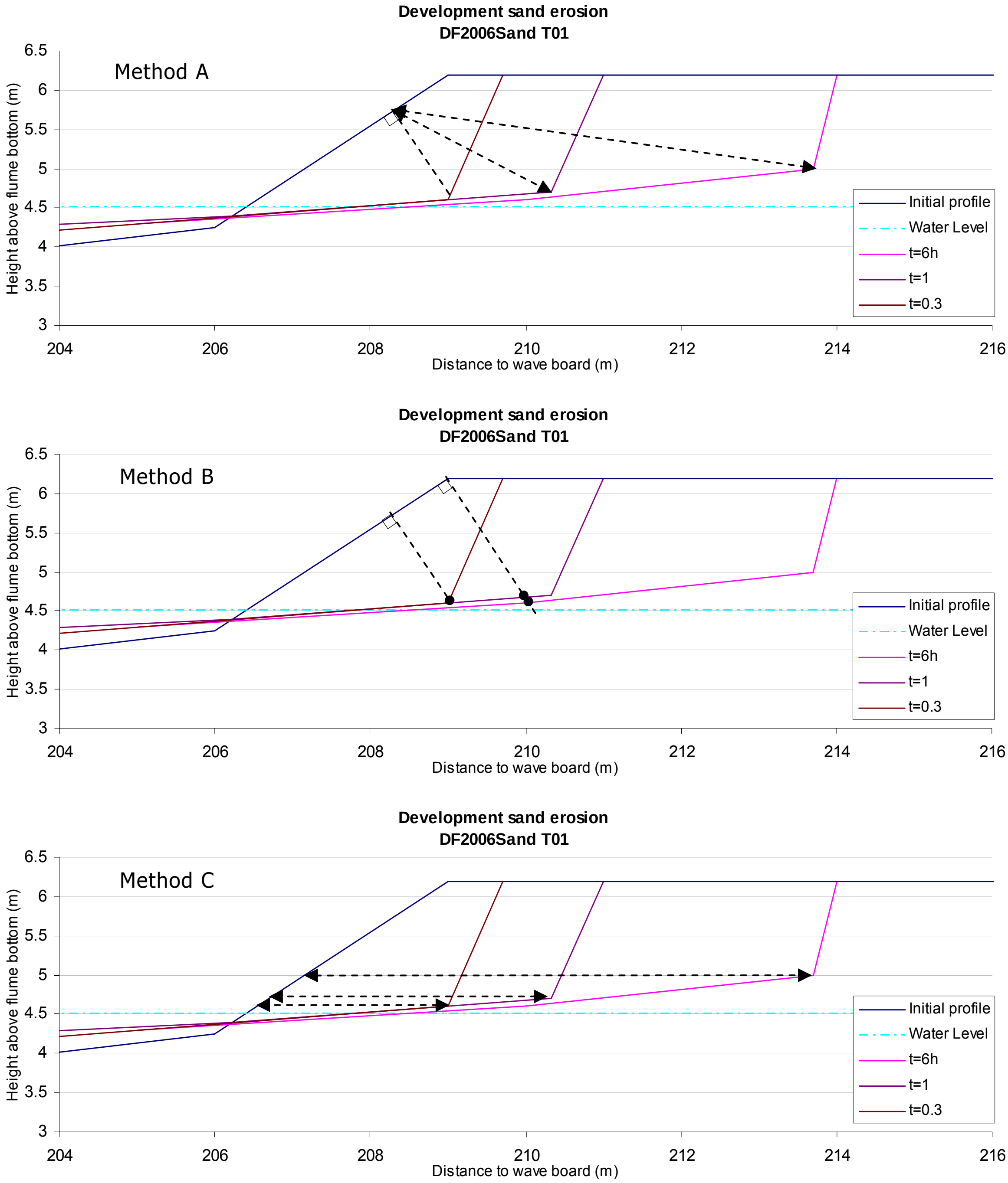


Figure 9.6 Three ways of measuring erosion depth. A: Maximum perpendicular erosion depth, and use that point on the slope for further measurements. B: Constant perpendicular erosion depth. C: Horizontal erosion depth.

Erosion perpendicular to the slope, method A and B

Method A

The erosion depth of sand is measured perpendicular to the slope according to method A, given in Figure 9.6 A. This perpendicular erosion is approached by taking first the maximum perpendicular erosion depth. This point on the slope is used for further measurements. The sand erosion determined by this method is compared to the perpendicular erosion depth of clay in Figure 9.7.

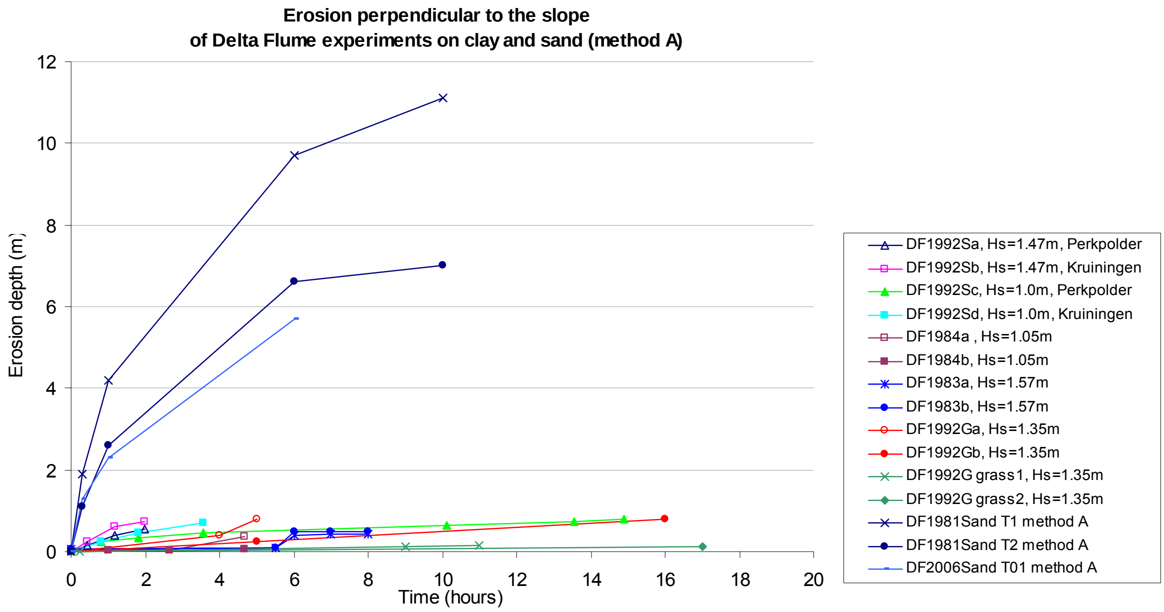


Figure 9.7 Erosion of clay and sand in Delta Flume experiments. The erosion depth of sand determined with method A.

Results

- The erosion rate of sand is clearly higher than of clay.
- After approximately two hours the erosion depth of sand is 4.0m, which is approximately a factor of 3-5 larger than clay.
- After approximately six hours the erosion depth of sand is 7.3m, which is approximately a factor of 6-10 larger than clay.
- The perpendicular erosion of sand determined with method A is thus approximately a factor of 3-10 higher than clay.

Method B

For this approach the erosion depth of sand is measured according to method B given in Figure 9.8. The perpendicular erosion depth is constantly used. In time the erosion depth will thus be almost constant if the crest is reached. The erosion depth obtained in this way is compared to the perpendicular erosion depth of clay in Figure 9.8, where the results of the method A are also included.

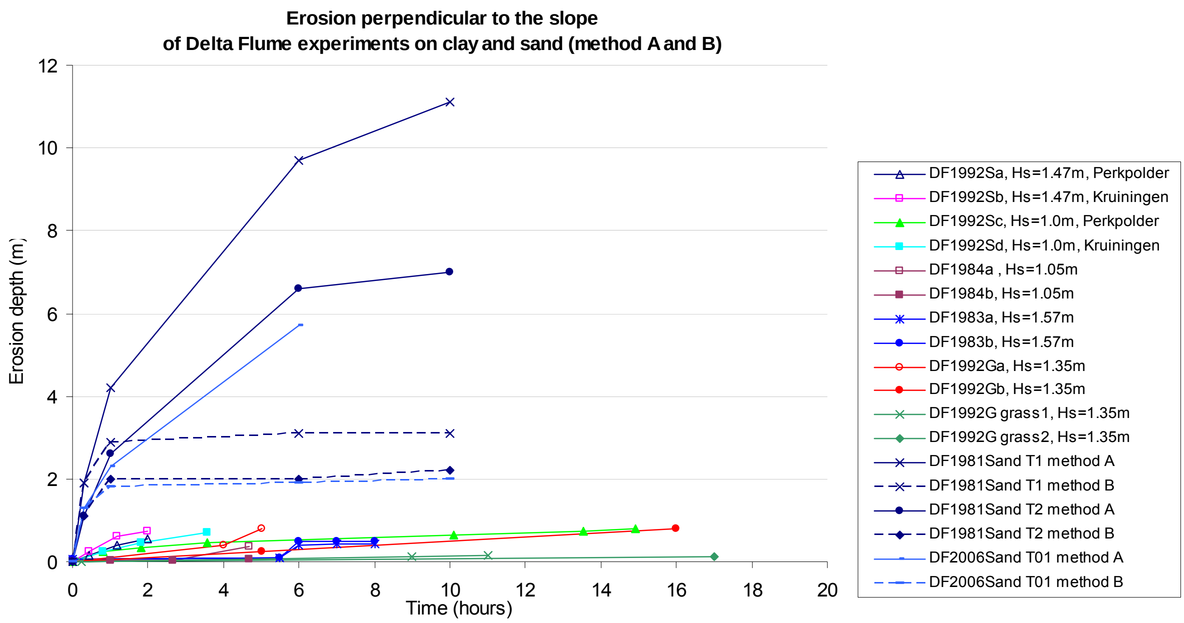


Figure 9.8 Erosion of clay and sand in Delta Flume experiments. The erosion depth of sand determined with method A and B.

Results
- The erosion rate of sand is clearly higher than that of clay, but it is lower than the sand erosion determined with method A.
- After approximately two hours the erosion depth of sand is 2.2m, which is approximately a factor of 2-3 larger than clay.
- After approximately six hours the erosion depth of sand is 2.3m, which is approximately a factor of 2-3 larger than clay.
- The perpendicular erosion of sand determined with method B is thus approximately a factor of 2-3 higher than clay.

Method A and B
The differences of the determined erosion rates between the two methods is significant. If the erosion rate is approached by the average of the two methods, this results in an erosion rate of a factor of 2-5 higher than of sand after two hours and of a factor of 2-10 higher after six hours. The perpendicular erosion of sand is in that case approximately a factor of 2-10 higher than clay.

Horizontal erosion, method C

Horizontal erosion clay
In order to make a comparison of the horizontal erosion of the Delta Flume experiments on clay and sand, also for clay the horizontal erosion has to be determined. These values are not measured in the Delta Flume, and are therefore determined from the drawn erosion profiles. The results are given in the tables below (Table 9.8, Table 9.9 and Table 9.10). From DF1992G no erosion profiles are available and therefore the horizontal erosion is not determined.

Table 9.8 Horizontal erosion of Delta Flume experiment DF1984.

Time (h)	Maximum horizontal erosion (m)	
	a High sand %	b Low sand %
2.67	0.1	-
4.67	1.3	0.25

Table 9.9 Horizontal erosion of Delta Flume experiment DF1992S.

Experiment	Time (h)	Maximum horizontal erosion (m)			
		Perkpolder		Kruiningen	
High water level	0.4	a	0.8	b	1.0
	1.2		1.8		2.4
	2.0		2.2		2.9
Low water level	0.8	c	1.4	d	1.2
	1.8		1.9		1.9
	3.6		2.1		2.7
	10.1		2.7		-
	14.9		2.9		-

Table 9.10 Horizontal erosion of Delta Flume experiment DF1983.

Time (h)	Maximum horizontal erosion (m)		
	a Hole 1	b Hole 2	Averaged
0	0.2	0.2	0.2
6	2.1	1.7	1.9
8	3.6	3.1	6.7

Table 9.11 Overview properties of the Delta Flume experiments on sand and clay.

Name experiment	H_s (m)	T (s)	Water level (m)	Slope	Duration t (h)	Clay type	Clay condition
Clay							
DF1984a	1.05	12	5.56	1:3.5	4.40	High sand percentage	Unstructured
DF1984b	1.05	12	5.56	1:3.5	4.40	Low sand percentage	Unstructured
DF1992Sa Perkpolder	1.47	4.9	5.0	1:4	2	Category 2 (Moderate)	Moderately structured
DF1992Sb Kruiningen	1.47	4.9	5.0	1:4	2	Category 1 (Low sand percentage)	Structured
DF1992Sc Perkpolder	1.0	4.2	3.5	1:4	15	Category 2 (Moderate)	Moderately structured
DF1992Sd Kruiningen	1.0	4.2	3.5	1:4	4	Category 1 (Low sand percentage)	Structured
DF1983a	1.57	5.26	5.0	1:8	8	In grass layer: High sand percentage In sublayer: Low sand percentage	Unstructured
DF1983b	1.57	5.26	5.0	1:8	8	In grass layer: High sand percentage In sublayer: Low sand percentage	Unstructured
Sand							
DF1981Sand T1	1.52	5.4	4.2	1:1.05	10	-	-
DF1981Sand T2	1.52	5.4	4.2	1:1.4	10	-	-
DF1981Sand T5	2.0	7.6	5	1:1.3	10	-	-
DF2006Sand T01	1.41	4.9	4.5	1:1.6	6	-	-
DF2006Sand T02	1.49	6.1	4.5	1:1.6	6	-	-
DF2006Sand T03	1.51	7.2	4.5	1:1.6	6	-	-

General comparison
An overview of the horizontal erosion of the experiments on sand is repeated in Table 9.12. The development of the maximum horizontal erosion over time of the Delta Flume experiments for sand and clay are given in the graph of Figure 9.9. It can clearly be observed that the development of the horizontal erosion of clay and sand over time is approximately the same. The amount of erosion of clay is however lower. Only DF1984 and DF1983 have a different erosion development over time.

The differences in the horizontal erosion of clay and sand are clearly smaller than the measured perpendicular erosion and therefore it is possible to discuss the results for each clay condition.

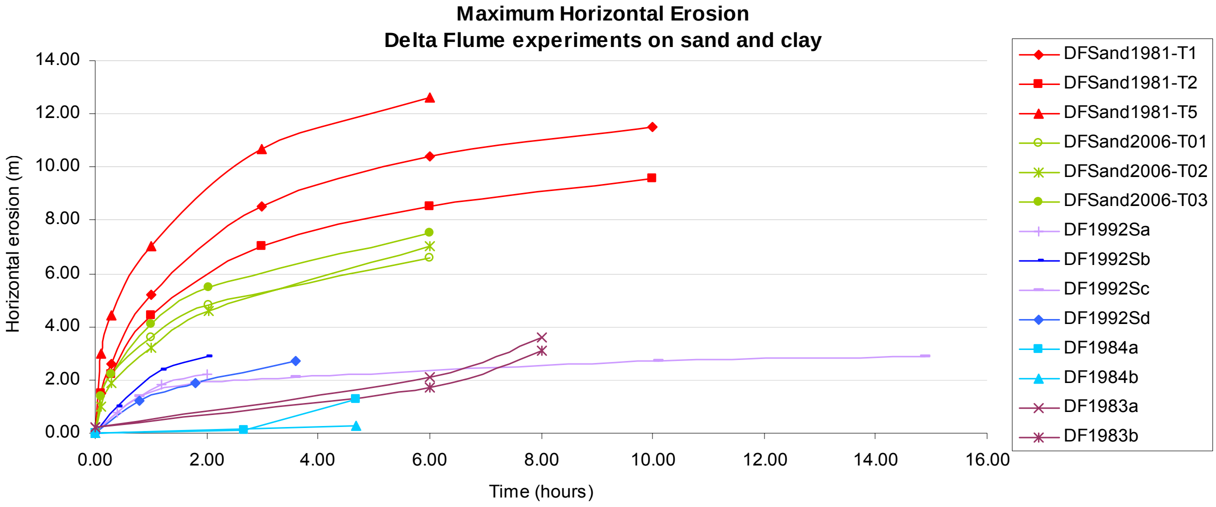


Figure 9.9 Development of the maximum horizontal erosion over time of the Delta Flume experiments on clay and sand.

Table 9.12 Erosion development of the Delta Flume experiments on sand.

Time (h)	Horizontal erosion DFSand1981 (m)			Time (h)	Horizontal erosion DFSand2006 (m)		
	T1	T2	T5		T01	T02	T03
0	0	0	0	0	0	0	0
0.1	1.5	1.5	3.0	0.1	1.4	1.0	1.4
0.3	2.6	2.2	4.4	0.3	2.2	1.9	2.2
1	5.2	4.4	7.0	1	3.6	3.2	4.1
3	8.5	7.0	10.7	2.04	4.8	4.6	5.5
6	10.4	8.5	12.6	6	6.6	7.0	7.5
10	11.5	9.6	-	-	-	-	-

Table 9.13 Used averaged horizontal erosion sand experiments.

Time (h)	Obtained from	Used experiments	Averaged horizontal erosion (m)
1	Measured	DF1981 T1,T2 +DF2006 T01	4.4
2	T1+T2: Graph	DF1981 T1,T2 +DF2006 T01	6.4
6	Measured	DF1981 T1,T2	8.5
8	T1+T2: Graph	DF1981 T1,T2	10

Clay condition
Structured clay
DF1992Sb is representative for structured clay and has a low sand percentage:
- After approximately one hour the erosion of sand is approximately a factor of 1.5-3 larger than of structured clay.
- After approximately two hours erosion of sand is approximately a factor of 1.5-3 larger than of structured clay.

Moderately structured clay
DF1992Sa is representative for moderately structured clay and has a moderately high sand percentage:

- After approximately one hour the erosion of sand is approximately a factor of 2-3.5 larger than of moderately structured clay.
- After approximately two hours erosion of sand is approximately a factor of 2-3.5 larger than of moderately structured clay.

Unstructured clay*
DF1983a and b are representative for unstructured clay, and have a high sand percentage in grass layer and a low sand percentage below this:

- After approximately six hours the erosion of sand is approximately a factor of 3-6 larger than of unstructured clay.
- After approximately eight hours the erosion of sand is approximately a factor of 2-3.5 larger than of unstructured clay.

*First part erosion in grass layer, will therefore probably be structured

Structured clay erodes approximately a factor of 1.5-3 slower than sand and moderately structured clay approximately a factor of 2-3.5. This is just a small difference. These results are not unambiguous. After 6 hours of wave loading the erosion did not progress in the depth, but in length and width of the slope, as can be seen in Figure 9.11 and Paragraph 4.2.1. At first the horizontal erosion of sand is approximately a factor of 3-6 higher than of the unstructured clay. However after 6 hours when the hole does not deepen anymore, the horizontal erosion of the unstructured clay increases and after eight hours the erosion of sand is decreased to a factor of 2-3.5 higher than of the unstructured clay. In this period only erosion in length and width was taking place, not into depth and this has probably distorted the outcome of the horizontal erosion rate of unstructured clay in comparison to sand. See also discussion.

Clay type
See the comparison of the clay condition. This comparison does not give a clear view on the influence of the sand percentage in the clay on the erosion rate. Clay with a low sand percentage (the clay type of the compared structured clay) erodes more similar to sand than clay with a moderately high sand percentage (the clay type of the compared moderately structured clay). This is not what would be expected. Also if the other experiments on clay with a wave height of 1.0m are included in the comparison, it cannot be proved that clay with a higher sand percentage behaves more like sand.

Wave height
DF1984 has a wave height of 1.0m and consists of unstructured clay similar to DF1983 (H_s=1.5m). The horizontal erosion of DF1984 develops slower than that of DF1983. This lower erosion could be due to a lower wave height, but because different clays are used, also other properties could play a role.
DF1992Sc and DF1992Sd also have a wave height of 1.0m, and the horizontal erosion also develops slower than that of DF1992a and DF1992Sb which have the same soil properties but a wave height of 1.5m. For Kruiningen clay the horizontal erosion after 2 hours of the low water experiment was a factor of 1.5 smaller than that of the high water experiment. For Perkpolder clay there is almost no difference (1.1 times smaller). Maybe the clay condition here is of influence, because Kruiningen clay is structured and Perkpolder clay moderately structured.

It could be assumed that a lower wave height results in a lower erosion rate. Unfortunately however the data is too limited to draw valuable conclusions.

Wave period
From the experiments on dune erosion (WL|Delft Hydraulics, 2006a) is concluded that a longer wave period results in more dune erosion. The erosion volume above still water level increases with 24% for an increase in wave period from T_p =12s to 18s (for prototypes), thus a wave period of 4.9 to 7.2 for a model in the Delta Flume). The data of clay is however too limited and there are too many influences from other parameters to make a comparison of the wave period for clay and sand.

Slope
The influence of the initial shape of the profile on the erosion rate of sand is unknown as yet. WL|Delft Hydraulics (2006a) recommended research be done into this subject.

The initial slopes of the sand profiles are much steeper than that of the clay profiles.

9.4.3 Location of erosion

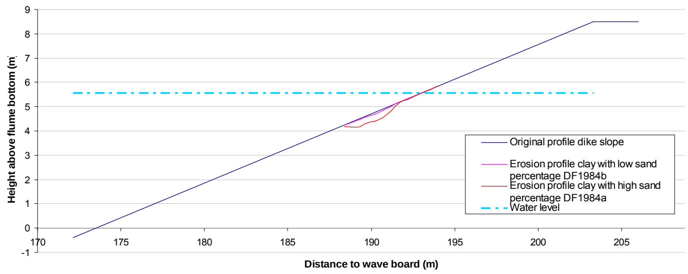
In sand the erosion develops mainly above the water level and only a little below this level, see 9.3. The erosion of clay starts in all cases below the water level and develops over time further into the slope and also above the water level, depending on the condition of the clay. See also 8.6.
The location of erosion is thus different for clay and sand.

Probably the erosion mechanisms of clay and sand are different.

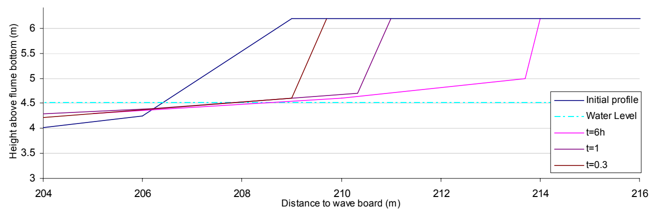
See Figure 9.10 for some examples of the erosion profiles of clay and sand.

Examples of erosion profiles

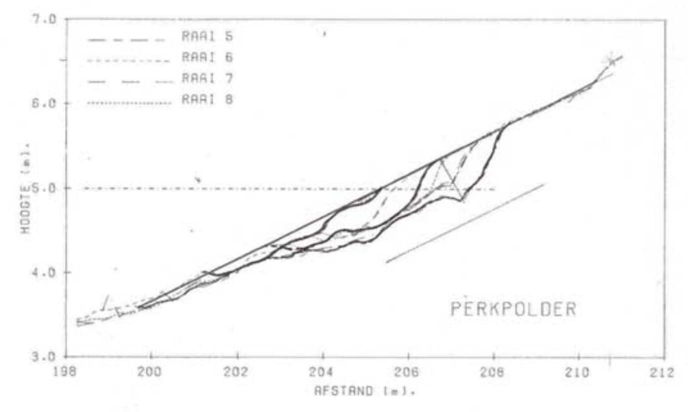
Experiments on Clay	Experiments on Sand



DF1984a and DF1984b: Unstructured clay with high (a) and low (b) sand percentage, after 4.67h.



DF2006SandT01: Profiles at 0h, 0.3h, 1h and 6h.



DF1992Sa: Moderately structured clay with moderately high sand percentage, profiles at 0h, 0.4h, 1.2h and 2h.

Figure 9.10 Some examples of erosion development in clay and sand. In clay (left) the erosion starts below the water level and can develop until above the water level. In sand (right) the erosion takes mainly place above the water level.

9.4.4 Shape erosion profile

After 0.1h a steep erosion profile was developed in the experiments on sand of approximately 70° with a horizontal. In the remaining part of the experiment, the steep dune front retreats in a horizontal direction. In clay the differences between the profiles of the different experiments is larger, from 20°-70°. If a hole was developed and was getting time to develop further into the clay a steep erosion profile with a cliff shape was developed. This is for example the case for DF1992. If only a small hole was developed, like for DF1984, a cliff shape is not formed (15°-20°). This could however still develop over time if the experiment was proceeded. See Figure 9.10 for some examples of the erosion profiles of clay and sand.

An overview of the maximum slope angles of the final profiles of the Delta Flume experiments on sand and clay is given in Table 9.14. In clay a cliff profile with approximately the same steep cliff profile can develop, but that this is, in contrast with sand, not always the case.

Table 9.14 Maximum slope angle.

Experiment		Angle (°)	Clay condition	Clay type
Sand	DF1981 T1	70	-	-
	DF1981 T2	70	-	-
	DF1996 T01	80	-	-
Clay	DF1984a	30	Unstructured	High sand percentage
	DF1984b	15-20	Unstructured	Low sand percentage
	DF1992Sa	65	Moderately structured	Moderately high sand percentage
	DF1992Sb	75	Structured	Low sand percentage
	DF1992Sc	-	Moderately structured	Moderately high sand percentage
	DF1992Sd	70	Structured	Low sand percentage
	DF1983a	80	Unstructured	High sand percentage
	DF1983b	45	Unstructured	High sand percentage

Size erosion

In the experiment with sand the erosion was developed over the whole width of the flume. In the clay slopes a hole developed which expanded in some experiments on structured and moderately structured clay over the whole width of the flume. In unstructured clay the hole only expanded a little.

9.5 Discussion and conclusions

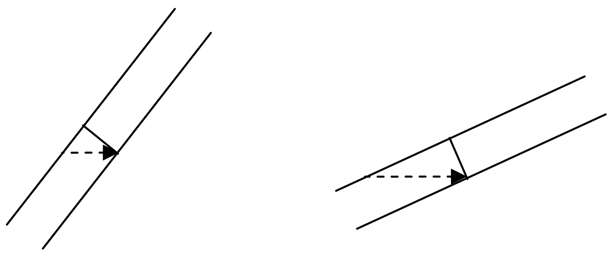
9.5.1 Discussion

- The difference between the determined erosion rates of sand by method A and B are significant, and therefore the differences in erosion rate with clay as well. The erosion determined by method A is larger than determined by method B, but this approaches after some time the horizontal erosion. The erosion determined by method B remains almost constant after one hour. The differences are a result of the different measurement methods and it is not possible to determine the best method. The best comparable erosion depth is probably somewhere between the values determined by these two methods.
  If this is approached by the average of the two methods, this results in a higher erosion rate of sand after two hours of a factor of 2-5 and after six hours of a factor of 2-10. The averaged perpendicular erosion of sand is in that case approximately a factor of 2-10 higher than that of clay.

- After approximately 1 hour the erosion determined by method B is constant. The erosion of sand will slow down over time due to a lower wave load as a result of accretion, but this process will take longer than 1 hour. This process is visible for the horizontal erosion.

- For the determined horizontal erosion the influence of the slope angle is large in comparison to the perpendicular measured erosion. The slope angles of the clay experiments are larger than the initial slope angles of the sand experiments, which results in a relative large horizontal erosion for clay, see figure.

  However, because the differences between the erosion of clay and sand are smaller than measured perpendicular to the slope, the results can be discussed for each clay condition. This can be a valuable contribution which continues with the comparisons made in the former chapter which are also based on the differences in clay condition.



- The development of the horizontal erosion of clay and sand over time is approximately the same, only the amount of erosion of clay is lower. However DF1984 and DF1983 have a different erosion development over time. The erosion rate is increasing over time instead of decreasing as is the case for the other experiments. This is remarkable and could possibly be due to the unstructured clay conditions.

  For unstructured clay the erosion did not progress in the depth, but in length and width of the slope after 6 hours of wave loading, as can be seen in Figure 9.11 and Paragraph 44. This could be due to the presence of the unstructured clay layer with a low sand percentage below the grass sod with a high sand percentage. It can be clearly seen that the erosion depth stops around the transition of the clay layers and expands only in length and width of the layer with a high sand percentage.
  At first the horizontal erosion of sand is approximately a factor of 3-6 higher than of the unstructured clay. However, after 6 hours when the hole does not deepen anymore, the horizontal erosion of the unstructured clay increases and after eight hours the erosion of sand is decreased to a factor of 2-3.5 higher than of the unstructured clay. In this period only erosion in length and width was taking place, not into depth and this has probably distorted the outcome of the horizontal erosion rate of unstructured clay in comparison to sand. However it could be concluded that unstructured clay with a low sand percentage has a very high erosion resistance to wave loading and has much lower erosion rate than sand, because this layer has

an erosion rate of zero in this experiment. Unfortunately only two experiments have been performed with these conditions (DF1983a and b) and therefore this conclusion is uncertain.

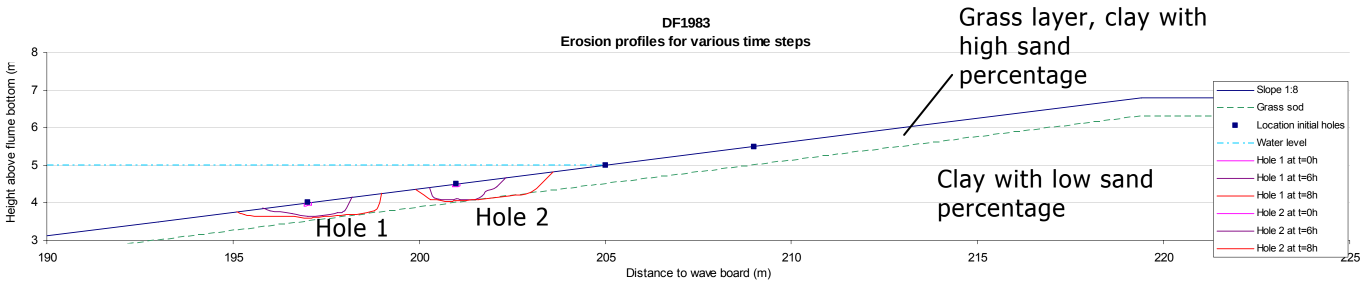


Figure 9.11 Erosion profile of the two holes of DF1983.

- For making a design of a revetment consisting of a thick clay layer, the thickness of the clay layer is normative, because the dike will fail if the clay layer is eroded. The maximum erosion of this clay layer is measured perpendicular to the slope and therefore the perpendicular erosion will in that case be more important than the horizontal erosion. However, in this chapter a comparison is made of the description of the erosion process and therefore both measures are of importance.

- It is unknown if the slope angle is of influence on the erosion development and all sand experiments have steeper slopes than the experiments on clay.

- The erosion of sand decreases over time due to the accretion of in front of the slope. The clay particles are smaller and will not accrete but are suspended in the water and will be washed away by the waves. The erosion process of clay will therefore not decrease. However, unstructured clay in the clay layer has a lower erosion rate, and therefore the erosion of clay can decrease over time if unstructured clay is present, deeper into the clay.

9.5.2 Conclusions

A rough comparison of the erosion of sand with the erosion of clay is made obtain more insight into the behaviour of clay in respect to that of sand. The order of magnitude of the erosion rate, the location of erosion and shape the of the erosion profile are compared to each other.

The erosion of sand is measured in volumes and the values of the perpendicular erosion depth can only be approached. This is done by the use of two different methods. Also the horizontal erosion of sand and clay is compared, because this is besides volumes the most accurate way to measure the erosion of sand.

From all methods appears that the erosion rate of sand is clearly higher than that of clay. However the difference of the perpendicular erosion rates of sand determined by the two methods is significant. The perpendicular erosion of sand determined with method A is approximately 3-10 times higher than of clay, and determined with method B approximately 2-3 times. If these values of the two methods are averaged, the perpendicular erosion of sand is approximately 2-10 times higher than that of clay.

The development of the horizontal erosion of clay and sand over time is approximately the same, but the amount of erosion of clay is lower. Only the unstructured clays of DF1984 and DF1983 have a different erosion development over time. Because the differences in erosion between sand and clay in horizontal direction are clearly smaller than measured perpendicular to the slope, the results can be discussed for each clay condition.

Structured clay erodes approximately 1.5-3 times slower than sand, and moderately structured clay approximately 2-3.5 times. This is just a small difference. For unstructured clay the results are not unambiguous, because in the used experiment the structured clay consisted of two clay types with an upper layer of clay with a high sand percentage and a layer of clay with a low sand percentage below this. The erosion developed in depth till the border of the clay with a low sand percentage is approximately 3-6 times slower than sand. From here on the erosion developed only in length and width not in depth, which resulted, due to the gentle slope of 1:8, in a very large amount of

horizontal erosion compared to the structured and moderately structured clay (2-3.5 times slower than sand).
This makes it impossible to give reliable results for the horizontal erosion of unstructured clay over time, but from these results it could be concluded that unstructured clay with a low sand percentage has a very high erosion resistance to wave loading and has much lower erosion rate than sand.

The location of erosion is different for clay and sand, the erosion of clay is mainly below water level and the erosion of sand above the water level. This could be the result of different erosion processes.

In sand a steep erosion profile develops in 0.1h, which retreats in a horizontal direction. In clay a hole develops which deepens and in time also a cliff can develop, this is however not always the case. The different developed erosion profiles of clay are mutually more different than of sand, which could be a result of the different clay properties of the experiments. It could carefully be concluded that more structured clay has a steeper erosion profile, approximately the same as an erosion profile of sand. Unstructured clay has a more gentle erosion profile.

No conclusions can be drawn on the influence of the clay type (the sand percentage in the clay), the wave height, wave period and initial slope angle. It would be expected that a higher sand percentage in the clay would result in a more similar behaviour as sand.

It can carefully be concluded that the more soil structure has been development into the clay, the more the clay behaves like sand. Unstructured clay has a much lower erosion rate than sand.

However, all these conclusions are based on very limited data, therefore they should be verified by other data or by future experiments.

10 Comparison with erosion in actual storms

First in this chapter an overview is given of observations of outer slope erosion trough history. The observations made during the storm surge of 1953 in The Netherlands are further elaborated and used to compare the results of the experiments on clay and sand with erosion in actual storms.

10.1 Experiences of Huitema (1947)

According to Huitema (1947) an outer slope obtains the shape of Figure 10.1 when damaged by wave loading. If the height of the steep cliff becomes larger than the thickness of the clay layer, the sand core of the dike will be liquefied and washed away by the waves. In this way the crest of the dike will finally collapse. The steeper the slope, the more vulnerable the dike is to collapsing. [Huitema, 1947]

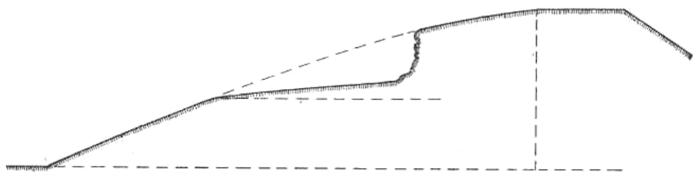


Figure 10.1 Outer slope damaged by waves. [Huitema, 1947] (Dimensions are not given)

10.2 Storm surge disaster of 1962 in Germany

In Germany dikes were originally build in clay. Halfway the 20th century however a sand core with clay cover was applied to fulfil the demands for larger cross-sections. During the storm surge disaster of 16/17 February 1962 large damages occurred on the outer slope of the dike in the zone of the highest tide water level; large parts were washed away from the slope, see Figure 10.2. Research led to the conclusion that these damages occurred at weak places of the cover layer, where the grass layer was damaged by plants and/or animals. [Pohl, 2006]

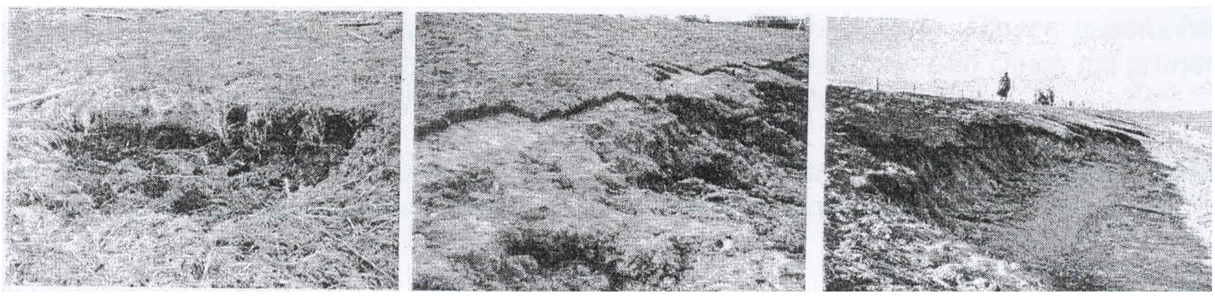


Figure 10.2 Damages on the outer slope at highest tide water level during storm flood of February 1962, Germany. [Pohl, 2006]

At Ulvesbüller Koog many damages developed at the outer slope of the dikes, see Figure 10.3. [Wohlenberg, 1963]

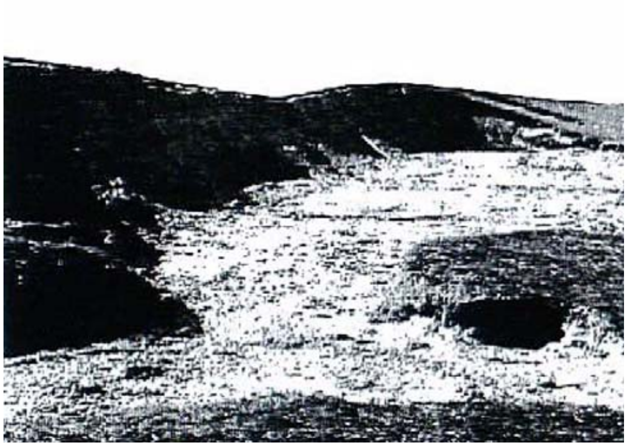


Figure 10.3 Sea dike Ulvesbüll after the storm of February 1962. A part of the outer slope revetment and the dike core is eroded. [Wohlenberg, 1963]

10.3 Vietnam

In Vietnam more recently erosion of the outer slopes of dikes was occurred. See Figure 10.4 for some photographs. Significant waves have a maximum wave height of approximately 2.5m. [Mai Van, 2004]

Erosion of outer slope after storm at Hai Thinh sea dikes in 1998, Nam Dinh province, Vietnam.

Heavy damage of outer slope after storm at Hai Hau sea dikes in 1996, Nam Dinh province, Vietnam.

Severely eroded dike with planted casuarinas trees along HaiHau beach, Nam Dinh province, Vietnam.

Figure 10.4 Outer slope erosion in Vietnam. [Mai Van, 2004]

10.4 Storm surge disaster of 1953 in The Netherlands

During the night of 31 January to 1 February 1953 a westerly gale in combination with spring flood occurred along the Dutch coast which led to the failure of many water defences and the development of many breaches. Many people lost their lives and there was a lot of damage. All the damages took place in approximately 6 hours.

Information about the storm surge disaster is gained by air pictures, enquiries taken from eyewitnesses, polder- and Water Boards, supervisors, sluice guards and other professionals and geotechnical research. It was however not possible to gain all information of all damages.

Most of the observed damages were on the inner slopes and crests of the dikes. Severe damages of the outer slopes, without damage of the inner slope or crest, are very rare. In many cases the inner slope was very severely damaged while the outer slope was completely intact. The damages on the outer slopes were most of the times restricted to erosion of a small strip in the surfzone and local damages to the stone pitching. In some cases the grass cover above the stone revetment had disappeared or contained large holes, sometimes up to the crest of the dikes. In these cases the inner slopes were not damaged. These damages were exceptions, and therefore Rijkswaterstaat did not pay much attention to the collection of information on the damages of the outer slopes. These damages are therefore not treated in the report about the storm surge disaster of 1953 of Rijkswaterstaat (1961). Most, or all, breaches are according to Rijkswaterstaat (1961) the result of other damages than the damages of the outer slopes. The damages of the outer slopes are in the report almost all represented as light damages.

General description of the failure
In general the dike crests were too low. The height of the dikes was at that time determined by the highest known storm water level, the local known maximum wave run-up and a certain safety margin. During this storm the water levels were approximately 1m higher than the known water levels. Failure of the dikes primarily occurred in the form of sliding of the inner slopes of the dikes, sometimes combined with deformations due to the swirling water.

The failure of the dikes started with irregular overtopping of the highest waves. Because the water level was still rising, the overtopping became more, till much water was continually flowing over the inner slopes. The inner slopes were weakening and the development of longitudinal cracks (length 1m-several meters, width 1cm-1dm) were the first signs that sliding was threatening. If the cracks were on time covered with sheets or sand bags, sliding could be prevented, but else the cracks transported the water deeper into the dike and sliding of the inner slopes occurred. Sliding could occur, because water in the ground was flowing just below the grass cover, parallel to the inner slope. This resulted in a large pressure on the soil which made steep slopes slide. When the inner slopes were slide away, the water eroded the dike core material and a breach occurred. This could lead to the development of a flow channel. If a high foreland was present in front of the dike the development of a flow channel could be prevented if the foreland had sufficient height and width. The foreland acted than as a threshold and prevented the water from flowing through the breach.

Damages to the inner slopes were considered to be more representative for the storm surge disaster of 1953, although also many damages to the outer slopes of the water defences occurred. At locations where damage to the outer slope of the dike developed, the stone revetments were washed away. If the underlying clay layers were also eroded, also a part of the sand core of the dike was eroded. In some of these cases Rijkswaterstaat (1961) has made drawings of the eroded profiles or photographs. If the erosion was limited, no detailed information is given. All the detailed information of outer slope erosion of dikes as is present in Rijkswaterstaat (1961) is given in this paragraph, as well as the detailed information of outer slope dune erosion, if given by Rijkswaterstaat to compare this with each other.

10.4.1 Loading conditions

A storm surge takes place in the Netherlands when tilting of the water level by a storm coincides with normal astronomical high water. During the storm of 1953 a north-west directed wind piled up the water in the North Sea which could not escape through The Channel. The water levels along the Dutch coast were approximately 2.5m-3m higher than normal.

Water level

Information about the water levels occurring during the storm surge is collected for different locations along the coastline by Rijkswaterstaat (1961). For the locations of the available outer slope observations given in 10.4.1 and 10.4.3 the water levels of representative locations nearby are used. For the Wieringen Den Oever is representative, for the Hondsbosse Seadefence Petten, for the Brielse Dam Hoek van Holland, for Domburg Westkapelle and for Theodoruspolder Bergen op Zoom. The water levels for the representative locations are given in Table 10.1.

The storm surge is the difference between the actual measured and the predicted astronomical water level variation and in Rijkswaterstaat (1961) determined to be the highest average water level without wave motion, wave run up and 'buistoten' of more than 50 minutes.

Table 10.1 Water levels during the storm surge of 1953 according to Rijkswaterstaat (1961).

Location	Astronomical High Water level according to tide Table above NAP (m ±0.05m)	Storm surge water level 1953 above NAP (m ±0.05m)
Den Oever	0.47	3.70
Petten	-	3.50
Hoek van Holland	0.81	3.85
Westkapelle	1.72	4.35
Bergen op Zoom	1.87	5.17

Wave characteristics

Accurate measurements of the wave heights are not performed during the storm surge. In Rijkswaterstaat (1961) no information is given on this. The wave heights at the Brielse Dam are estimated to be 2-2.5m and at Wieringen and Theodoruspolder 1-1.5m. [prof.drs.ir. J.K.Vrijling]

Duration

Within 2 days the surge level rose from zero to 2.8m and fell down to zero again. The maximum wave loading occurred is assumed to have occurred during a period of approximately 3 hours. It is assumed that a constant maximum observed water level for a period of approximately 3 hours is representative for the maximum loading of the water defence structure. This represents the in reality fluctuating water level. In reality the duration also differed per location along the coast. It is assumed that approximately 90% of the load acted on the water defences in this period. This assumed duration is used in the comparison of the observations in 1953 with the experiments on clay and sand made in this chapter.

See Figure 10.5 for a figure of the water levels during the storm surge disaster of 1953 at Vlissingen and Appendix XV for a more detailed figure and more locations.

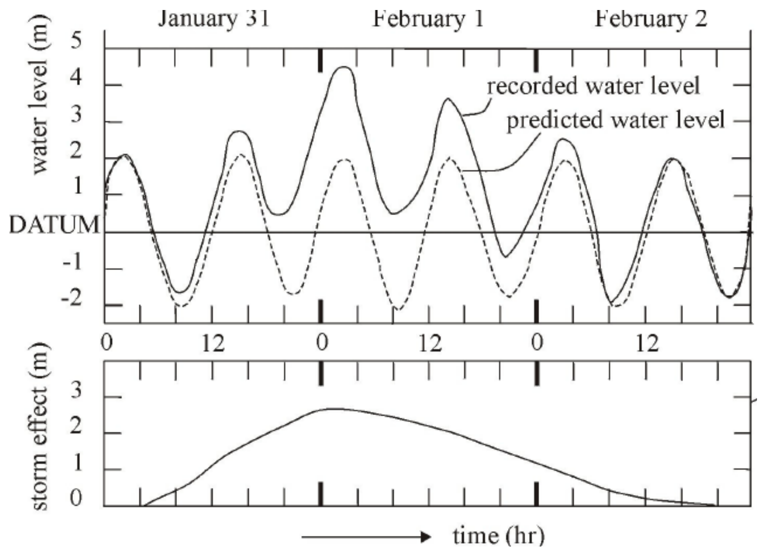


Figure 10.5 Measured and predicted water levels during the storm surge disaster of 1953 at Vlissingen. [van de Graaff, 2004] See Appendix XV for a more detailed figure.

10.4.2 Outer slope erosion of dikes

In this paragraph an overview of the outer slope damages of sea dikes in the Netherlands is given which occurred during the storm surge of 1953 is given. This overview is based on information of the damages and repairs of the water defences in Rijkswaterstaat (1961). In this overview all available observations of outer slope damage given in Rijkwaterstaat (1961) are discussed.

The discussed dikes are located at:

1. The Brielse Maas closure dam
2. Wieringen
3. Afsluitdijk
4. Texel
5. Friesland
6. Theodoruspolder
7. Unknown location

1. The Brielse Maas closure dam

Figure 10.6 Location of the Brielse Maas closure dam.

The stone revetment consisting of light Belgium stones was washed away by the storm surge over a length of 540m. Also the clay cover layer under the stone revetment was locally eroded as well as a part of the sandy core. This erosion continued at several places into the outer slope above the outer berm and at one location even trough the crest of the dam into the inner slope. See Figure 10.7.

After the stone revetments and the clay cover were eroded away, the erosion developed further into the sand core of the dike. The given erosion profile in Figure 10.7 is thus the final developed profile in the sand core, not in the clay. The profile given in Figure 10.7 a profile of the repair work is given and the original profile of the dam is not given. Therefore can on basis of this figure not be concluded with certainty that this is an erosion profile as it will develop in clay.

The measured erosion length and width, and the angles of the erosion profile with respect to horizontal and vertical axes are given in Figure 10.8 and with the other properties given in Table 10.2.

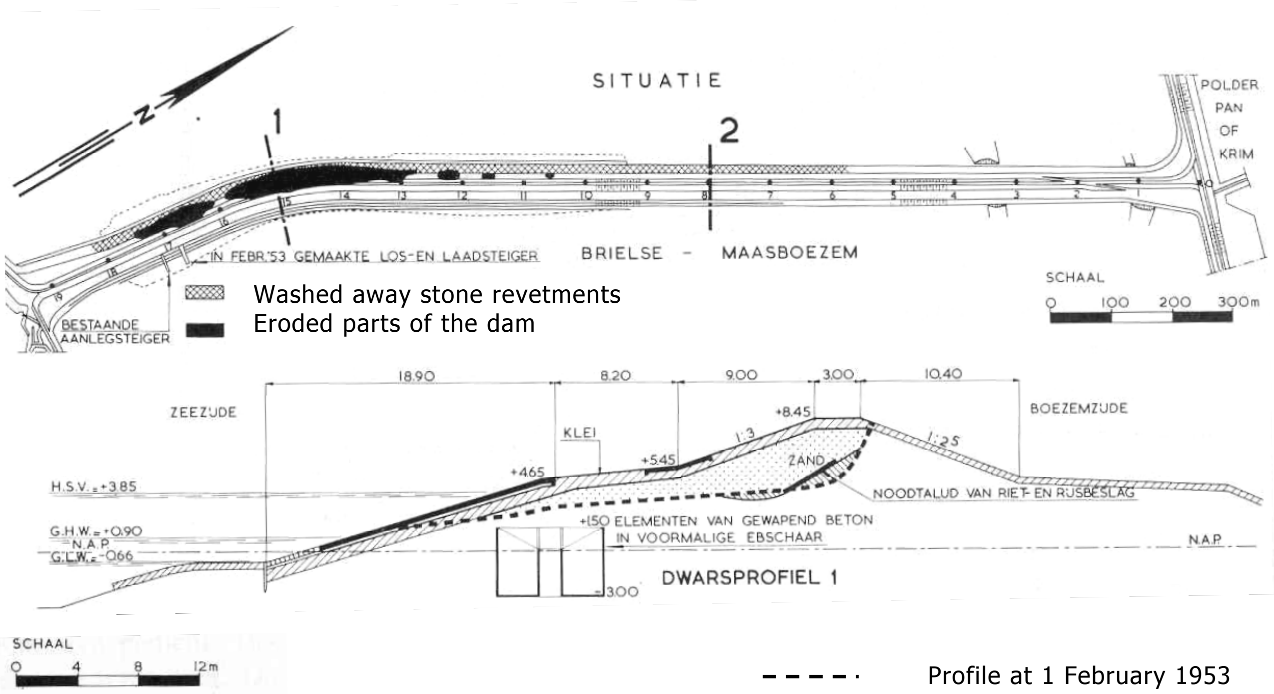


Figure 10.7 The eroded closure dam of the Brielse Maas during the storm surge disaster of 1953, given by the dotted line. The profile of the repaired dam is also given. [Rijkswaterstaat, 1961]

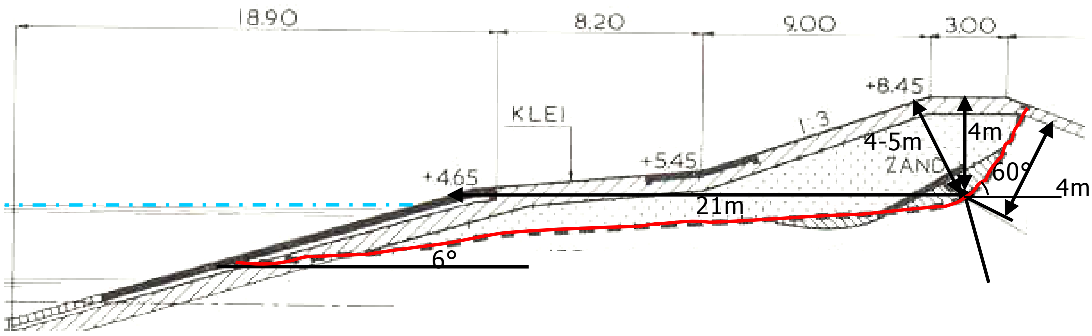


Figure 10.8 Erosion profile of the Brielse Maas closure dam with dimensions.

Table 10.2 Properties erosion profile Brielse Maas closure dam.

Properties erosion profile Brielse Maas closure dam	
Maximum erosion depth	4-5m
Horizontal erosion length	21m
Vertical height cliff	4m
Angle at sea side	6°
Angle cliff with horizontal	60°
Wave height	2-2.5m
Water level	NAP+3.85m

Conclusions

- Damage to the outer slope was developed over a length of 540m and at several locations into the core of the dam, which consisted of sand.
- A clear cliff shape can be recognized with a steep upper slope.
- The erosion developed not only in the clay layer but also for a large part into the sand core of the dike.
- The erosion developed partly below and for a large part above the storm surge water level.

2. Wieringen

The dike on the Northern side of Wieringen suffered from erosion of the outer slope, see Figure 10.9. The photograph shows the situation after a first repair with sheets and sand bags. The final repair was performed with boulder clay covered by organic soil. The developed erosion profile with a steep slope is still clearly visible in the photograph. Measurements on this erosion profile were however not performed.

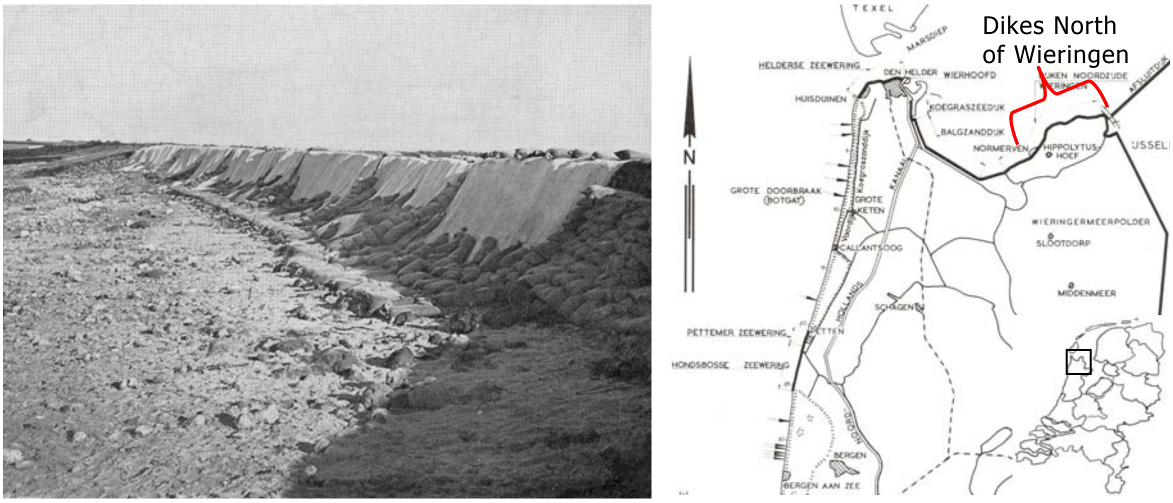


Figure 10.9 Left: Erosion profile of the Northern dikes of Wieringen, the steep slope is clearly visible. Right: Location of the Northern dikes of Wieringen in the province of North Holland. [Rijkswaterstaat, 1961]

Conclusions

- A cliff profile developed into the dike, with a very steep slope.
- The steepness of the cliff is comparable to that of the Brielse Dam, see Figure 10.9.
- Also a large part of the core of the dike was eroded. It is however unknown if the core consisted of sand or clay.
- The following properties can be estimated, see Table 10.3.

Table 10.3 Estimated properties erosion profile Wieringen.

Estimated properties erosion profile Wieringen	
Angle cliff with horizontal	60°
Wave height	1-1.5m
Water level	NAP+3.70m

3. Afsluitdijk

The stone revetment at the sea side of the Afsluitdijk was eroded, but the dam itself, which is consisting of boulder clay, did not fail. Unfortunately, no photographs or measurements of the erosion are given in Rijkswaterstaat (1961). The heaviest damage occurred at the Belgium stone revetments below NAP+4.00m.

The water level during the storm surge is related to measurements at Den Oever (NAP+3.70m) and Kornwerderzand (NAP+3.73m), and would have been approximately NAP+3.72m. The wave height was approximately 1-1.5m.

Conclusions

- The stone revetment was damaged, the boulder clay not significantly.
- The heaviest damage took place just above the storm surge water level.
- No quantitative information of the erosion is available.

4. Texel

Also on Texel many heavy damages to the outer slope of the sea dikes were developed, to both the stone revetments and the clay layer beneath it.

In polder De Eendragt the sea dike was damaged in 116 places and at some locations the whole width of the crest was eroded. The bad condition of the in 1931 applied clay cover appeared to be the cause. The thickness of the applied clay was too thin and the quality low. A breach developed, but in Rijkswaterstaat (1961) it is not given if the beach started from the inner slope or the outer slope. On the bottom of the breach stiff clay was present, and therefore deepening by scour did not take place.

In Waterschap De Dertig Gemeenschappelijke Polders the concrete slabs and stones of the dike were damaged and washed away, as well as the clay layer beneath it. On several places the core of the dike was disappeared.

Unfortunately however, no photographs, cross-sections or measurements of the developed erosion on these locations are given in Rijkswaterstaat (1961).

Conclusions
- Stone revetments and clay layers were eroded. At several locations the core of the dike as well.
- No quantitative information of the erosion is available.

5. Friesland

The sea dikes at the mainland of Friesland suffered from multiple damages of the stone revetments at the outer slopes of the dikes. However, no erosion profiles or measurements are given in Rijkswaterstaat (1961). It is unknown if the underlying clay layers were also eroded.

6. Theodoruspolder

The Theodoruspolder is located at the coast at the eastern side of the Oosterschelde. Several damages and three breaches were developed to the dike between the Oosterschelde and the Theodoruspolder. No detailed information on the damages of the outer slopes of the dike is given in Rijkswaterstaat (1961), only a cross section of one profile, profile 1 given in Figure 10.10. The erosion developed into the sand core of the dike.

A breach was not developed at this location, but the erosion developed far into the inner slope. Therefore the erosion developed for the greatest part into the core of sand, and the shape of the erosion profile of the outer slope could be influenced by the continuing erosion into the inner slope. This could have caused the development of a different erosion profile than for the start of the outer slope erosion.

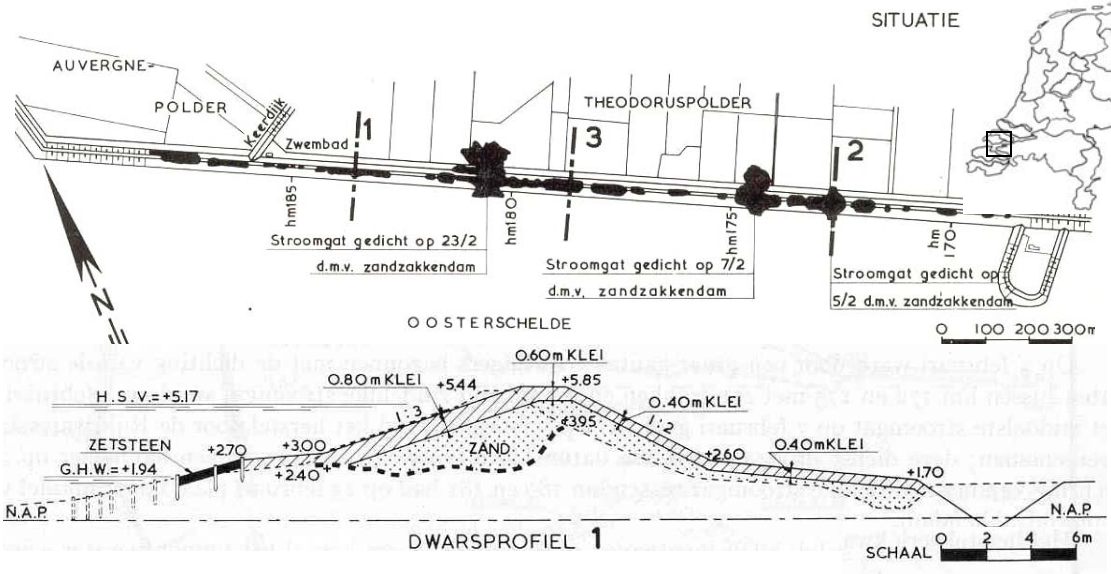


Figure 10.10 Failure of the outer slope of the dike along the Oosterschelde at Theodoruspolder. The dotted line gives the original dike profile before the storm. [Rijkswaterstaat, 1961]

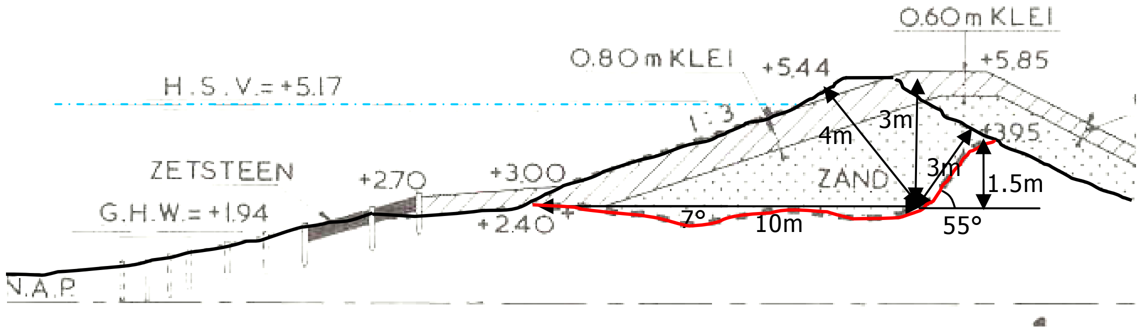


Figure 10.11 Erosion profile of the Theodoruspolder with dimensions.

Table 10.4 Properties erosion profile Theodoruspolder.

Properties erosion profile Theodoruspolder	
Maximum erosion depth	4m
Horizontal erosion length	10m
Vertical height cliff	3m
Angle at sea side	-7°
Angle cliff with horizontal	55°
Wave height	1-1.5m
Water level	NAP+5.17m

Conclusions

- A clear cliff shape can be recognized.
- The erosion developed mostly below the storm surge water level.
- The erosion developed far into the inner slope and thus also a large part of the core of the dike was eroded.

7. Unknown location

In this photograph of Figure 10.12 is visible that an almost vertical cliff was developed far into the dike. A Muralt wall is present on top of the dike.

Figure 10.12 A dike which just did not fail, during the storm surge disaster of 1953. The cliff erosion which developed till the core of the dike can clearly be seen. [www.deltawerken.com]

Conclusions
- A clear cliff profile developed. The developed cliff seems very steep, almost vertical.
- The erosion developed far into the dike. It is unknown if the core of the dike was consisted of sand or clay.

10.4.3 Erosion of dunes

All the available information given in Rijkswaterstaat (1961) on outer slope dune erosion is listed in this paragraph, to give an addition to the limited available data of the outer slope erosion of dikes. In this way more real erosion data is available to make a comparison with the experiment data of the pervious chapters.

The discussed dunes are located at:
I. Connection to the Hondsbosse Sea Defence
II. Coast North of the Brielse Maas closure dam
III. Coast South of the Brielse Maas closure dam
IV. Domburg

V. Connection to the Hondsbosse Sea Defence

The dunes at the connection south to the Hondsbosse sea defence were eroded over a length of 20-25m, because the protection at the toe of the dune was not well connected to the stone revetment of the sea defence. From the photograph of Figure 10.13 appears that a steep erosion profile was developed in the dune. This profile is similar to the profiles of clay erosion, so therefore it seems that the developed erosion profiles after a storm at the same for sand and clay.

Figure 10.13 Dune erosion at the connection to the Hondsbosse sea defence at Kamperduin. [Rijkswaterstaat, 1961]

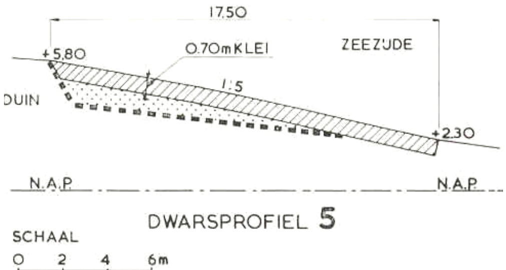
Conclusions

- A steep erosion profile was developed with a cliff shape.
- Based on the photograph the following measurements of the erosion can be estimated, see Table 10.5:

Table 10.5 Estimated properties erosion profile of the dune at the connection to the Hondsbosse sea defence. The measured erosion is based on the photograph of Figure 10.13.

Estimated properties erosion profile dune at connection Hondsbosse sea defence	
Maximum erosion depth	approx. 9m ±5m
Horizontal erosion length	approx. 25m ±5m
Vertical height cliff	approx. 9m ±5m
Angle cliff with horizontal	45°
Wave height	2-2.5m
Water level	NAP+3.50m

VI. Coast North of the Brielse Maas closure dam



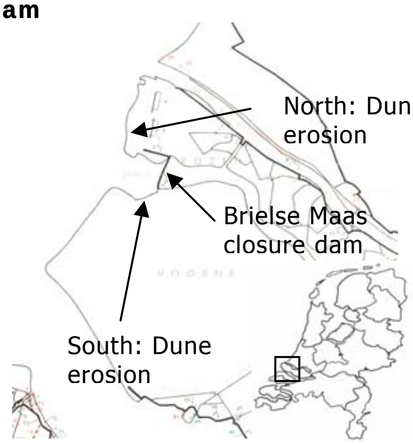


Figure 10.14 and Figure 10.15 Left: Repair profile of the dunes North of the Brielse Maas closure dam, with the erosion profile given by the dotted line. [Rijkswaterstaat, 1961]
Right: The locations of outer slope erosion around the Brielse Maas closure dam.

The dunes eroded while developing a clearly visible cliff profile. After the storm surge the dune was considered not to be wide enough and therefore a clay cover was applied during the repair works. Unfortunately, in the repair profile of Rijkswaterstaat (1961) as given in Figure 10.14, only the erosion profile is given, not the original profile before the storm. Therefore it is not possible to determine the erosion. Sand erodes rapidly in short time (see Chapter 8) and because a clay revetment was applied for extra strength during the repair works, it is expected that the erosion was significant. The estimated erosion measurements as are given in Figure 10.16 will probably underestimate the occurred erosion and are therefore considered not to be representative for dune erosion. Only the steepness of the cliff is considered to be representative, see Table 10.6.

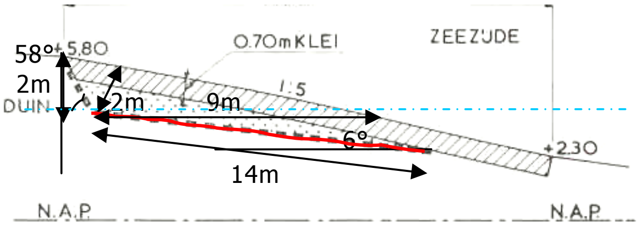


Figure 10.16 Erosion profile dunes North of the Brielse Maas closure dam with dimensions.

Table 10.6 Properties erosion profile North of the Brielse Maas closure dam.

Properties erosion profile North of the Brielse Maas closure dam	
Maximum erosion depth	-
Horizontal erosion length	-
Vertical height cliff	-
Angle at sea side	-
Angle cliff with horizontal	58°
Wave height	2-2.5m
Water level	NAP+3.85m

Conclusions

- A very clear cliff profile was developed.
- It is unknown if a clay layer was also present before the storm surge disaster.
- The erosion took place in a dune.
- The angle of the cliff is located just around the storm surge water level and therefore the erosion took place partly above and partly below the storm surge water level.

VII. Coast South of the Brielse Maas closure dam

Just south of the Brielse Maas closure dam, the stone revetment was damaged and the dune above the revetment was eroded. This damage was repaired by making a dike profile and applying a clay cover layer on top of the dune, see Figure 10.17. In the repair profile also the original profile and the erosion profile are given. This made it possible to measure the occurred erosion, see Figure 10.18 and Table 10.7.

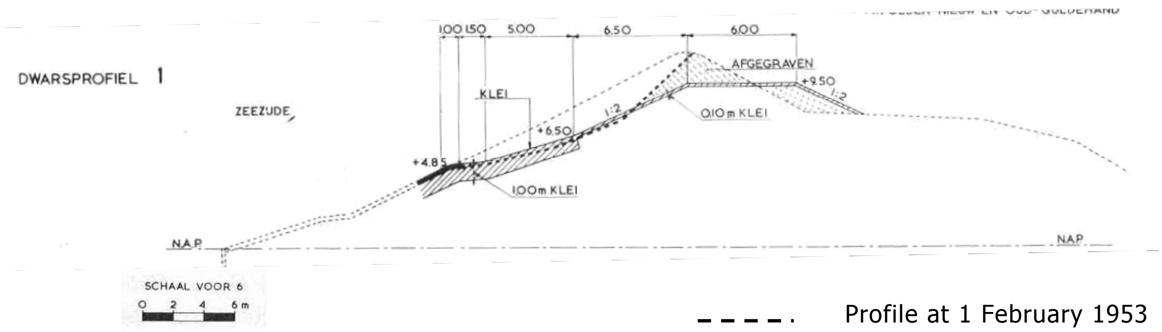


Figure 10.17 Water defence on Voorne-Putten, just South of the Brielse Maas closure dam. [Rijkswaterstaat, 1961]

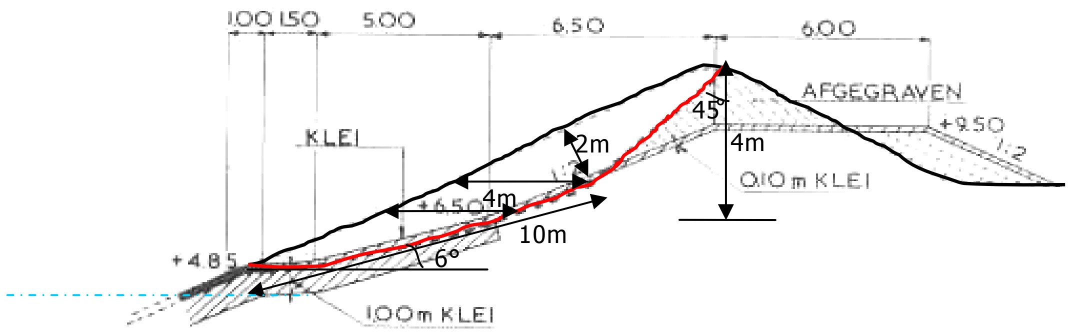


Figure 10.18 Erosion profile of a dune South of the Brielse Maas closure dam with dimensions.

Table 10.7 Properties erosion profile dune South of the Brielse Maas closure dam.

Properties erosion profile dune South of the Brielse Maas closure dam	
Maximum erosion depth	2m
Horizontal erosion length	4m
Vertical height cliff	4m
Angle at sea side	6°
Angle cliff with horizontal	45°
Wave height	2-2.5m
Water level	NAP+3.85m

Conclusions

- A parabolic erosion profile developed as is to be expected for dunes, not a cliff shape.
- The erosion took place above the storm surge water level.

VIII. Domburg

A dune at Domburg eroded developing a cliff profile with two steps as given in Figure 10.19. Unfortunately, in the repair profile of Rijkswaterstaat (1961) as given in Figure 10.19, only the erosion profile is given, not the original profile before the storm. Therefore it is not possible to measure the occurred erosion. Similar as for the dune North of the Brielse Maas closure dam (number II.), it is therefore not possible to determine the erosion. Sand erodes rapidly in short time (see Chapter 8) and because a revetment was applied for extra strength during the repair works, it is expected that the erosion was significantly larger than is measured in Figure 10.20. The measurements will probably underestimate the occurred erosion and are therefore considered not considered to be representative for dune erosion. It is however unknown if a revetment was also present before the storm surge.

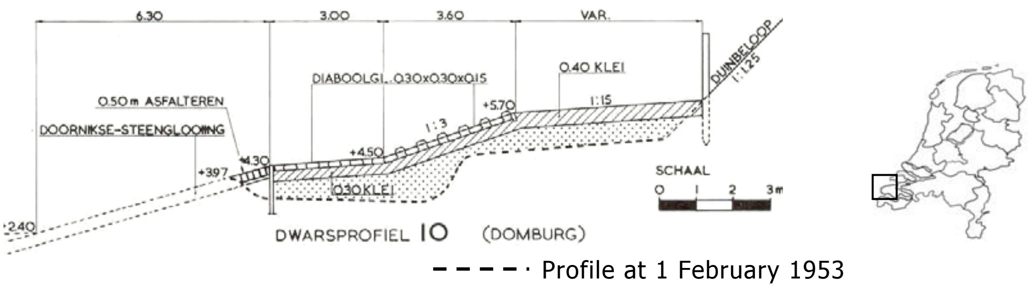


Figure 10.19 Erosion profile of outer slope damage of a dune near Domburg. [Rijkswaterstaat, 1961]

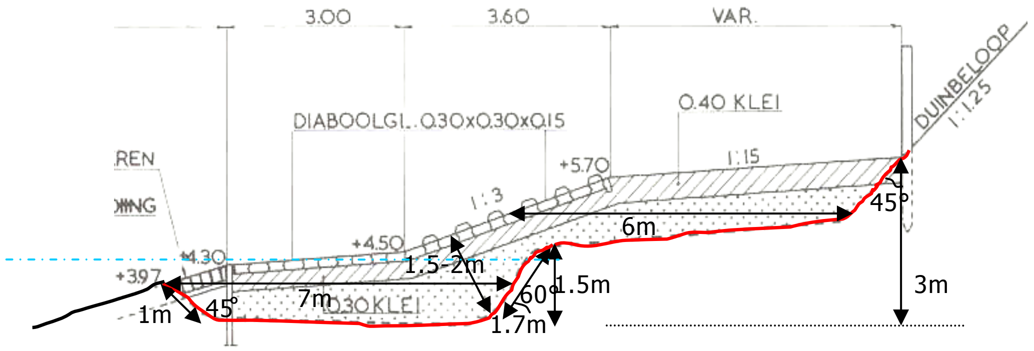


Figure 10.20 Erosion profile of a dune at Domburg with dimensions.

Table 10.8 Properties erosion profile dune at Domburg.

Properties erosion profile Domburg	
Maximum erosion depth	-
Horizontal erosion length	7m
Vertical height cliff	-
Angle at sea side	-45°
Angle cliff with horizontal	60°+45°
Wave height	2-2.5m
Water level	NAP+4.35m

Conclusions

- The position and shape of the initial profile is unknown and therefore it is not possible to determine the maximum erosion depth and the depth of the cliff.
- Two cliff shapes can be recognized, a stairs shape, and not a typical parabolic erosion profile for dunes has been developed. The reason of this development is not given in Rijkswaterstaat (1961), but could be due to a change in water level. During the storm the water level rose and fell because of the tide, see Figure 10.5, this has resulted in a rise and fall of the water load along the slope which could have caused the development of two cliffs.
- There is a negative angle of with the horizontal axe, a hole developed into the sand core of the dike. It is not clear if this is a result of deepening of the erosion into the sand core under the stone revetment or clay layer or due to other unknown influencing factors.
- The erosion took for the larger part place in the area above the storm surge water level.

10.4.4 Overview and conclusions of outer slope damage

Erosion development

- In the cases of dike erosion also the sand core of the dike was eroded.
- At locations where the stone revetments on the outer slope were heavily damaged, the under lying clay layers were also eroded, sometimes the erosion developed even far into the core of the dike which consisted of sand.
- The material properties of the clay of the different locations are not given in Rijkswaterstaat (1961) and are therefore unknown.
- The water levels occurring during the storm surge are measured, but at that time this was not possible for the wave heights. Therefore the wave heights are estimated later on.

Location erosion
1953

- Dikes: For the Brielse Maas closure dam the erosion was developed above and below the storm water level. For the Theodoruspolder the erosion was developed below the storm water level.
- Dunes: The dune North of the Brielse Maas closure dam eroded above and below the storm water level, with the location of the storm water level at the level of the angle of the cliff. At the other two dune profiles the erosion took place above the storm water level.

Shape erosion profile

- The erosion profiles of in clay and sand look similar based on the observations and photographs of Rijkswaterstaat (1961). In most cases a steep erosion profile was developed with a cliff shape.

An overview of all measurements obtained from the available data of locations with outer slope damage during the storm surge of 1953 is given in Table 10.9. These measurements are used in the next paragraph to compare the erosion development of real failure with the results of the Delta Flume experiments.

Table 10.9 Overview of all measurements of the available locations with outer slope damage of dikes and dunes in the storm surge of 1953.

Properties erosion profile	Dikes			Dunes			
	1. Brielse Maas closure dam	2. Wieringen	6. Theodorus polder	I. Connection to Hondsbosse Sea Defence	II. North of Brielse Maas closure dam	III. South of Brielse Maas closure dam	IV. Domburg
Source	profile	photograph	profile	photograph	profile	profile	profile
Maximum erosion depth*	4-5m	Unknown	4m	approx.9m	Unknown	2m	Unknown
Horizontal erosion length	21m	Unknown	10m	approx.25m	Unknown	4m	7m
Vertical height cliff	4m	Unknown	3m	approx.9m	Unknown	4m	Unknown
Angle at sea side	6°	Unknown	-7°	Unknown	Unknown	6°	-45°
Angle cliff with horizontal**	60°	approx 60°	55°	45°	58°	45°	60°+45°
Shape	Cliff	Cliff	Cliff	Cliff	Cliff	Parabolic	Cliff, 2 steps
Material	Clay+ sand	Clay+sand	Clay+sand	Sand	Sand	Sand	Sand
Wave height	2-2.5m	1-1.5m	1-1.5m	2-2.5m	2-2.5m	2-2.5m	2-2.5m
Water level	NAP+ 3.85m	NAP+ 3.85m	NAP+5.17m	NAP+3.50m	NAP+ 3.85m	NAP+ 3.85m	NAP+4.35m

* Perpendicular to the slope
**Huitema (1947) and the unknown location give an almost vertical cliff angle.

10.5 Comparison of the experiments with actual storms

In this paragraph a rough comparison is made between the damages on outer slopes observed in actual storms and the results of the Delta Flume experiments on clay and sand.

The comparison is again based on the three subjects:
- Erosion development
- Location of erosion
- Shape of the erosion profiles

For a comparison of the erosion development and the location of erosion, the obtained data of the damages of the storm surge of 1953 is used, as is given in 10.4. For the shape of erosion all observations of outer slope erosion given in this chapter is used.

No records of the material properties of the dikes in 1953 were given by Rijkswaterstaat (1961) and therefore a comparison of the material properties is not possible. However it can be assumed that the clay in the dikes was structured at that time, because usually the clay layers that were applied on a core of sand were approximately 0.5-1m thick.

10.5.1 Erosion development

The observed damages of the outer slopes as given in this chapter are final erosion profiles. In contrast to the experiments, the erosion development over time is not known. The estimated duration of the storm and the final erosion profiles are used to make a rough comparison between the results of the experiments and the erosion occurred during the storm surge of 1953.

A comparison of the determined maximum erosion depth perpendicular to the slope as well as horizontally measured to the slope is made between the results of the experiments and the observation in 1953. The data is combined in the following graphs. The erosion measured perpendicular to the slope is given in the graph of Figure 10.21 and the horizontal erosion in the graph of Figure 10.22. See also Appendix XVI.

A subdivision of dikes and dunes is made, but in 1953 the dikes were eroded till in the sand core of the dikes.

Assumptions
- It is assumed that the largest part of the erosion of 1953 took place in approximately two to four hours.
- The wave conditions of 1953 are generally larger (H_s=2-2.5m) than for the experiments (H_s=1-1.5m). Therefore actually only Theodoruspolder (H_s=1-1.5m) is comparable with the experiment results. Because that leaves insufficient data for a reliable comparison. This is discussed in 10.6.

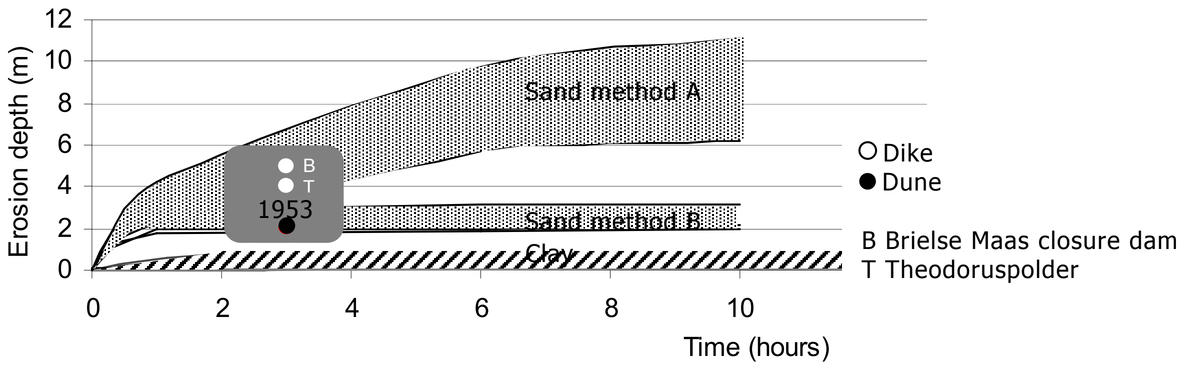


Figure 10.21 Maximum erosion of the experiments on sand and on clay and the observed erosion after the storm surge of 1953, measured perpendicular to the slope.

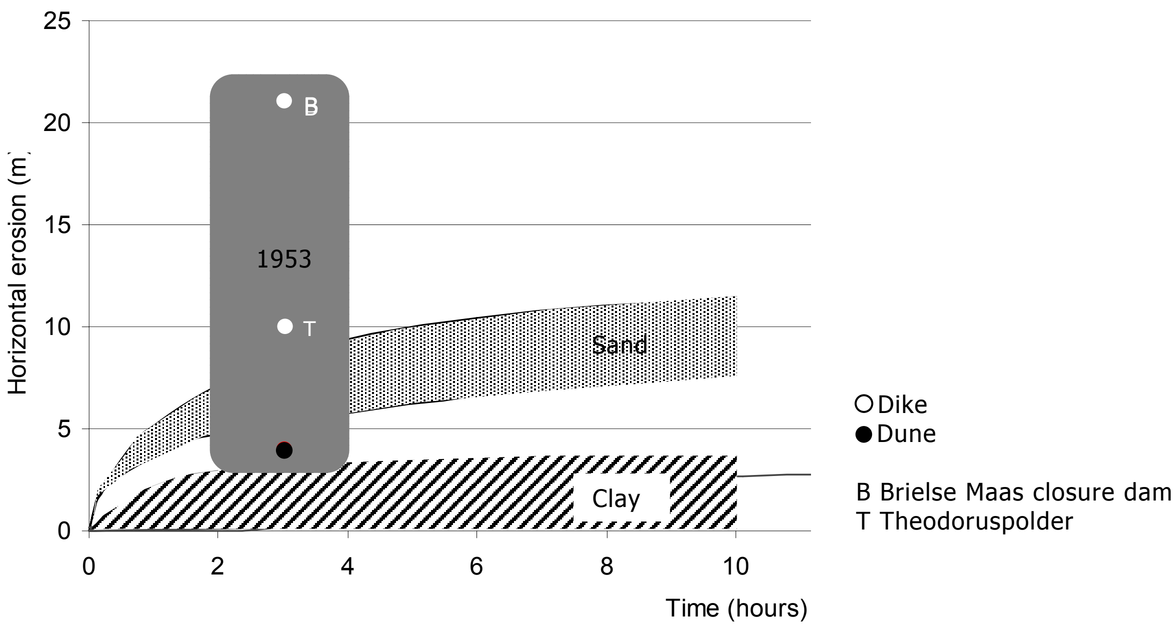


Figure 10.22 Maximum erosion of the experiments on sand and on clay and the observed erosion after the storm surge of 1953, measured horizontally.

Results
General
- The erosion of the dunes is remarkably lower than of the dikes.
- Horizontally measured there is a larger spread in the results of 1953 than perpendicularly measured.
- The erosion of the Brielse Maas closure dam is larger than the erosion of the Theodoruspolder. This is what is expected based on the fact that the wave height at the Brielse Maas closure dam (H_s=2-2.5m) was larger than at the Theodoruspolder (H_s=1-1.5m).
- A higher wave load is expected to result in a higher erosion rate. For the observations of 1953 this was not the case however. Theodoruspolder had a lower wave load than the other locations, but did not have the lowest amount of erosion.

Erosion perpendicular to the slope
- Measured perpendicular to the slope, the results of 1953 fall within the results of the experiments on sand and are therefore larger than the erosion development of the clay experiments:
- The erosion of the dikes falls within the range of erosion development of sand measured with method A.
- The erosion of the dune falls within the range of erosion development of sand measured with method B.
- The perpendicularly measured erosion of the dikes and dune of 1953 is approximately similar to the erosion of sand in the experiments. The dikes however were eroded till their sand cores.

Horizontal erosion
- The horizontally measured erosion of the dikes in 1953 is significantly larger than of the experiments on sand and clay.
- The horizontally measured erosion of the dunes in 1953 is within the experiment results on sand and clay.

On basis of these results for the dikes, it can be concluded that the experiments give safer results than observed in reality.

10.5.2 Location of erosion

The Delta Flume experiments showed that the erosion of clay starts by the development of a hole below the water level. The hole can develop and extent till above the water level, if the erosion process has sufficient time to continue. The erosion of sand initiates and develops above the water level.

For an analysis of the location of erosion in actual storms, the observations of outer slope erosion in the storm surge of 1953 is used:

Dikes
From the observations made in the storm surge of 1953 the erosion of Brielse Maas closure dam occurred above and below the storm surge water level, and at the Theodoruspolder below this level. For both cases the erosion developed into the core of the dike, which consisted of sand.

Dunes
The dunes eroded above the storm water level, except the dune north of the Brielse Maas closure dam which eroded above and below the storm water level. In this profile a cliff was developed with the angle of the cliff at the height of the storm water level.

Conclusion
Both the experiments as the damages of 1953 show, that the erosion of sand is generally located above the water level and the erosion of clay starts below the water level and can develop further till above the water level.

Discussion
From the experiments as well as the damages of 1953 can be concluded that the erosion of sand is generally located above the water level and the erosion of clay starts below the water level and can develop further till above the water level. Apparently clay and sand have different erosion mechanisms. The erosion mechanisms of dunes [Nishi, 1996] are discussed in 9.2.

10.5.3 Shape erosion profile

Table 10.10 Overview cliff angles for all situations.

Situation	Angle cliff with horizontal
Experiments Clay	15°-80°
Experiments Sand	50°-80°
1953 Dikes	55°-60°
1953 Dunes	45°-60°

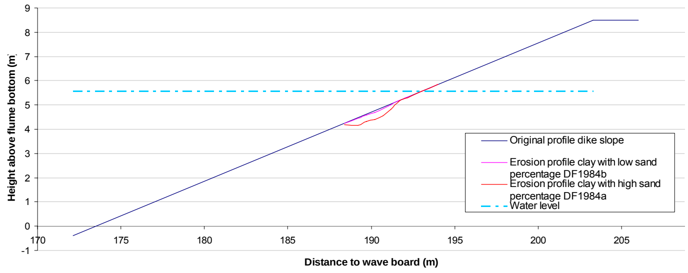
In sand a steep erosion profile develops in the Delta Flume. In clay a cliff profile with approximately the same steep cliff profile can develop, but the differences between the profiles of the different experiments is larger. If only a small hole was developed (like for unstructured clay), the cliff shape is less clear. However this could have been developed if the experiment was continued. If a hole was developed in the clay and was getting time to develop further into the dike, a steep erosion profile with a cliff shape developed. The development of a cliff shape could thus depend on the material properties of the clay in combination with the duration of the experiment.

In the recorded cases of 1953 in all dikes cliff shapes were developed, also in the dunes except of a dune south of the Brielse Maas closure dam where a parabolic shape developed. Thus it seems that the developed erosion profiles after a storm are the same for sand and clay. However, the dikes profiles developed all into the core of the dike which consisted of sand. Therefore the final profile developed into the sand, which could be the reason for the developed steep profiles with a slope angle so close to each other while during the experiments on clay the differences between the different slope angles were much larger.

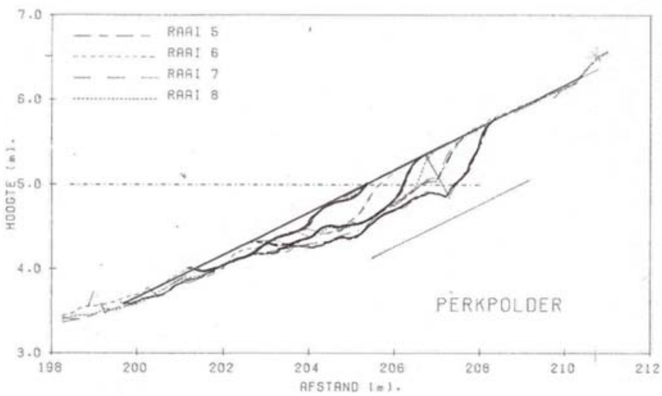
Table 10.10 gives an overview of the range of slope angles of the developed cliffs with a horizontal and Figure 10.23 on overview of some examples of erosion profiles.

Examples of erosion profiles

Experiments on Clay

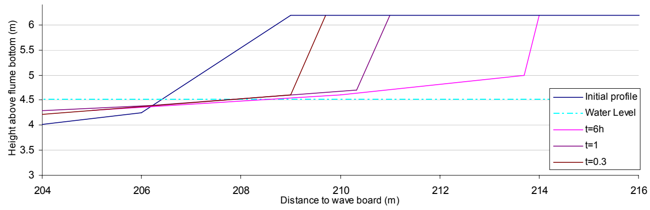


DF1984a and DF1984b: Unstructured clay with high (a) and low (b) sand percentage, after 4.67h.



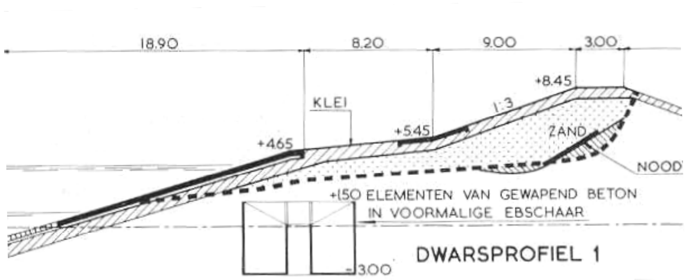
DF1992Sa: Moderately structured clay with moderately high sand percentage, profiles at 0h, 0.4h, 1.2h and 2h.

Experiments on Sand



DF2006SandT01: Profiles at 0h, 0.3h, 1h and 6h.

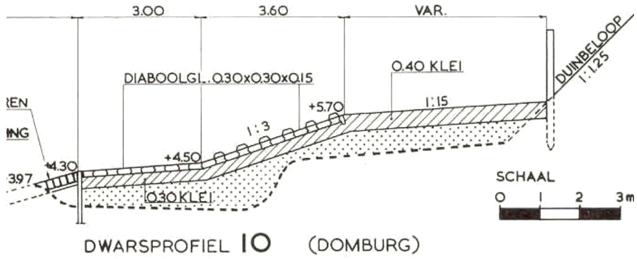
1953 Dike



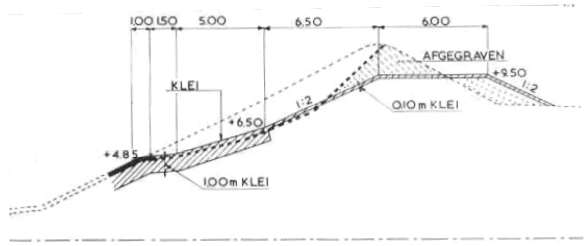
Brielse Maas closure dam: Core of sand

Wieringen

1953 Dune



Domburg



Coast South of the Brielse Maas closure dam

Figure 10.23 Some examples of erosion development of the experiments on clay and sand and the observations made after the storm surge of 1953. In clay (left) the erosion starts below the water level and can develop until above the water level. In sand (right) the erosion takes mainly place above the water level.

10.6 Discussion and conclusions

10.6.1 Discussion

- Only Theodoruspolder has a wave height of 1-1.5m, which is similar to the used Delta Flume experiments. Therefore a good comparison can only be made with this location. The other locations have a wave load of 2-2.5m which is higher than of the Delta Flume experiments. A higher wave load is expected to result in a higher erosion rate. However, the amounts of erosion of the locations with a wave load of 2-2.5m are not all higher than of the Theodoruspolder. This could probably be the result of different breaker types on the slope.

- Remarkably the erosion of the dunes is lower than of the dikes.
  It was not expected that the dune erosion was smaller than the erosion of the dikes, because a revetments was present on the dike and from the experiments became clear that the erosion of sand is significantly larger than of clay. The erosion of the dikes in 1953 developed into the sand core of the dike. This is thus a combination of clay and sand erosion. It was therefore expected that the erosion of the dikes in 1953 would be higher than the experiments on clay, but still be lower than the Delta Flume experiments on sand, because of the revetment present on the dike. This could be a result of material properties. However, the difference is more likely to be found in differences in loading between the experiments and actual storms:

  - Weak locations along the slope. The erosion of a clay layer will start at the weakest location on a dike slope. On a real dike more weak locations will be present than in the small width of the Delta Flume.

  - During a storm sand and debris are present in the water, which will influence the erosion process. The particles could increase the erosion development if they are smashed onto the dike slope by the waves. Scouring of the sand and debris on the slope can initiate and increase erosion. In the Delta Flume clean, fresh water is used.
    The sand and debris could also decrease the erosion development if it is deposited on and in front of the slope (in front of the developed cliff, on the bottom of the erosion hole). The material can form in this way a kind of revetment, which can protect the small clay particles from eroding. The wave loading will then act on the deposited material and not on the clay slope itself. The process could be similar to the erosion process of sand, where eroded sand is accreted in front of the dune front with an equilibrium profile as a result. It is however unknown if this equilibrium will develop during the storm of if the storm is stopped before this is reached. If the return current is strong, the material will however be transported into the sea and not deposit in front of the slope.

  - During the experiments in the Delta Flume fresh water is used. Sea water is salt. The data of the storm surge of 1953 thus contains erosion data of loading with salt water. This difference in salt content could have an influence on the erosion process of clay. Clay consists of minerals such as quartz, iron, aluminium compounds and chalk, which could make chemical bonds with the salt ions in the water. The main salt ions are chloride and sodium.
    The salt content in the water is also of influence on vegetation. Vegetation which uses fresh water will reduce in strength when covered with salt water.

  - The influence of oblique incoming waves. In the Delta Flume the waves are approaching the dike perpendicular, in actual storm surges the waves will approach the dike often under an angle. This could result in a different wave load in the slope. Also the wave characteristics and breaker types in actual storms can be different than obtained in the Delta Flume.

- Both the experiments and the damages of 1953 show that the erosion of sand is generally located above the water level and the erosion of clay starts below the water level and can develop further till above the water level. Apparently sand and clay have different erosion mechanisms. The erosion mechanisms of dunes [Nishi, 1996] are discussed in 9.2. Further research is necessary to the erosion mechanisms of a clay slope on a dike.

- It is unknown if a compaction obtained in the Delta Flume is similar to the compaction of clay in real dikes, because in real dikes the used equipment will be bigger. Because the relatively small test slopes built in the Delta Flume are relatively small, the compaction could be obtained with more care. The compaction is of influence of the extent of soil structure development of the clay and thus the condition of unstructured clay.

- A part of these remarks can only be investigated if an experiment on a real dike instead of in the flume would be performed.

10.6.2 Conclusions

On basis of the available results of the storm surge of 1953 can be concluded, that for dikes the experiments give less erosion than observed in 1953.

Observations during the storm surge disaster of 1953 in The Netherlands give for dikes horizontally measured more erosion than observed in the experiments on sand. Measured perpendicular to the slope, the observed amount of erosion dikes and dunes is approximately similar to the experiments on sand. The erosion of all dikes was developed through the clay layer into the sand cores of the dikes, but unexpectedly the observed erosion of the dikes was larger than of the dunes. The erosion data of 1953 are however not accurate and the wave heights were generally higher than used in the experiments.

Several factors could be of influence on the erosion process of clay in dikes and the differences in results of the experiments and observed real damage (See Discussion (10.6.1) for an explanation):

- Weak locations are present in a dike.
- During a storm sand and debris will be present in the water which will have an influence on the erosion process. These particles present can increase the erosion process of the clay, by inducing scour.
- Sea water is salt, though in the Delta Flume experiments fresh water was used.
- The wave conditions and breaker types at the locations from the storm surge in 1953 could be different than used in the Delta Flume.

Both the experiments and the real situation in 1953 show that the erosion of sand is generally located above the water level and the erosion of clay start below the water level and can develop further till above the water level. Apparently sand and clay have different erosion mechanisms.

The erosion profile of clay has a steep slope, the shape of a cliff, if the initial developed hole has sufficient time to expand. In sand a steep slope develops, which retreats in horizontal direction. The cliffs developed in clay are mutually more different and sometimes more gentle than in sand.
The developed erosion profiles after a storm are the same for sand and clay. However the erosion of the dikes all developed into the core of the dike, which consisted of sand.

11 Modelling the erosion of clay under wave loading, based on the experiments

In this chapter a model is developed to determine the erosion depth of clay under wave loading over time, based on the conclusions made in the former chapters.

11.1 Start of a theoretical model

From analyzing the different large scale experiments, appeared that a distinction between the experiment results based on the amount of soil structure development is clearly possible. The soil structure development is thus of large influence on the erosion resistance of clay under wave loading.

To make a theoretical model formulas have to be made or already exist to describe the relation between the different influencing parameters. For the erosion of clay under the influence of waves is still too much unknown to make a theoretical model. However, the first modelling steps with an analysis and schematisation of the influencing parameters can be given.

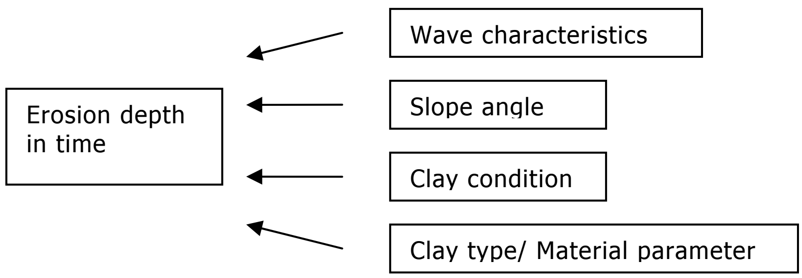
The erosion of clay over time under influence of waves is to be measured by a certain maximum erosion depth. The following parameters are of influence on modelling of the erosion depth (d):

- Wave characteristics (H/T) or combined in Iribarren parameter (ξ)
- Slope angle (α)
- Clay condition: amount of soil structure development
- Clay type/ material parameter/depending for example at the sand percentage in the clay
- Amount of soil structure development
- Duration (t)

The material parameter could be in the form of a sand percentage.

Analysis
Goal: Obtaining a model to determine the erosion of a clay layer of a dike over time under wave loading as a function of the wave characteristics and the slope angle of the dike.

This erosion over time is influenced by the following parameters:



Assumptions

- The influence of the breaker type and thus the slope angle and the wave characteristics (wave steepness) is important.
- The amount of soil structure development is important for the erosion resistance of the clay.
- The soil structure development is amongst others a function of the time and the clay type. (The other factors are not taken into account in the model.)

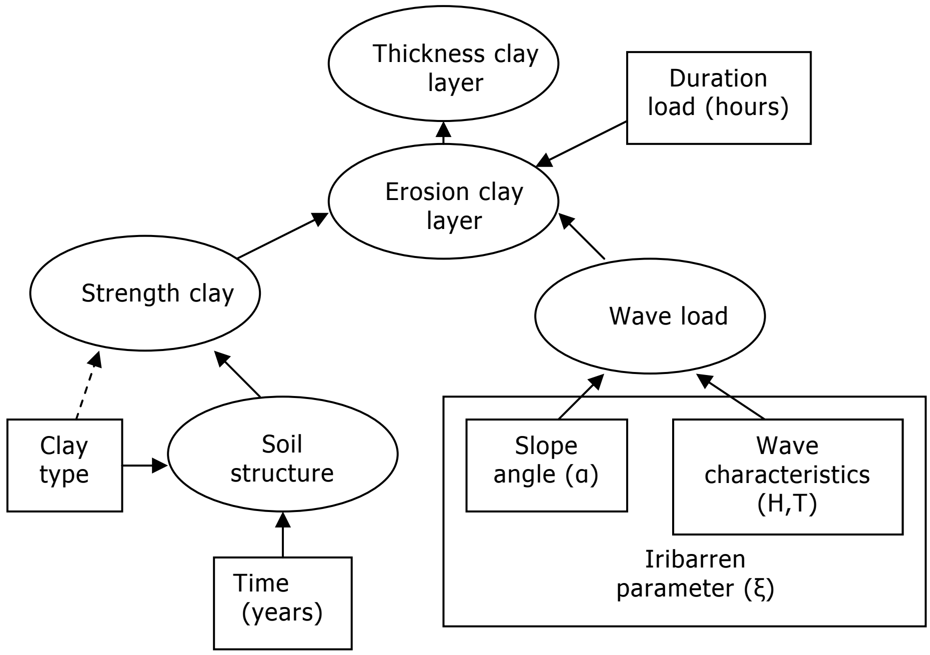
- The distribution of the strength and weak locations along the slope is not taken into account in the modelling.

Schematisation
System borders: The total thickness of the clay layer on a dike
Time scale: hours

Process analysis:
- Due to the erosion the thickness of the clay layer decreases.
- A hole develops in the clay layer due to wave loading. This hole develops further into the clay due to the wave loading.
- A cliff shape develops.
- The erosion takes place due to wave loading on the dike.
- The wave load on the slope is depended on the breaker type, and thus on the slope angle and the wave characteristics.
- The strength of the clay layer is decreased by the soil structure development. A soil structure develops as a function of time and the clay type.
- The eroded clay particles are removed from the dikes into the sea by the wave current.

Relevant:
The amount of soil structure development over time.

Causal relation diagram:



Parameters:

Table 11.1 Influencing parameter on the erosion of clay with unit and order of magnitude

Parameter	Symbol	Unit	Order of magnitude
Thickness clay layer	d	m	m
Wave height	H	m	m
Wave period	T	s	s
Slope angle	α	o	o
Iribarren parameter	ξ	-	-
Duration load	t	h or s	hours
Erosion velocity	e	m/s	dm/hour
Total erosion after x hours	$E_{t,x}$	m	dm
Depth of soil structure	D_c	m	dm
Soil structure development over time	d_s	m/year	dm/year
Clay type/Material parameter	c	-	-

11.2 Schematized erosion process

The start of erosion
For a certain combination of a water level and wave height, the erosion process starts. This starting point is dependent on the strength of the vegetation layer present in the clay and the presence of weak locations in the grass and the clay cover layer. This resistance to the initiation of the actual erosion of the clay layer determines the starting point of the erosion process. The point of the actual erosion of the clay layer is given by point S in Figure 11.1.

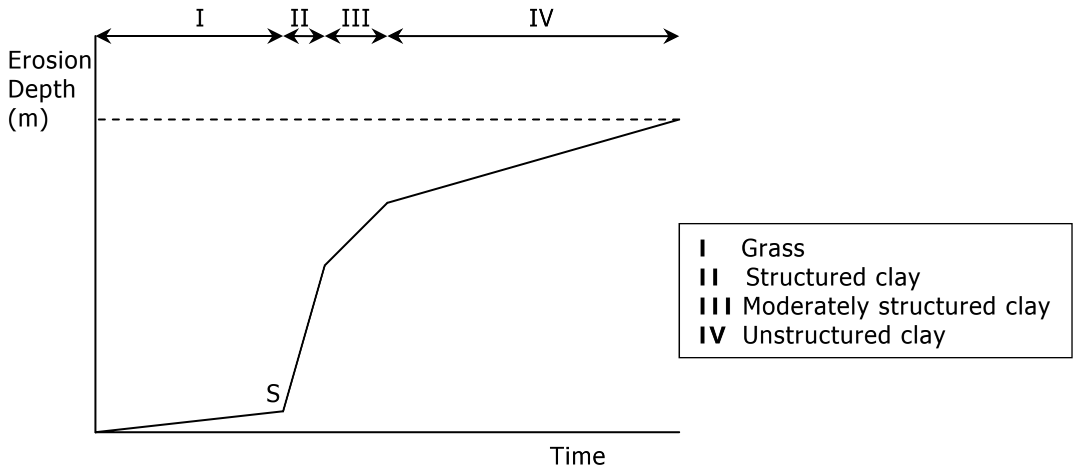


Figure 11.1 Schematized erosion process of a revetment consisting of a thick clay layer. The starting point (S) of the actual erosion of the clay layer is dependent on the strength of the vegetation layer present in the clay and the presence of weak locations in the grass and the clay cover layer.

The progress of the erosion
If the initial hole has been developed, the shape of the hole has already approximately the shape of the final erosion profile. The steep cliff shape develops after the start of the initial hole. This steep cliff shape has been developed, the erosion proceeds into the dike, see Figure 11.2. During this process the cliff has approximately the same shape.

The angles of the erosion profile can be approached based on the averaged results of the experiments and observations in actual storms. The cliff has approximately an angle of 50° with a horizontal. The bottom of the erosion profile has an angle of approximately 5° with the horizontal. See Figure 11.2. The size of the developed hole is and the erosion rate are dependent on the hydraulic conditions and clay properties.

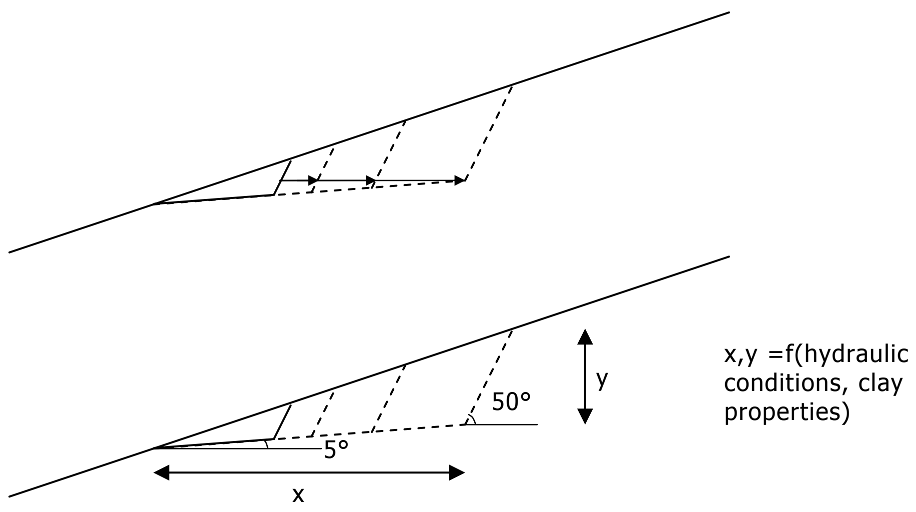


Figure 11.2 Schematized development of an initial erosion hole. The angles of the erosion profile are averaged values, based on the results of the experiments on clay and observations of erosion in actual storms. The size of the developed hole and the erosion rate are dependent on the hydraulic conditions and clay properties.

The progress of the erosion after the development of an initial erosion hole is given by x. this progress is a function of the hydraulic conditions and the material properties. If fixed angles used for the shape of the erosion profile of 5° and 50°, the erosion depth, given by y, is dependent of x and the slope of the initial profile.

y=f(x, slope)

Dimensions erosion profile
The dimensions of the schematized erosion profile can be derived from this profile, given in Figure 11.3.

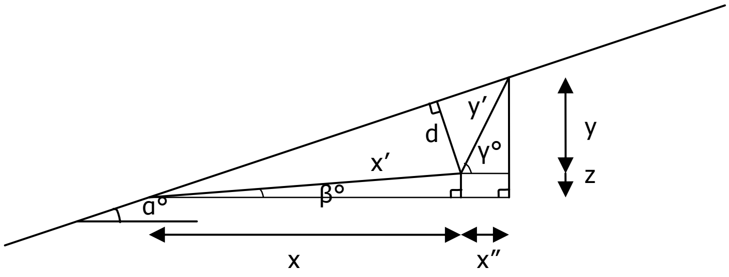


Figure 11.3 Schematized profile with dimensions

For a given x, the following relations are derived, with y and d as the main interests:

$$z = x\tan\beta$$

$$x'' = \frac{y}{\tan\gamma}$$

$$y = \frac{x\tan\alpha - x\tan\beta}{\left(1 - \frac{\tan\alpha}{\tan\gamma}\right)}$$

$$d = \frac{x}{\cos\beta}\sin(\alpha - \beta)$$

If γ° is 45°, then:

$$y = \frac{x\tan\alpha - x\tan\beta}{(1 - \tan\alpha)}$$

For the dimensions of the shape of the erosion profile x' and y' the following relations are formulated:

$$x' = \frac{x}{\cos\beta}$$

$$y' = \frac{y}{\sin\gamma}$$

If given x=2m, than d=0.3m, which is approximately in accordance with the results of DF1992Sa and DF1992Sc.

Water level fluctuation
During a storm a varying water level is present, as a result of which the wave load on the slope is moving up and downwards along the slope together with the rise and fall of the water level, Figure 11.4. Therefore the wave load is not constantly on the same location along the slope during a storm. Also the erosion can therefore initiate on different locations along the slope, with the result that initially multiple holes can develop which connect later on if the storm progresses. A water level dependent and time dependent erosion profile is the result. The final shape of the connected profiles could have the form of a steep cliff shape or have a more parabolic shape. A step profile is visible in the erosion profile of Domburg in 1953, see 10.4. In reality the difference between the two shapes will probably be minor. For modelling purposes the angular shape is the easiest to use. This shape is similar to a dune profile. See Figure 11.4.

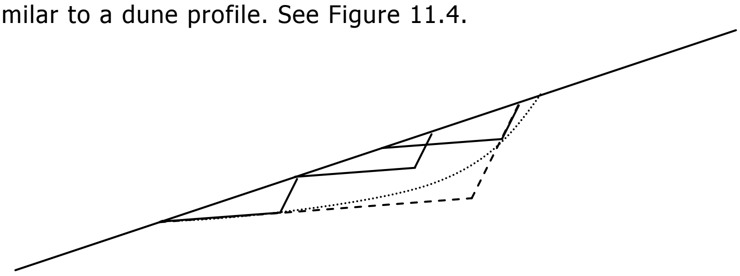


Figure 11.4 Due the water level fluctuation, the erosion initiates at different locations along the slope, which can be connected into a final profile.

The time and water level dependence of the erosion process can be schematized by dividing the water level fluctuation in several time steps of a certain amount of hours. The water level can than be averaged over these hours. In this way a complete storm duration can be schematized in periods with a certain constant water level. In Figure 11.5 an example is given if the storm surge of 1953 would be divided in periods of four hours. With this method a storm is schematized by stairs shaped water level development over time.

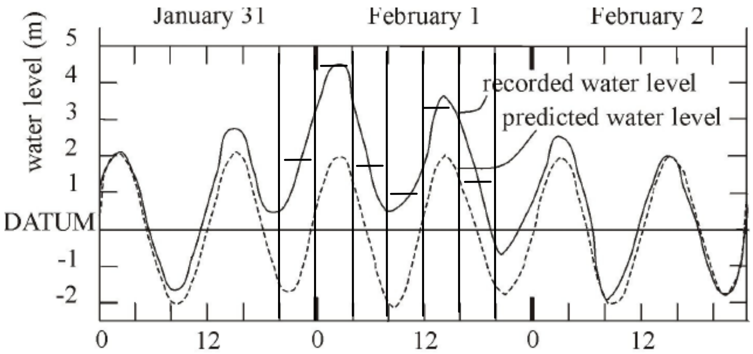


Figure 11.5 The storm duration can be schematized in periods with a certain constant water level. In this figure given for the storm surge of 1953.

By TAW (WG, 2001) the duration of a normative storm along the Dutch coast and the Westerschelde is 35 hours and the course of the storm is schematized in the following way:

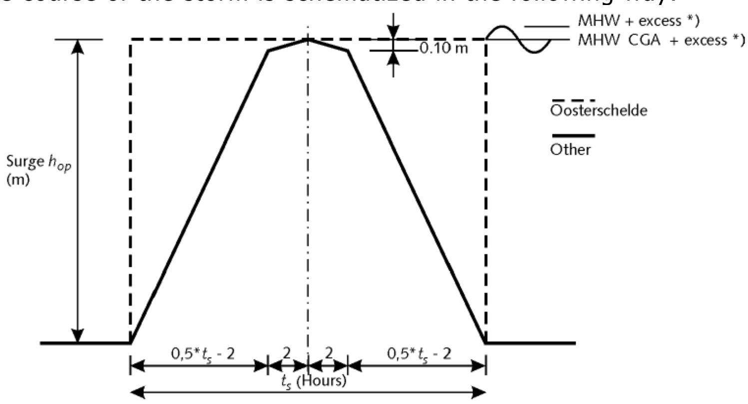


Figure 11.6 The course of a storm duration [TAW WG, 2001]

For an even more simplified schematisation a constant normative water level can be used. This is can be done for a design method, but for an accurate description of the erosion process this will be too simplified.

Not just the erosion during a design storm is interesting, also the erosion developed in smaller storms, in order to describe a maintenance policy.

The erosion measured horizontal and perpendicular to the slope
The progress of the erosion is horizontal directed under an angle of approximately 5°. This horizontal erosion is constantly measured on the same location.
The maximum erosion measured perpendicular to slope is varying in location on the slope over time, as is visualised in Figure 11.7. For a constant water level, the location of maximum erosion is over time moving upwards along the slope.

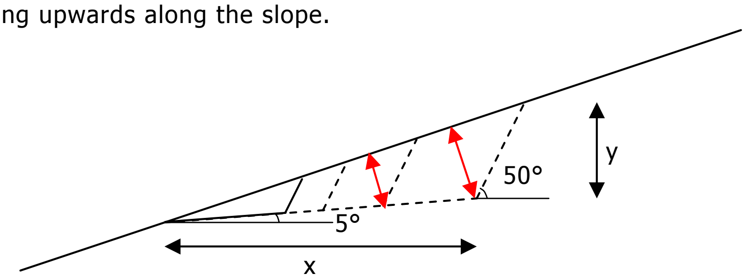


Figure 11.7 The maximum erosion measured perpendicular to slope is varying in location on the slope over time

Model
The erosion over time can be modelled in both ways. Both directions are for the given approximated angles dependent on each other as can be seen in the tekstbox 'Dimensions erosion profile'.

In the model determined in this chapter, the erosion measured perpendicular to the slope is used. This is chosen, because the model will be based on the results of the comparison of the large scale experiments on clay. (Chapter 8) In the reports of these experiments, the maximum erosion is given perpendicular to the slope. The horizontal erosion, which is used in the comparison with erosion of sand (Chapter 9) is based on drawn erosion profiles and is therefore less accurate.
Additional reasons are the fact that the design method for clay dikes and the empirical model (WL, 2006b) determine also the thickness of the clay layer, which is perpendicular to the slope.

11.3 Semi quantitative model

The goal of this chapter is to obtain a model to determine the erosion of a clay layer of a dike under wave loading over time. From the comparison of the large scale experiments in chapter 8 has been concluded that the clay condition (the soil structure development) has a significant influence on the strength of the clay under wave loading. Based on a subdivision in clay conditions, the clay type and the wave conditions are of influence. The breaker type will probably also of influence on the erosion process. (See paragraph 11.4.2 for further information.) Therefore the starting point of the model is the clay condition.

The development of the erosion depth over time could be described roughly as given in the graph in Figure 11.8 A. The grass on the cover layer is included here for the overall picture, but is not a detailed further. This grass is assumed to have a high resistance against wave loading. The erosion of the structured clay (II) will develop with a certain large erosion rate, if the amount of structure decreases (III), the erosion rate also decreases. Unstructured clay (IV) has a low erosion rate.

A beginning of quantitatively describing the erosion process of the clay layer can be made if the gradients of the different linear parts of the graph and the locations and values of the angles in the graph are determined.

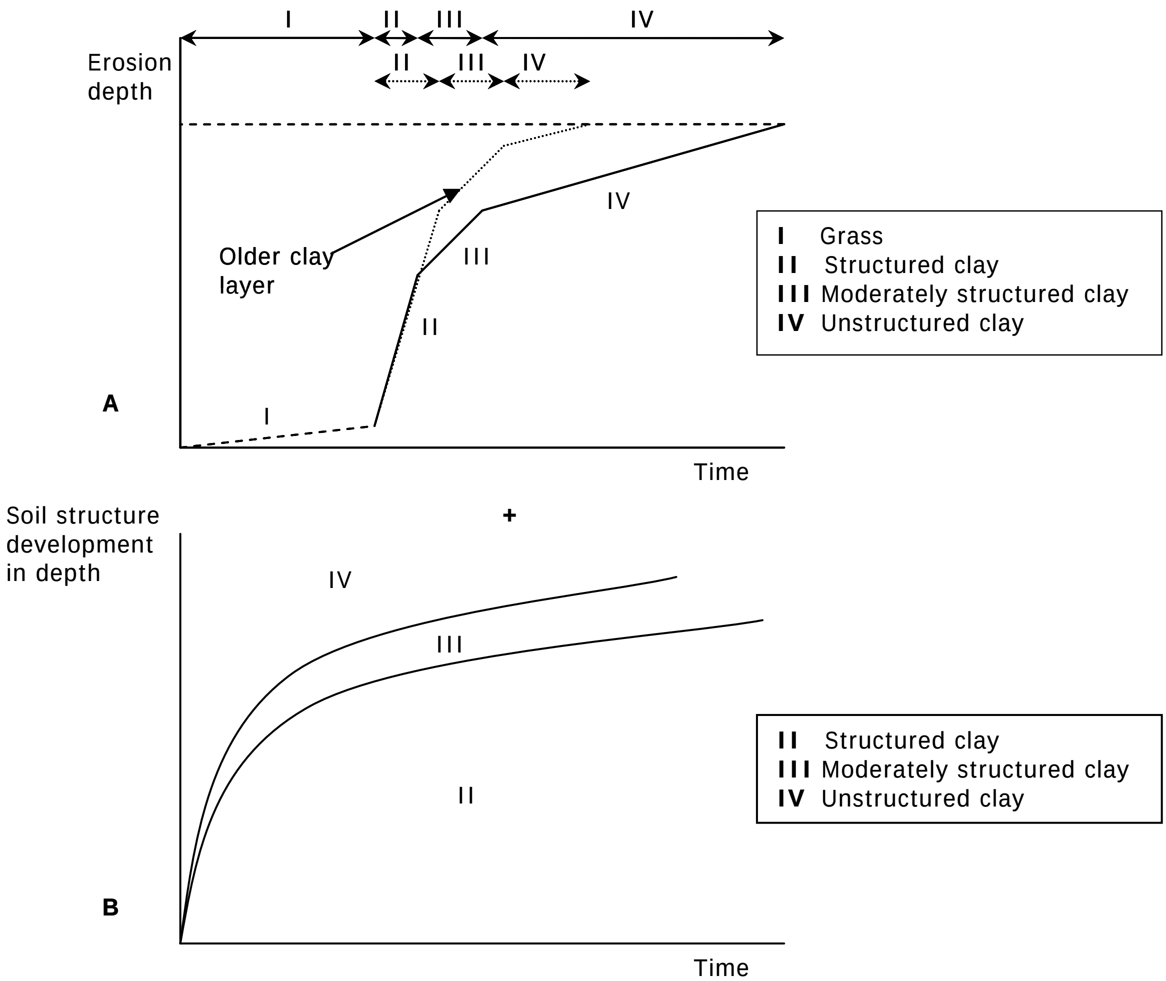


Figure 11.8 Basis of the determination of the development of the erosion depth over time (A). If the amount of soil structured development of a clay layer over time (B) is known, the locations and values of the angles of the graph of the erosion depth over time can be determined.

A soil structure will develop over time, therefore the upper part of the clay layer will always be structured in a few years, see also chapter 7. An older clay layer will for a larger part be structured. This is also given in Figure 11.8 A.

If the amount of soil structured development of a clay layer over time is known, the locations and values of the angles in the graph of Figure 11.8 A can be determined. A model of the erosion over time can be formulated if the erosion rates of the different clay conditions are determined and the amount of soil structure development of a clay layer over time is known. See Figure 11.9.

11.3.1 Erosion development over time and in depth per clay condition

Based on the results of the large scale experiments a formula for the erosion development over time can be made on the basis of the erosion rates of the different clay conditions. For the three parts of the clay conditions three different formulas can be obtained. Based on the comparison of the large scale experiments these three formulas can be assumed to be all linear.

First approach

A formula for each part of the graph can be obtained from the data of the Delta Flume experiments. The gradients of the trendline through the datasets of each test are averaged per clay condition. With this method a gradient per clay condition is obtained which can be used per linear part of the graph. The total range of the gradients of the dataset per clay condition is given by the grey surface. See Figure 11.9.

Table 11.2 Averaged gradient per clay condition

Clay condition	Gradient, based on dataset experiments (m/hour)
Structured	0.3
Moderately structured	0.1
Unstructured	0.05

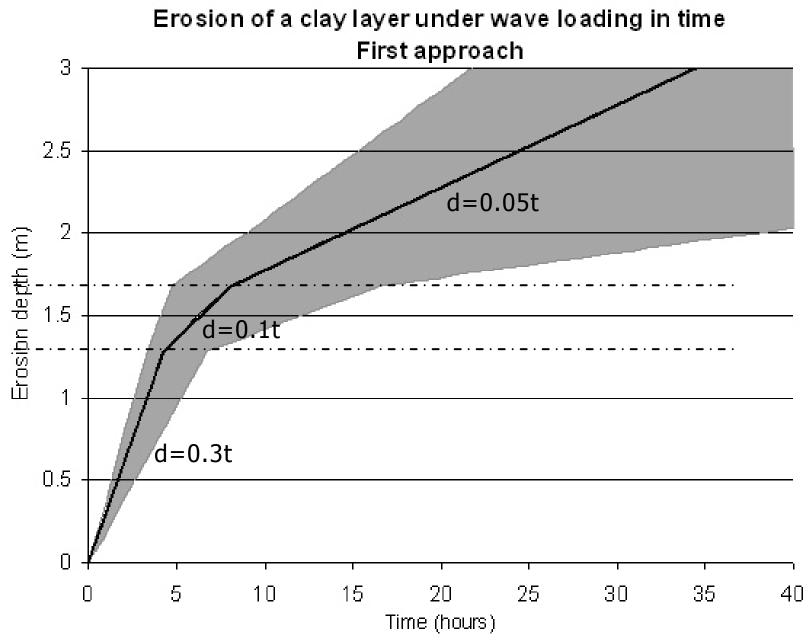


Figure 11.9 First approach of a model of the erosion of a clay layer over time under wave loading The grey surface gives the range in gradients of the dataset per clay condition. The line is the averaged gradient per clay condition. The locations of the transitions, given by the dotted lines, are determined in 11.3.2.

More accurate

For a more accurate determination of the gradient for each part of the graph is more closely looked into the datasets per clay condition.

Structured clay
Structured clay is present in the upper part of a clay layer, and therefore the results of the large scale experiments on the 0.8m thick clay layers form representative data sets. However it could be stated that the last data points of the datasets can be excluded, because below a depth of approximately 0.5m, the clay in the experiment could be more moderately structure, which can explain the more flat course of the last part of the graph. If the last points of both datasets are excluded, the gradient of structured clay will be higher and an average gradient of 0.4 is obtained, instead of 0.3.

Moderately structured clay
Moderately structured clay is present under the structured clay a bit deeper into the clay layer. It forms a transition between structured and unstructured clay. Therefore could be stated that the first data points of the datasets, to a depth of approximately 0.3m, can be excluded for a more representative gradient. If this is done a gradient of 0.2 is obtained, so the erosion rate is larger.

Unstructured clay
Unstructured clay is placed deep in the clay layer and therefore the first few data points could be excluded from the dataset. In DF1984a suddenly a hole developed, probably a large clay particle or due to the high sand percentage of this clay. If this point is included, the gradient of DF1984a will be much larger than that of the other unstructured datasets. If it is excluded, the gradient will be almost similar to that of DF1984b and also more similar to other unstructured experiments. Because this model is not made to determine an erosion development of clay over time with a large safety (like a design method), but to give an indication of the actual erosion rate of clay, this point is not included in the determination of the gradient of unstructured clay. If this model will be used as a design method a safety factor can be added later.

An overview of the gradients and datasets used for determining the averaged gradient of unstructured clay is given in figure 16.6. The averaged gradient is 0.01.

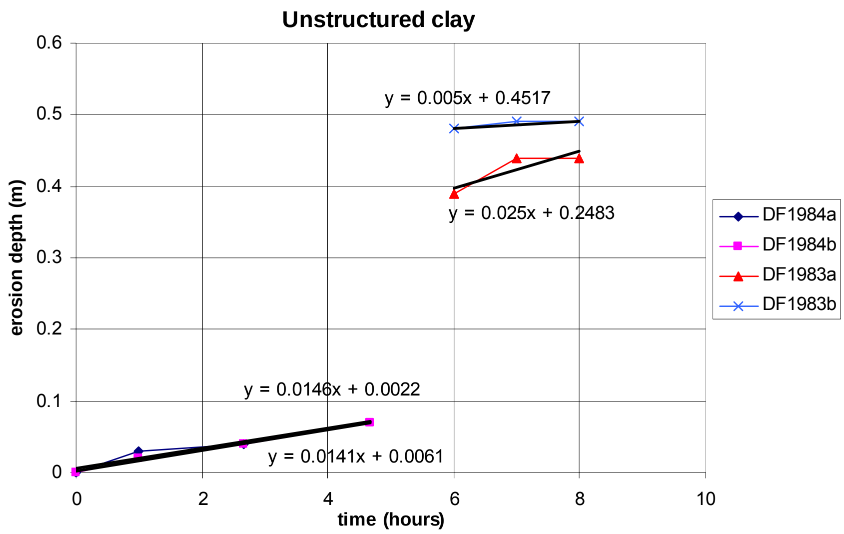


Figure 11.10 Datasets used for determination of average gradient of unstructured clay.

Results
An overview the gradients determined with this more accurate way by looking into the used data of the datasets is given in Table 16.3. The graph of the erosion of a clay layer under wave loading over time based on the more accurately determined gradients, the semi quantitative model, is given in Figure 16.7. The location and values of the angles in the graph, which form the transitions between the different clay conditions, are not determined yet. This has to be done on basis of the soil structure development over time and is done in the next paragraph. The location of the angles in this graph is based on a certain average expected location of the transitions.

Table 11.3 Averaged gradient per clay condition, more accurate

Clay condition	Gradient, based on dataset experiments more accurate (m/hour)
Structured	0.4
Moderately structured	0.2
Unstructured	0.01

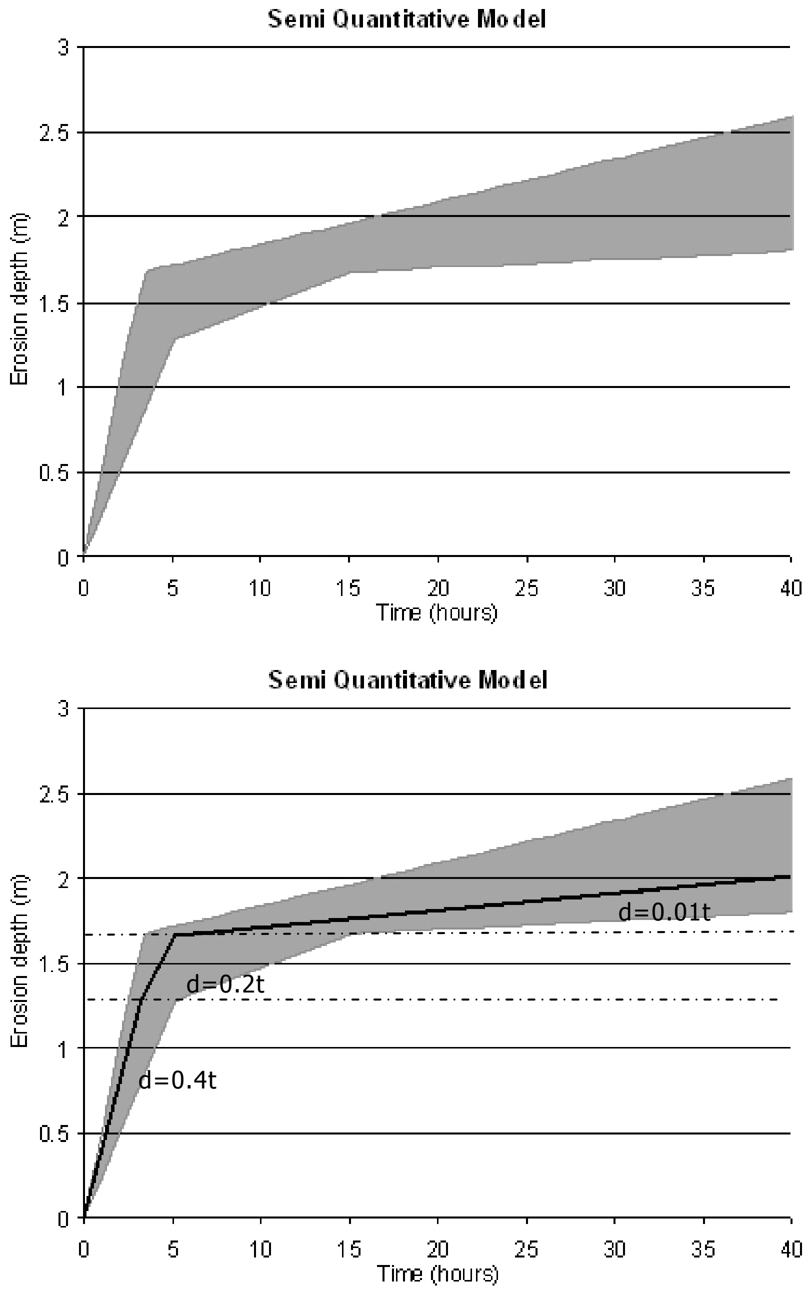


Figure 11.11 Semi quantitative model of the erosion of a clay layer over time under wave loading. The grey surface gives the range in gradients of the dataset per clay condition. The line is the averaged gradient per clay condition. The locations of the transitions, given by the dotted lines, are determined in 11.3.2.

Discussion
It is debatable if it is right to adjust the datasets, because the available data is already so limited that the representativeness of this data, and especially the adapted data, is low. With these adjustments the erosion rates are more accurate. The structured and moderately structured clay have a higher, and the unstructured clay a lower gradient than the first approach.
However this model is not made to determine a safe erosion rate of clay over time. This model has to give an approximation of the actual erosion rate of clay. If this model later will be used as a design method a certain safety factor can be added.

The averaged values of the gradients of the Delta Flume are used. So the differences in wave height or other properties are ignored. Therefore it is possible to state that this model can be used to determine erosion development over time for wave characteristics which are within the range of wave characteristics of the used Delta Flume experiments. This will thus apply for wave height till 1.5m.

This semi quantitative model can be refined by making it dependent on influencing parameters.

11.3.2 Soil structure development

The locations of the angles in the graph are dependent on the soil structure present in the clay layer. This soil structure develops over time and depends on several factors, given in chapter 7, but more knowledge should be obtained. Just after construction the whole clay layer is unstructured. In a few years the first few meters will be structured, after 50 years a whole 0.8m thick clay layer can be structured. Based on the quantitative data of TAW Klei (1996) and Kruse (2000), see Table 16.4, a graph with the development over time of years is made, see Figure 16.8. A trendline is drawn through the available data points. This line forms the boundary between structured and unstructured clay.

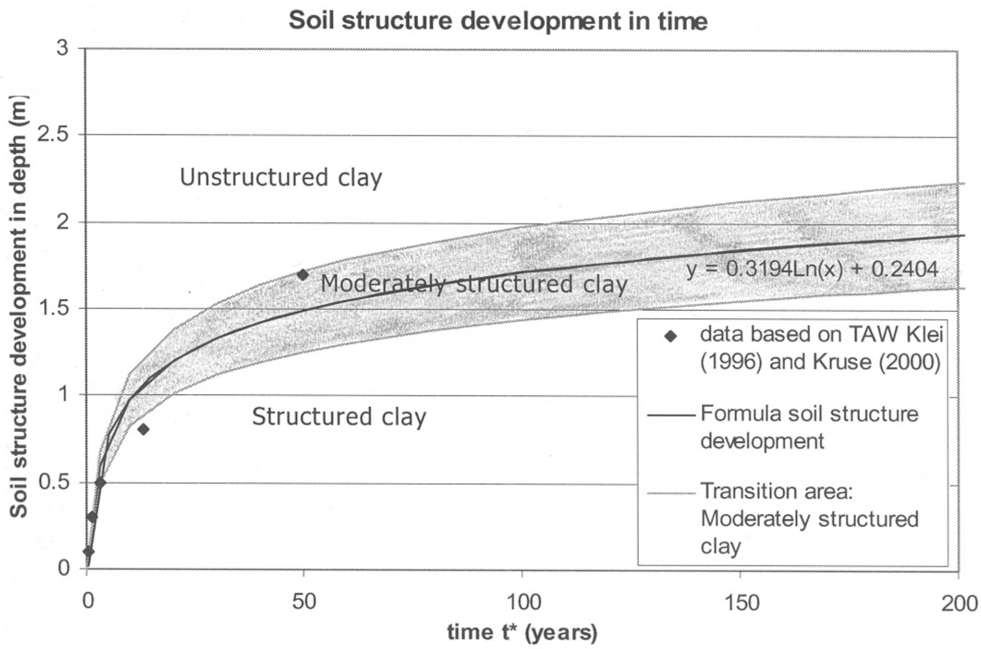


Figure 11.12 Soil structure development over time, with a boundary of moderately structured clay.

On basis of this data and the graph a formula for the soil structure development over time can be obtained from the equation of the trendline. This formula has however no physical basis and is based on very few data.

$$d_{soilstruc} = 0.32 \ln t^* + 0.24$$

t^* time in years after construction

Based on this graph, the transitions between structured, moderately structured and unstructured clay can be determined. The line forms the boundary between structured and unstructured clay, but from the large scale experiments appeared that moderately structured clay as a present clay condition. A boundary layer of moderately structured clay is present, given in the graph by the grey border. This boundary area will also function as a safety margin for the little data which is used to base the formula on. The graph is thus better to use than the formula.

With this graph the transitions between the clay conditions of the clay layer can be determined, which can be used on the semi quantitative model.

Table 11.4 Used quantitative data of soil structure development

Time (years)	Soil structure development in depth (m)	Reference
1-3	0.3-0.5	Kruse (2000)
10-15	0.8	TAW Klei (1996)
50	1.6-1.8	Kruse (2000)

Discussion
This formula is based on the age of the clay layer. Thus for calculating the depth of the soil structure development the date of construction of the cover layer should be known or assumed.

The influencing factors of this development over time are not included due to a lack of information. One of the influencing factors is the clay type, which is probably an important factor in the soil structured development over time and also in the erosion process itself. No quantitative values are known about this factor.

The determined formula for the soil structure development over time has no physical basis. This is a mathematical formula based on few data. However, it is known that a soil structure develops over time, but due to the qualitative knowledge of this development and the influencing factors, it can be concluded that a certain limit is reached in time. A LN-formula has certain limit, thus this could for a good basis for a formula which describes the soil structure development over time. The derived formula is the best fitted to the limited data which is known at the moment.

11.3.3 Semi quantitative model

For determining the erosion of a certain clay layer under the influence of wave loading the results of the two modelling steps have to be combined.

On the basis of a known lifetime of the clay layer, the transitions between structured, moderately structured and unstructured clay can be determined. With this information the angles of the three-linear graph are known and the erosion depth over time is known.

The semi quantitative model is based on wave heights of 1.0m-1.5m. The outer line of the grey area in the semi quantitative model represents the wave height of 1.5m. Because the available amount of data is limited and observations of actual storms with slightly higher wave heights give clearly more erosion than the experiment results, it is advised against extrapolation of the semi quantitative model for higher wave heights without further research.

An example of this model is given in a Case Study in 17.2.

The semi quantitative model could form the basis of a design method. The design life time of the clay layer, before large scale maintenance will be performed, has to be used to determine the depth of the soil structure development. A minimum required thickness of the clay layer is in this case determined instead of a maximum erosion depth. A safety coefficient should be included.

11.4 Refining the semi quantitative model by including influencing parameters

11.4.1 Theory

The obtained semi quantitative model can be refined by making it dependent on influencing parameters of the load and the strength. The influence of the different parameters can be only proved by the present data of the large scale experiments.

The influencing parameters are:

- Wave characteristics (H/T) — or combined in Iribarren parameter (ξ), breaker type
- Slope angle (α)
- Clay type/ material parameter

The clay condition is already included in the semi quantitative model.

Interesting to know is what the result of a change in erosion rate is, if an influencing parameter changes. For example the change in erosion rate if the wave height increases or decreases, or if the Iribarren parameter increases or decreases.

A formula for the erosion depth could have the form of

$$d = c^m \cdot y^n \cdot t^x$$

d erosion depth (m)
c influencing parameter 1
y influencing parameter 2
t time (h)
m, n, x certain powers

The wave characteristics and the breaker type have probably an influence on the erosion process. Therefore can be stated that is looked for a formula for the erosion depth of a clay layer over time under wave loading as function of the wave characteristics (H, T) and the slope angle (α). Both these parameters are combined in the Iribarren parameter (ξ).
The clay type is also of influence on the erosion rate. This influence could be captured in a certain factor C_{clay}.

This results in: Erosion=f(Iribarren parameter, clay type, t)
or

$$\frac{dd}{dt} = f(\xi^n, C_{clay})$$

If the linear relationship of the erosion depth over time is assumed as is assumed for the semi quantitative model, x can be stated to be 1.

for example:

$$d = c^m \cdot \xi^n \cdot t$$

d erosion depth [m]
c certain (material) parameter
ξ Iribarren parameter [-]
t time [h]
m, n, x certain powers

For each large scale experiment a list of the different possible influencing factors per clay condition is given in Table 11.5.

11.4.2 Results

Table 11.5 Properties experiments per clay condition

Clay condition	Experiment	ξ (-)	U_{max} (m/s)	H_s (m)	T_p (s)	H_s* ξ (m)	tan α
Structured	DF1992Sb	1.3	6.5	1.47	4.9	1.9	1:4
	DF1992Sd	1.3	5.4	1.0	4.2	1.3	1:4
Moderately structured	DF1992Sa	1.3	6.5	1.47	4.9	1.9	1:4
	DF1992Sc	1.3	5.4	1.0	4.2	1.3	1:4
	DF1992Ga	1.3	6.2	1.35	4.7	1.8	1:4
	DF1992Gb	1.3	6.2	1.35	4.7	1.8	1:4
Unstructured	DF1984a	4.2	9.9	1.05	12	4.4	1:3.5
	DF1984b	4.2	9.9	1.05	12	4.4	1:3.5
	DF1983a	0.7	4.9	1.57	5.3	1.1	1:8
	DF1983b	0.7	4.9	1.57	5.3	1.1	1:8

$$d = c^m \cdot \xi^n \cdot t \qquad d = c^m \cdot U_{\max}^n \cdot t$$

$$d = c^m \cdot H_s^n \cdot t \qquad d = c^m \cdot (H_s \xi)^n \cdot t$$

Table 11.6 Optimum for n, per parameter if c^m =1

Clay condition	Optimum for n (-), if m=1				
	ξ	U_{max}	H_s	T_p	H_s* ξ
Structured	-	-	1.8	-	4.5
Moderately structured	-	-	(4.5)	-	-
Unstructured	-	-	-	-	-

Velocity on the slope

There is no reliable formula to predict the maximum velocities on a slope during uprush and downrush. The wave loading on a slope can as a first approximation be roughly transformed into a maximum velocity component for run-up and run-down. [Pilarczyk 1990, 1998] The maximum velocity component on the slope calculated for each experiment is given in Table 12.7

$$U_{\max} \cong a\sqrt{gH_s\xi_z}$$

U_{max} maximum velocity component on the slope during run-up and run-down [m/s]
a coefficient:
1.5 for irregular waves [Pilarczyk, 1998]
1 for irregular waves and smooth slopes [Pilarczyk, 1990]
g gravity [m/s^2]
H_s significant wave height [m]
ξ_z Iribarren parameter [-]

Table 11.7 The maximum velocity component on the slope calculated for each experiment

Properties	Experiment					
	DF1984	DF1992Sa,b	DF1992Sc,d	DF1983	DF1992G	DF1992Ggrass
Hs [m]	1.05	1.47	1	1.57	1.35	1.35
tan α [-]	1:3.5	1:4	1:4	1:8	1:4	1:4
ξ [-]	4.2	1.3	1.3	0.7	1.3	1.3
Umax [m/s], a=1	6.6	4.3	3.6	3.3	4.1	4.1
Umax [m/s], a=1.5	9.9	6.5	5.4	4.9	6.2	6.2

It appears from the large scale experiments that the erosion of the clay layer takes place just below the water level. The run-up and run-down of the water takes place above the water level, thus this formula to determine the velocity on the slope is calculated at a location on the slope where the erosion does not take place.

Influence breaker type
Obtaining a model to determine the erosion of a clay layer of a dike over time under wave loading as a function of the wave characteristics and the slope angle of the dike (breaker type) is not possible on basis of these available data sets. For the structured clay and moderately structured clay the Iribarren parameters of the dataset are the same, and therefore an optimization of the Iribarren parameter per clay condition is not possible. Only for the unstructured clay an optimization is possible.

Influence maximum flow velocities along the slope
Also the maximum flow velocities along the slope for structured and moderately structured clay are too close to each other to form a clear distinction. For the formulas given below and a c^m-value of 1, an optimum is only possible to obtain for the wave height.

Influence wave height

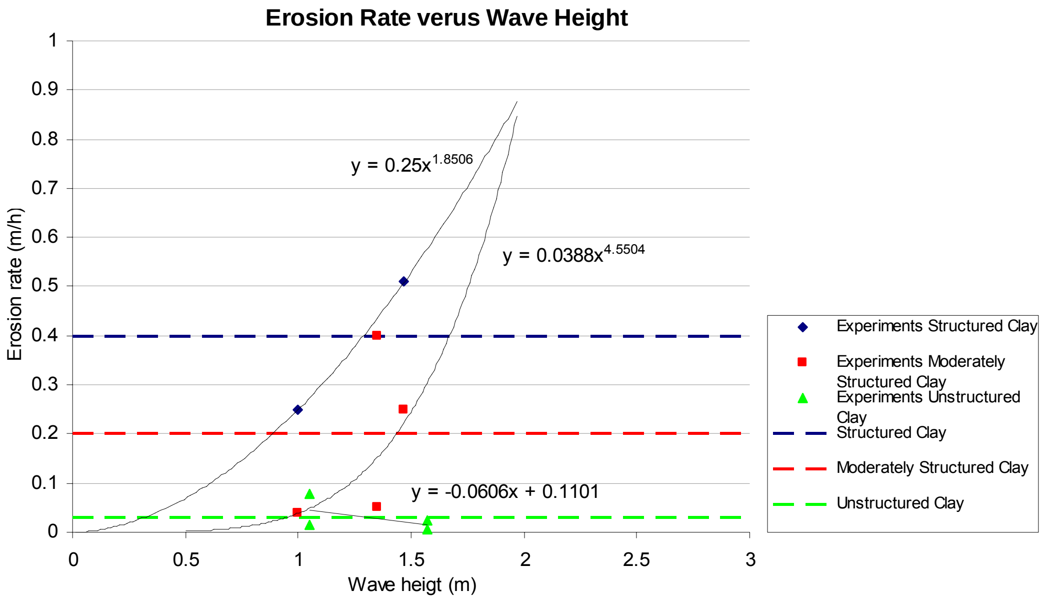


Figure 11.13 Influence of the wave height on the erosion rate of structured clay under wave loading

Structured clay
For structured clay a clear relation with n=1.8 is to recognize, see Figure 11.13. This can be round up to a relation with H^2.

$$d = 0.25H^2t$$

For this calculation method the value for c is in both cases the same, but different clay types are used in the experiment. Therefore c has to be variable as well.

Moderately structured clay
From a trendline through the data a relation to the power of 4.5 is obtained. The given data is however very scattered and probably other factors are of influence on the erosion rate as well.

Unstructured clay
The trendline through the data points are replaced through the origin, to see if they converge to a certain n-value. This appeared not to be possible.

Conclusion
At the moment there is not sufficient data to quantify the influence of the parameters on the erosion development of clay under wave loading. Therefore it is also not possible to refine the semi quantitative model. More information is needed on this subject.

11.5 Sensitivity of the semi quantitative model to the wave height

The sensitivity of the semi quantitative model to the wave height is given by the grey surface of the model in Figure 11.11. Based on the given datasets (Table 11.8) the influence of the wave height on the development of erosion of a clay layer is more clearly given in Figure 11.14.

A higher wave height results in a higher gradient and thus a higher erosion rate. The semi quantitative model could be extended for higher wave heights if more data is obtained.

Table 11.8 Datasets experiments

Clay condition	Experiment	Wave height (m)	Trendline	Trendline adapted
Structured	DF1992Sb	1.47	d=0.38t+0.06	d=0.51t+0.01
	DF1992Sd	1.0	d=0.19t+0.06	d=0.25t+0.02
	Average		d=0.29t	d=0.38t
Moderately structured	DF1992Sa	1.47	d=0.28t+0.02	d=0.25t+0.06
	DF1992Sc	1.0	d=0.04t+0.19	d=0.04t+0.27
	DF1992Ga	1.35	d=0.14t-0.03	d=0.4t-1.2
	DF1992Gb	1.35	d=0.05t	d=0.05t
	Average		d=0.13t	d=0.19t
Unstructured	DF1984a	1.05	d=0.078t-0.049	d=0.014t-0.006
	DF1984b	1.05	d=0.015t+0.002	d=0.015t+0.002
	DF1983a	1.57	d=0.048t+0.034	d=0.025t+0.25
	DF1983b	1.57	d=0.055t+0.034	d=0.005t+0.45
	Average		d=0.049t	d=0.01t

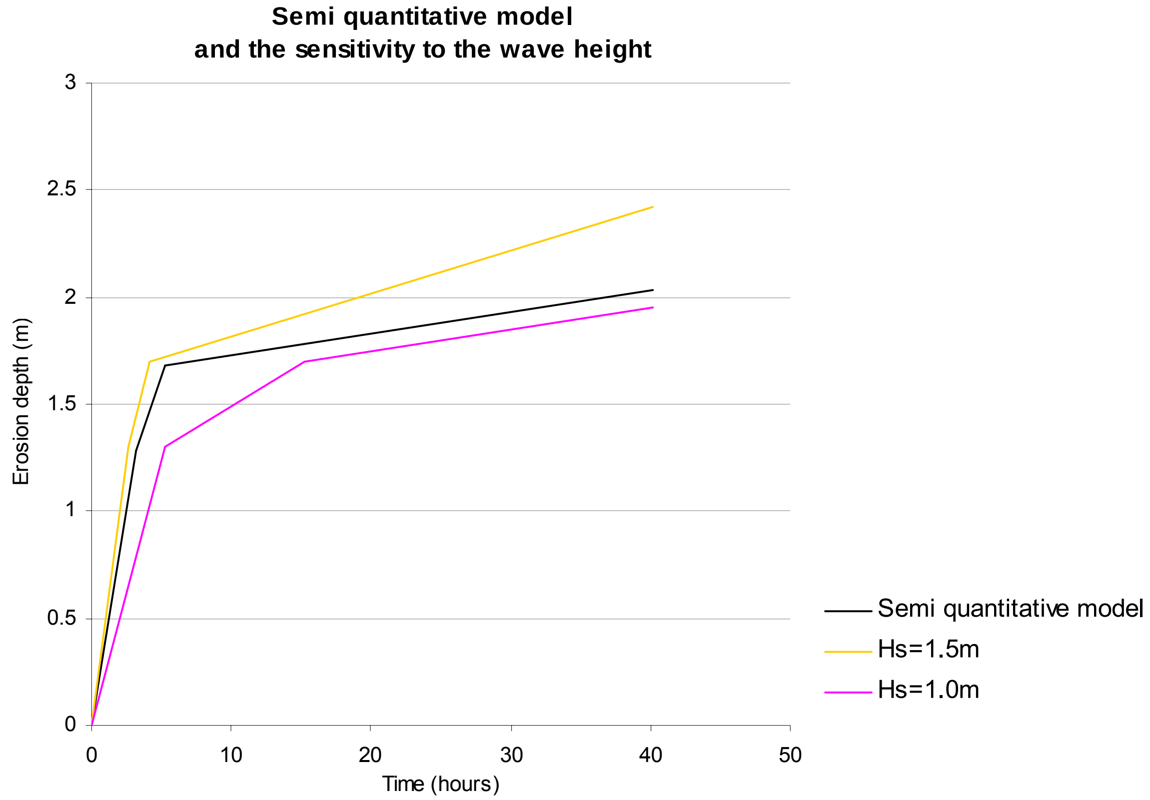


Figure 11.14 Sensitivity of the semi quantitative model to the wave height

11.6 Comparison of the semi quantitative model to existing models

In this paragraph the formulated semi quantitative model is compared to the existing empirical formula of the residual strength of clay (WL|Delft Hydraulics, 2006) and the existing design method by INFRAM (2003).

11.6.1 Empirical formula for the residual strength of clay

In 8.5 a comparison of the empirical formula formulated by WL|Delft Hydraulics (2006b) is made for all large scale experiments of clay under wave loading included in this research. From this comparison appeared that it does not give valuable results for the other experiments than it was fitted on. However, because this is the only available existing model, it is used to compare the semi quantitative model to. This is given in Figure 11.15.

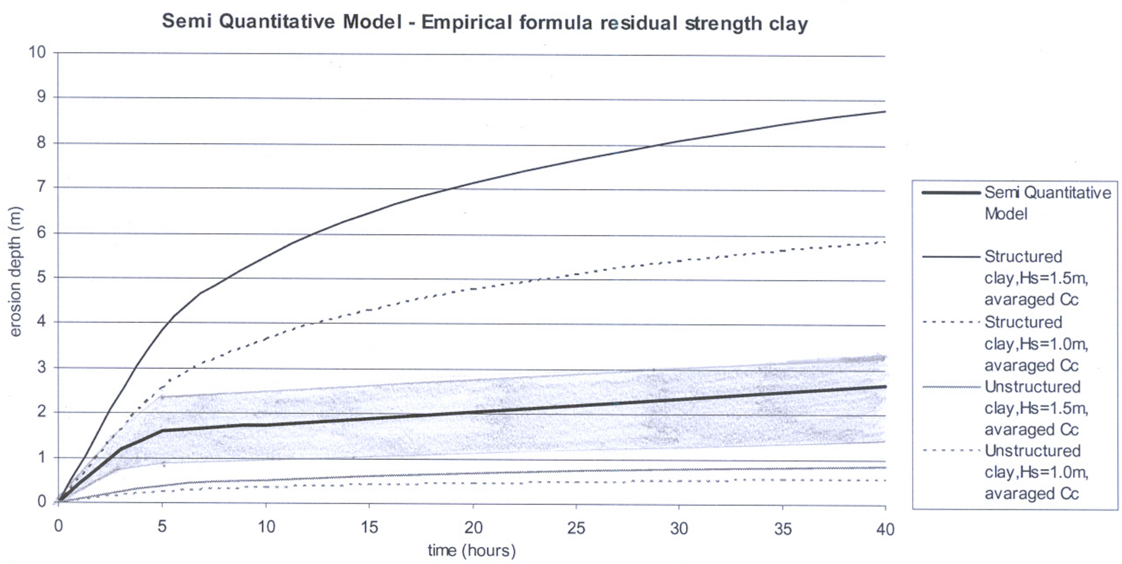


Figure 11.15 Comparison of the semi quantitative model with the empirical model to the residual strength of clay

Result

- The semi quantitative model determines approximately the average erosion over time in comparison to the developed empirical formula to the residual strength of clay by WL|Delft Hydraulics (2006b).

11.6.2 Design method INFRAM

The semi quantitative model is also compared to the current design method for clay cover layers by INFRAM (2003). This is given in Figure 11.16. The gradients of the design method are given in Table 11.9.

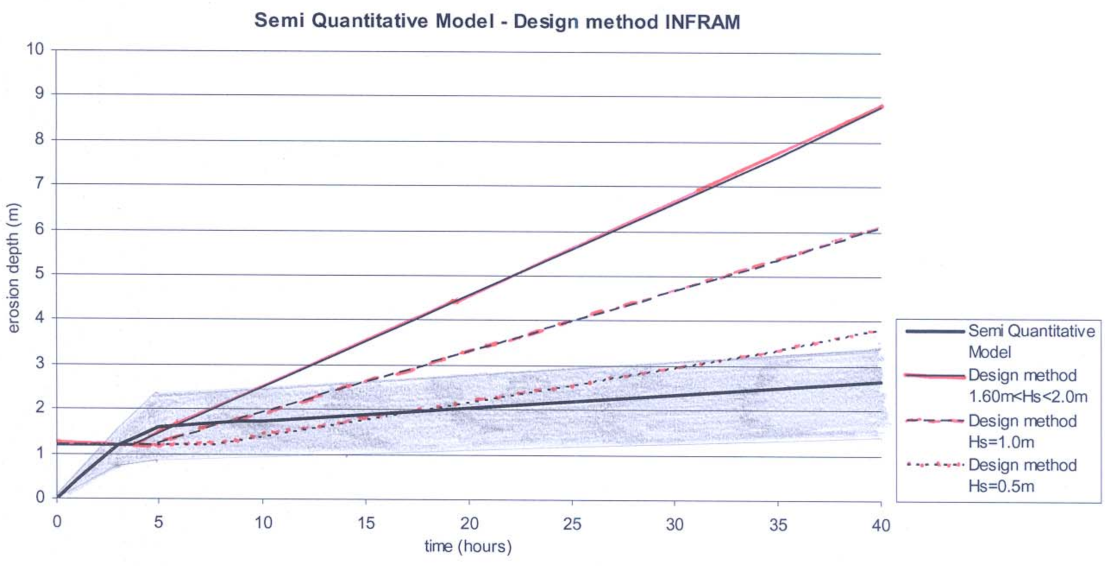


Figure 11.16 Comparison of the semi quantitative model with the design method for clay cover layers.

Table 11.9 Gradients design method.

H_s (m)	Gradients design method
1.60- 2.0	0.21
1.0	0.14
0.5	0.08

Results

- The gradients of the design method are generally larger than of the semi quantitative model, as expected. A design method has a certain safety factor. However between approximately 3 and 10 hours the design method determines less erosion than the semi quantitative model:
  For 1.60m<H_s<2.0m: Between approximately 3 and 10 hours.
  For H_s=1.0m: Between approximately 3 and 7 hours.
  For Hs=0.5m: Unknown, because the semi quantitative model is based on wave heights of 1.0m-1.5m.
  This is alarming, because a design method is supposed to have a certain safety margin.
- During longer storm durations, the erosion of the design method is larger than the semi quantitative model, as expected.
- The gradient of the design method of 1.60m<H_s<2.0m is the same as the gradient of moderately structured clay in the semi quantitative model, both 0.2m/h.

11.6.3 Combination

An overview of the semi quantitative model, the empirical model for the residual strength of clay and the design method is given in Figure 11.17.

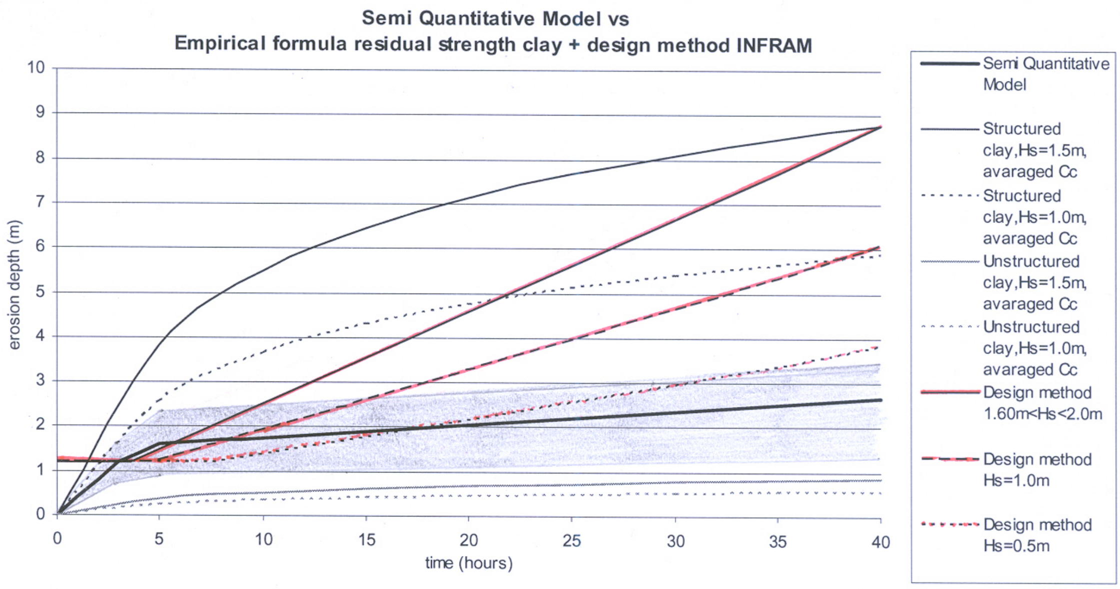


Figure 11.17 An overview of a comparison of the semi quantitative model, the empirical model and the design method.

11.6.4 Conclusions

- The semi quantitative model determines approximately the average erosion over time in comparison to the developed empirical formula to the residual strength of clay by WL|Delft Hydraulics (2006b).
- Between approximately 3 and 10 hours the design method determines less erosion than the semi quantitative model:

  For 1.60m<H_s<2.0m: Between approximately 3 and 10 hours.

  For H_s=1.0m: Between approximately 3 and 7 hours.

  For Hs=0.5m: Unknown, because the semi quantitative model is based on wave heights of 1.0m-1.5m.

  This is alarming, because a design method is supposed to have a certain safety margin. During longer storm durations, the erosion of the design method is larger than the semi quantitative model, as expected.

11.7 Use of DUROSTA for clay

DUROSTA is a dune erosion model, which is however several times used to determine the erosion of cohesive soils. [CUR, 1993; Oranjewoud, 1996; Stoutjesdijk, 2001] In this paragraph first the model is shortly described, after which the use for computing cohesive soils is treated.

11.7.1 DUROSTA

DUROSTA is a time dependent, physical-based cross-shore transport model which determines dune erosion based on sediment transport. Instead of a description of a final erosion profile, DUROSTA is able to compute the development of the erosion profile over time from the instantaneous cross-shore transport. The basic principle of the model is that the cross-shore sediment transport rate is computed from the product of local velocities and sediment concentrations. (For a more extensive description of the model is referred to Steetzel (1993).)

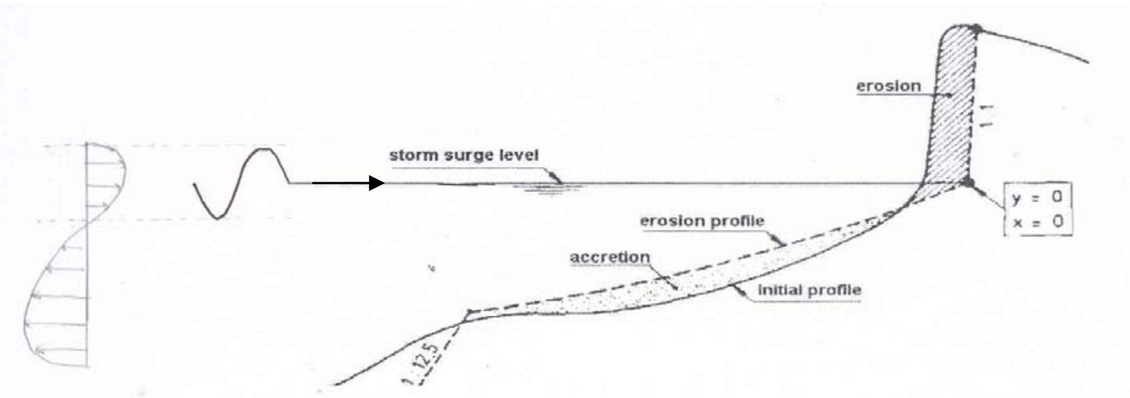


Figure 11.18 DUROSTA determines the erosion under the water level. Mass flux of the waves, return flow along the bottom.

The cross-shore sediment transport is calculated in DUROSTA by the depth integrated product of the time averaged velocity profile and the time averaged sediment concentration profile. The sediment transport is divided into a transport above the mean wave trough level (landward directed) and a transport below this level (directed offshore). See Figure 11.18. The landward transport is described by the product of the time-averaged concentration at the mean water level and the net amount of shoreward moving water, the mass flux. The offshore directed transport is the transport by the return flow.

Because of the lack of knowledge regarding the vertical distribution of the velocities and concentrations a simplification was made:

- The so-called wave related transport is neglected and only the current related transport is taken into account. (During extreme conditions this simplification is valid, because the amount of sediments in the vertical is very large and the transport depends mainly on the average velocities resulting from the relatively strong undertow. Outside the breaker zone this assumption is not valid because of the increasing correlation between the fluctuating water movement and sediment transport.)
- Only suspended transport is taken into account and bottom transport is neglected. Because the bottom transport is often landward directed, this assumption may imply an underestimation of the landward directed sediment transport. However for calculations in the breaker zone and short times scales this neglect is considered acceptable.

The sediment transports are computed to a critical water depth. From that point, the sediment transport is empirically extrapolated over the swash and dune face till a computed run-up level. In the model the waves thus do not necessarily have to impact the dune face in order for it to retreat. The geotechnical properties of the dune face are not considered in the model.
To be able to execute this extrapolation, a transition is point chosen first, with a transport which is used as a reference transport. The transport landward from the transition point is described the transport in that point times a reduction factor. This transport is thus expressed as a fraction of the reference transport, depending on the relative bed level and relative wave run-up. In this way the transport depends on the local bed level and thus differs over the entire beach-dune area.

Input DUROSTA:
- Cross section (in coordinates)
- Grain size diameter (D_{50})
- Significant wave height (H_{0s})
- Water level (h)
- Wave period (T_p)
- Fall velocity of the grains (w)

11.7.2 DUROSTA used for computing the erosion of clay

In 1993 and 1996, under the authority of CUR, research was performed to the use of DUROSTA as a model to determine the erosion of cohesive banks by ship waves. [CUR, 1993; Oranjewoud, 1996]. In 2001 DUROSTA was used to determine the erosion of a foreshore consisting of clay by Stoutjesdijk (2001). In CUR (1993) was concluded that due to the properties of clay the use of DUROSTA was for determining the erosion of cohesive material is debatable. However, the calculations give reasonable results.

In CUR (1993) calculations for two river banks were made, with the assumption that the banks were consisting of sand. For both calculations reasonable good results were obtained.
- For the Princes Margrietkanaal this assumption resulted in good results. A sand diameter of D_n=0.1mm and w=8mm/s was used, while the bank actually consisted of peat.
- For Dokkumer Grootdiep this assumption resulted in good results if a sand diameter of D_n=0.4mm was used. With a diameter D_n=0.1mm the results were not similar to the measured erosion. The bank actually consisted of a sand-clay mixture.

11.7.3 Discussion

Reasons for the use of DUROSTA to model the erosion development of clay:
- Sand and clay appeared to have similar shapes during erosion, a cliff. A hole develops with a steep cliff which retreats in horizontal direction. This is or both clay and sand the case. However, DURSTA determines the suspended sediments transport.
- The erosion development of the horizontal erosion of clay and sand is the same, except for unstructured clay. Only the erosion rate of clay is lower.
- The erosion of clay initiates below the water level and DUROSTA determines the sediment transport below the water level.
- Former results of the model on clay gave reasonable results.
- The erosion of clay is also water level, wave height and time dependent. The course of the water level over time can be included.
- DUROSTA determines the erosion profile for different time steps. This is also desired in a model of the erosion process of clay. Therefore a model similar to DUROSTA could be logical choice.

Reasons for not using DUROSTA to model the erosion development in a clay layer:
- There is still little known of the erosion mechanism of clay under wave loading.
- Clay is a cohesive material, which implies other influencing factors on the erosion process than for sand. Sand is considered as loose particles with a certain fall velocity.
- DUROSTA determines the sediment transport of the dune face from a transition point. (See also erosion mechanisms of sand in Chapter 9) This material is added to the suspended material below the water level. The erosion of clay initiates below the water level. The use of DUROSTA implies the addition of particles from above the water level (from above the transition point), which will not get into suspension in reality. Only if the erosion process of clay continues, the erosion can develop till above the water level and only then clay particles are indeed eroded.
- DUROSTA is based on sediment transport of suspended particles. Clay particles are much smaller than sand particles and will probably dissolve in the water. It is therefore unknown if the erosion process of clay can be based on sediment transport.
- The salt in the sea water has a different influence on clay than on sand, due to the clay minerals.
- A clay layer consists of different clay conditions with accompanying different erosion rates which very in depth. For sand the properties over depth are considered to be constant.
- Sand will erode over the whole width of the dune. For clay the erosion will start at the weakest location. At the weakest location a hole will develop. The start of erosion of clay is thus dependent on the distribution and properties of the weak locations. This should be included in the model, or a model of the erosion development of a weak location can be made. This could later on be extended if more knowledge is obtained on the distribution of weak locations along the slope.

- The eroded sand particles accrete, which delays the erosion process of sand. Clay erodes in lumps or small particles. Both have different fall velocities and a different behaviour in the water. Accretion of small particles will probably not occur. If a similar erosion process of sand is assumed, like DUROSTA does, the erosion process is than not decreasing over time but continuing.
  Only if a certain a material parameter can be varied for the clay condition and the accompanying different erosion rate per clay condition, a more accurate erosion process can be represented.
  Or if sand and debris, which are present in the water, will deposit in the erosion profile, a similar erosion process as for sand can be obtained. (See also 10.6) Also in this case the erosion process of clay will decrease.

Possible reasons for the reasonable results of use of DUROSTA for clay
The use of DUROSTA has several times resulted in reasonable results when used to determine the erosion of clay, while there are several reasons why it will not give good results, given in the list above. Apparently there are several factors which compensate each other.

- The obtained reasonable results for the erosion of clay with DUROSTA could possibly be the fact that this concerns ship waves loading river banks. The duration of ship waves is short and the waves are lower than wind waves. Therefore time is too short to develop an upper and under flow as is the case for wind waves and the amount of erosion is much lower than for wind waves. The differences in amount of erosion for sand and clay are therefore low which could lead to reasonable results for DUROSTA.
- DUROSTA determines the erosion above the transition point and includes this as suspended material. For clay no erosion takes place above the water level, but apparently this additional suspended material is necessary for obtaining reasonable results.
- Possibly the clay of the banks was completely structured. Structured clay has a more similar erosion rate to sand than unstructured clay.

Adjustments of DUROSTA for use of clay
From the given reasons in this paragraph the general conclusion is that more information needs to be obtained on the erosion process of clay.

Knowledge to be obtained
More information in the erosion process of clay should be obtained. In this case, especially the differences with the erosion process of sand. The following research questions can be formulated:
Is it possible to base the erosion process of clay and suspended sediment transport, which forms the basis of DUROSTA? The sediment transport formulation for cohesive material should be obtained.
Is clay accreting like sand is?
What is the influence of sand and debris in the water on the erosion process of clay?

If DUROSTA will be used for the erosion of a clay layer, the following adjustments should be made:
- In the model the soil structure development in depth should be integrated, with the accompanying erosion rates. The semi quantitative model could form a basis, with the subdivision in three clay conditions and the formula for the soil structure development over time.
  Additional, the clay type, the sand percentage in the clay, has to be included.
- The sediment transport process needs adaptations, as a different transport mechanism will take place for clay than for sand.
  In sand the erosion process delays over time, due to accretion of the sand. In clay unstructured clay has a lower erosion rate as structured clay.
  The process of accretion will not be similar as for sand. If there is no accretion, as assumed, the erosion profile of clay has to connect to the original profile per time step.
- For clay the erosion will start at the weakest location. At the weakest location a hole will develop. The distribution and properties of the weak locations should be included in the model. The development of the hole in clay could be integrated as a certain delay-factor. Or DUROSTA could determine just the erosion development of a weak location. This could later on be extended if more knowledge is obtained on the distribution of weak locations along the slope.

11.8 Conclusions

A semi quantitative model to determine the erosion depth of a clay layer over time was developed. This model is based on the determined subdivision of the experiments on clay in the three different clay conditions and a developed basic formula for the soil structure development over time and in depth.
On the basis of the present available data of the large scale experiments, it is not possible to quantify the influence of the clay type, breaker type and wave characteristics. Therefore it is not possible to refine the semi quantitative model. Only for structured clay a relation between the erosion rate and the wave height is to observe, however this is based on just two data points and no material factor can be included. An approximate influence of the wave height can be defined. The semi quantitative model can be extended, if the influencing factors on the erosion process of clay under wave loading are quantified.

The semi quantitative model could form the basis of a design method, if a safety coefficient will be included. The design life time of the clay layer, before large scale maintenance will be performed, has to be used to determine the depth of the soil structure development. A minimum required thickness of the clay layer is in this case determined instead of a maximum erosion depth.

The semi quantitative model is based on wave heights of 1.0m-1.5m. Because the available amount of data is limited and observations of actual storms with slightly higher wave heights give clearly more erosion than the experiment results, it is advised against extrapolation of the semi quantitative model for higher wave heights without further research.

The semi quantitative model determines approximately the average erosion over time in comparison to the developed empirical model to the residual strength of clay by WL|Delft Hydraulics (2006b). The empirical model however appeared not to be applicable for all Delta Flume experiments on clay.

Between approximately 3 and 10 hours the design method determines less erosion than the semi quantitative model. This is alarming, because a design method is supposed to have a certain safety margin. During longer storm durations, the erosion of the design method is larger than the semi quantitative model, as expected.

The use of DUROSTA as a model to determine the erosion of a clay layer under wave loading, which has been used several times in recent history, is debatable despite reasonable obtained results.

Part 4

Application of clay as revetment

Based on the analysis of the behaviour of clay under wave loading, preliminary research to the application of a thick clay layer as revetment for the outer slope of sea dikes was performed.

12 Application of clay as revetment

12.1 Design philosophy clay revetment

In order to define a certain safety approach for the use of clay cover layer as dike revetments demands have to be determined for the load which has to be resisted.
Two different approaches can be distinguished: a safety approach and a maintenance approach. Both approaches have differences and correspondences and will be explained in this paragraph.

Safety approach

- The clay cover layer should be able to sustain a normative load. The loading conditions for a normative load differ per location.
- The erosion will start at a weak location in the clay layer. Therefore the distribution of weak locations along the slope needs to be known and a definition of a weak location. For a safety approach can be assumed that the erosion will start in a weak location, the distribution and definition can be added later.

Maintenance approach

After a certain amount of erosion of the clay layer maintenance is needed to repair the clay layer. After a normative storm maintenance is always necessary. The frequency of this maintenance is dependent on the amount of erosion and the thickness of the constructed clay layer. The amount of erosion is dependent on the load on the dike, this differs per storm. The thickness of the clay layer should at least be able to resist a normative wave load, this will be the minimum required thickness. However, not only normative storms occur along the coast, smaller loads can also induce erosion.

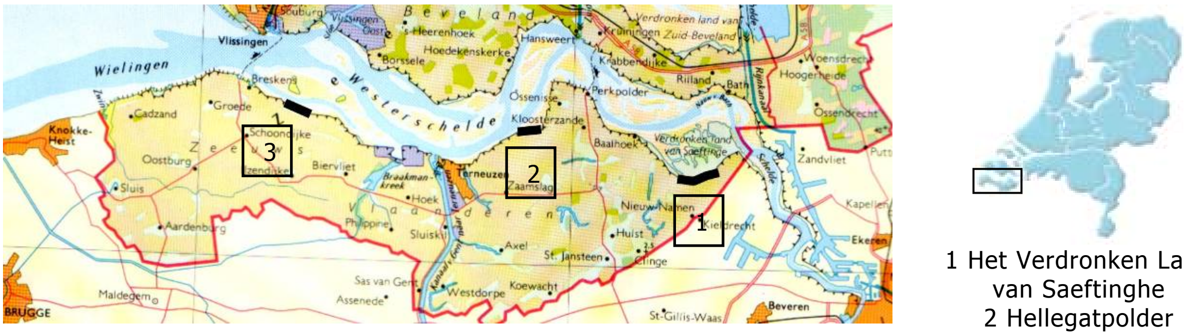
A maintenance approach should be formulated to determine when maintenance should be performed and what the required thickness of the clay layer should be. Maintenance could be done after one normative storm, two normative storms, two yearly storms and one normative storm or another approach. These different choices imply each another needed thickness of the constructed clay layer and also a different frequency of maintenance. With the choice of an approach certain costs aspects and safety aspects are involved, because only chance of occurrence of a storm can be predicted, which implies a certain risk of failure of the cover layer. An optimum in the thickness of the clay layer, the risks and the costs should be determined.

12.2 Case Study: Het Verdronken Land van Saeftinghe

To get a rough idea of the practical use of the Semi quantitative model which was developed in this research, a Case Study was performed on the dikes sections at the ‘Het Verdronken Land van Seaftinge’. The amount of erosion will be determined with the semi quantitative model given in Chapter 11.

In this paragraph an example of the erosion development over time will be given for a clay layer of 50 years and 100 years old.

Description of the area
‘Het Verdronken Land van Saeftinge’ is located in the eastern part of Zeeuws-Vlaanderen (Number 1 in Figure 12.1). The hydraulic boundary conditions are given in Table 12.1 ‘Het Verdronken Land van Saeftinghe’ is located between dike pole number 71 and 79.



1 Het Verdronken Land van Saeftinghe
2 Hellegatpolder
3 Voorland Nummer Een

Figure 12.1 Locations of clay dikes in Zeeuws Vlaanderen, Zeeland.

Table 12.1 Hydraulic boundary conditions 2006 [www.hydraulischerandvoorwaarden.nl]

Hydraulic boundary conditions 2006 Het Verdronken Land van Saeftinghe	
Slope	1:4
Test level 2006	NAP+6.6m
H_s	1.29-1.77m
Design storm duration	30h
Norm frequency	1/4000

Semi quantitative model
The transitions between structured clay and moderately structured clay, and between moderately structured clay and unstructured clay can be determined with the graph of the soil structure development over time. The locations in depth of these transitions represent the transitions in the linear lines of the semi quantitative model. See Chapter 11 for more information on the model.

50 years
See Figure 12.2.

From the soil structure development appears the transition in clay conditions is located at a depth of 1.3m and 1.7m. The thickness of structured clay is 1.3m and of moderately structured clay 0.4m.

After a wave loading of 3 hours the erosion depth will be 0.8m-1.5m, after 10 hours 1.5m-1.8m and after 30 hours 1.8m-2.4m.

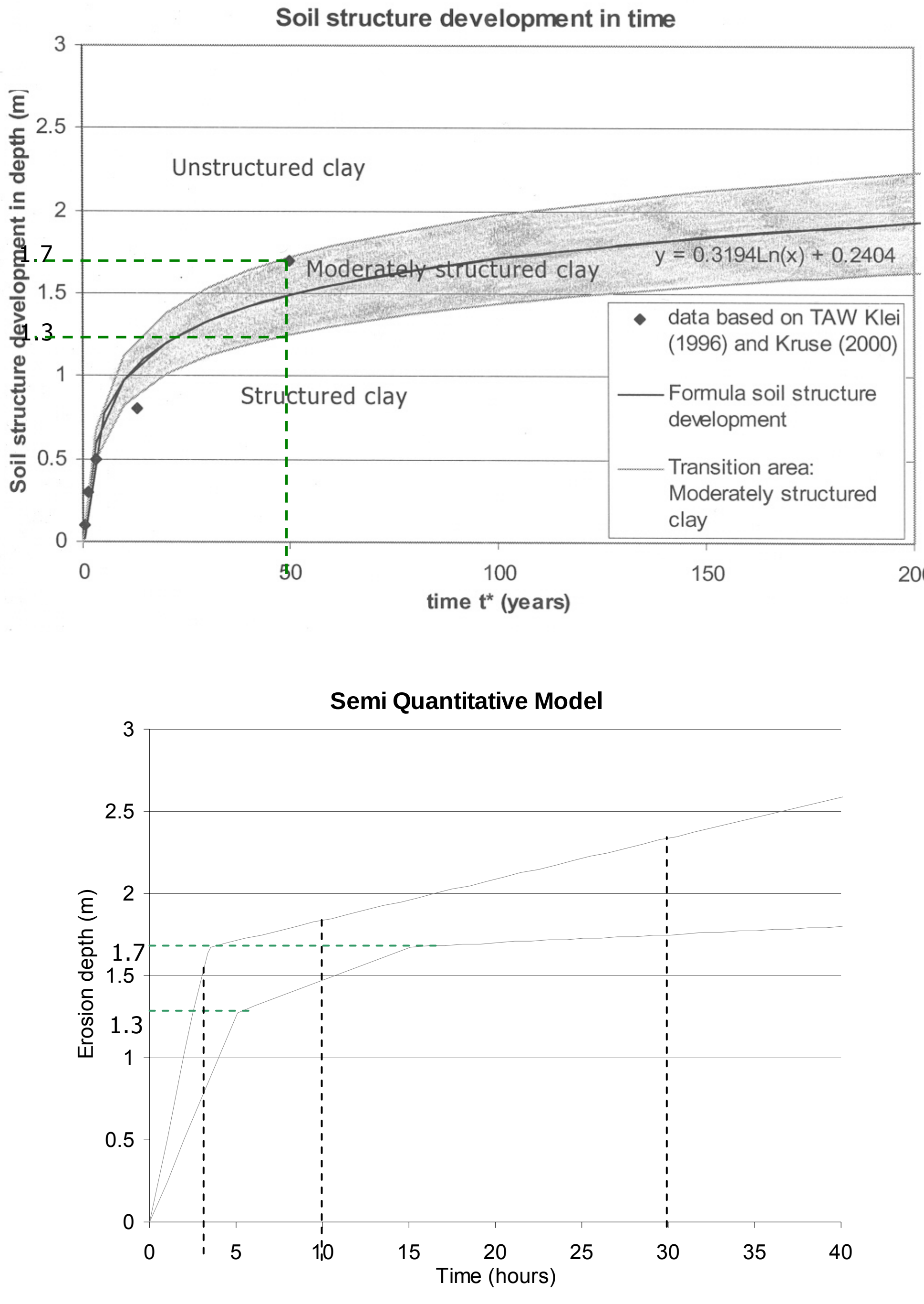
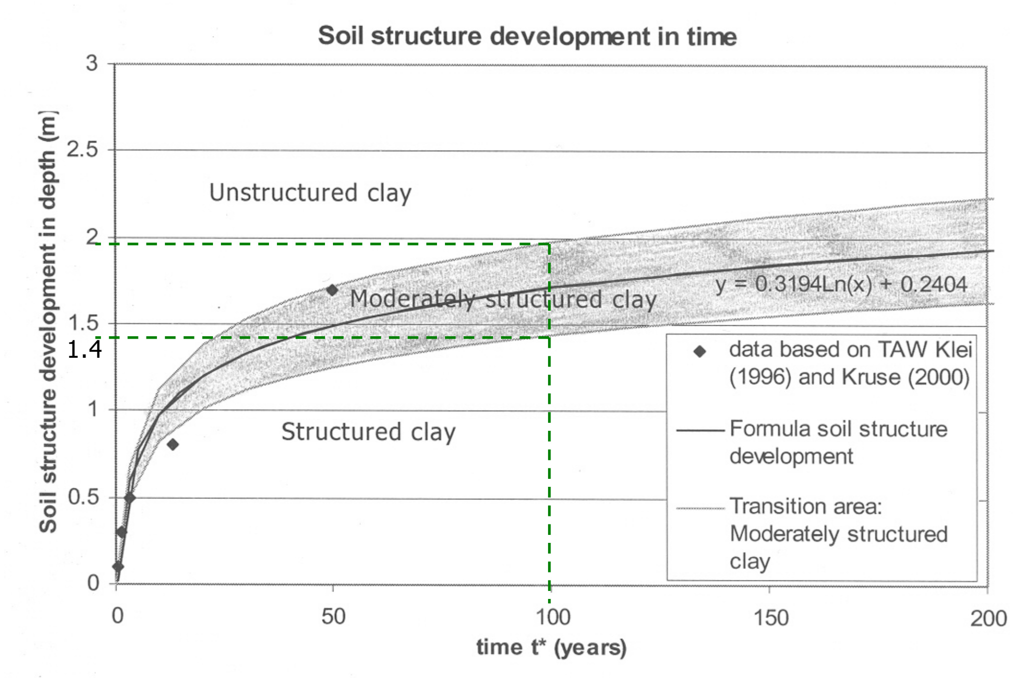


Figure 12.2 Determination of the erosion depth over time for Het Verdonken Land van Saeftinghe for a 50 years old clay layer.

100 years
See Figure 12.3.

From soil structure development appears the transition in clay conditions is located at a depth of 1.4m and 2.0m. The thickness of the structured clay is 1.4m and of the moderately structured clay 0.6m.

After a wave loading of 3 hours the erosion depth will be 0.8m-1.6m, after 10 hours 1.6m-2.2m and after 30 hours 2.0m-2.7m.



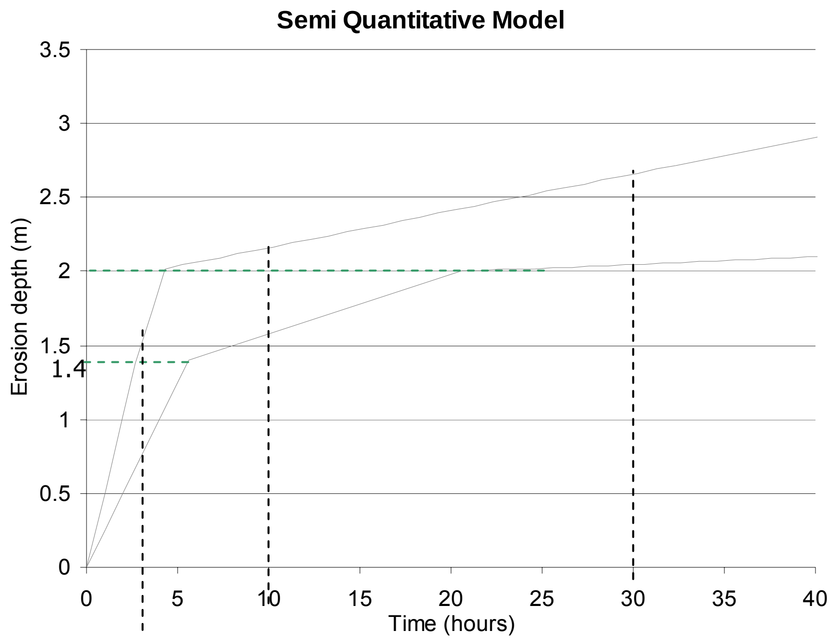


Figure 12.3 Determination of the erosion depth over time for Het Verdonken Land van Saeftinghe for a 100 years old clay layer.

Overview
An overview of the erosion depth of clay after different loading durations for different ages of the clay layer is given in Table 12.2.

Table 12.2 Overview of the erosion depth at 'Het Verdronken Land van Saeftinghe' for different hours of wave loading. Determined with the Semi quantitative model.

Age clay layer (years)	Erosion depth (m)		
	3h	10h	30h
50	0.8-1.5	1.5-1.8	1.8-2.4
100	0.8-1.6	1.6-2.2	2.0-2.7

The results of this Case Study have to be interpreted with care.

The semi quantitative model should not be used in practice.

The semi quantitative model is determined based on wave heights of 1.0m-1.5m. The outer line of the grey area in the semi quantitative model represents the wave height of 1.5m. The normative wave height at Het Verdonken Land van Saeftinghe is 1.29-1.77m. Therefore the results of this Case Study have to be interpreted with care. The range of the depth of erosion will be higher than determined with the model. However, because the available amount of data is limited and observations of actual storms with slightly higher wave heights give clearly more erosion than the experiment results, it is advised against extrapolation of the semi quantitative model for higher wave heights without further research. These results should only be used to have a first indication of the erosion depth over time.

12.3 Feasibility

In this paragraph the conditions for the feasibility of a clay cover layer as a revetment for the outer slopes of sea dikes are given.

12.3.1 Requirements for a sustainable and feasible solution

The alternative of a thick clay layer as cover layer for the outer side of sea dikes is not just a good solution if it is strong and stable enough to provide dike breaching during a normative load, a storm. There are other aspects that determine if the alternative will be a feasible and sustainable solution. An overview of these aspects is given in the schedule of Figure 12.4. These aspects also interact with each other. To keep the figure clear, this is not given in the schedule, but it is described in the list below. On all these aspects the alternative of the use of a clay cover layer on the outer slope of dikes can be compared to other types of revetments, in order to see if this alternative forms a good solution.

In order to obtain a sustainable and feasible solution, the design should fulfil several requirements, see Figure 12.4. All options have to be compared and deliberated on the following requirements:

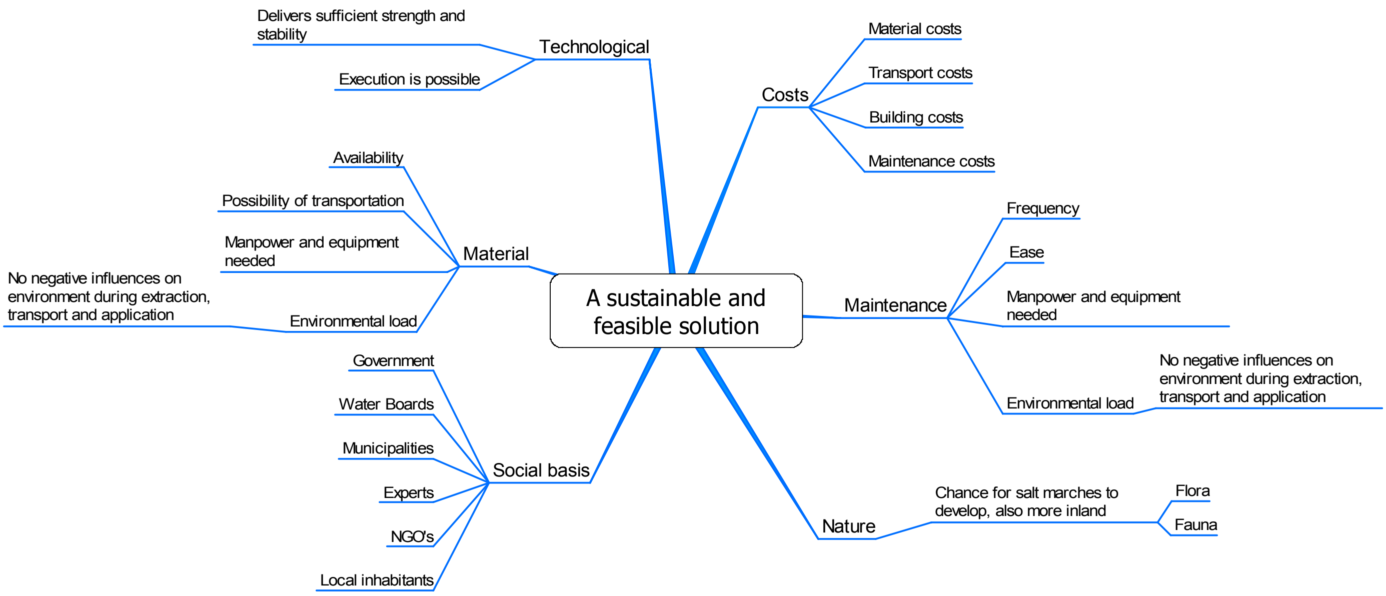


Figure 12.4 Overview of required aspects of a revetment to form a sustainable and feasible solution.

Technological possible
The solution should in the first place be technological possible. It has to fulfil the technical requirements, of amongst others sufficient strength and stability during a normative load, and the possibility to construct and execute the design.

Costs
The costs should be within an acceptable range. The total costs include material costs, transport costs, building costs and maintenance costs. For a feasible design of a clay revetment these total costs should preferably be lower to the total costs of traditional used revetments. The costs will be calculated for a several amount of years and therefore the costs comparison will count for this same amount of years. If a different amount of year is chosen, the total costs will be different as well.
It is preferred to look at lifetime costs, calculated back to annual costs.

Material
The material, the clay, needed for the construction if the cover layer should be available and it has to be possible to transport it to the building site. On the location where the clay is extracted, the

environment should not be negatively influenced. This has also to be taken into account. (The costs of the extraction, transport and building are included in the costs-aspect.) There should be limited negative influences on the environment due to the use of this material. This environmental load includes emissions during extraction, transport and application. Therefore a material which is obtained closer to the building location can be a better choice, because transport includes not only costs, but also emission of harmful substances, like CO_2 due to fossil fuel use.
In interesting side step is the use of modified dredging materials (SmartSoils®).

Maintenance
Maintenance of the construction is an important aspect, because the maintenance of a clay layer will be different from other revetments. The frequency of necessary maintenance will be important, because this implies the amount of people, equipment and load on the environment. If the frequency is higher, it can still be a good solution if the maintenance is easier to perform in comparison to the traditional revetments.
Also for maintenance material is needed, which involves again the aspect of material. Just as for the aspect ‘material’, the costs, needed equipment and manpower and the load on the environment are also concerned.

Nature and environment
The development of nature on forelands and the outer slope of the dike is a requirement for the sustainability and feasibility of the solution, because this project is initiated as an alternative which, in comparison to stone covers, allows vegetation development.

Social basis
In order to implement a solution a social basis is necessary. Government and the Water Board who spend money and the water defences have to approve the solution and can favour a solution of their choice. Experts and NGO’s can express their preference and opinion and try to influence the other parties. Also the parties more closely involved like the municipalities and local inhabitants can influence the decision making process.

The alternative has to be feasible and sustainable on the long term, 50-100 years.

Conclusion
As becomes clear from the listing above, obtaining a sustainable and feasible solution involves many aspects. All these aspects should be elaborated to come to a conscious decision on the feasibility of the use of clay as a cover layer for sea dikes.

12.4 Other applications

The results of this research project are not limited to sea dikes. It is also possible to determine if a thick clay layer can be applied on other locations. Ideas for other applications are listed below:

- River dikes: The wave loading on river dikes is lower, because the wave load is mainly caused by ship waves. (In this case more research should be done to the influence of currents on the slope.)

- Dikes along the Oosterschelde: Due to the storm surge barrier, the wave load at the dikes along the Oosterschelde is much lower. In and along the Oosterschelde natural an increasing number of wetland areas are made. Like on Schouwen-Duiveland between Zierikzee and Serooskerke. (Recreatieplan Zuidkust Schouwen, where nature development is combined with recreation.)

- Foreign countries, where clay is present: A thick clay layer could be applied if the wave heights are not too high and if clay is widely available and therefore cheap. (For example in Bangladesh or Vietnam.)

Figure 12.5 Dike along the Oosterschelde

- If the crest height of the dike is relatively low and the waves partly overtop the dikes, not all the wave load is located on the outer slope. In this case the wave load on the outer slope is reduced. If the overtopping is allowed, like in the ComCoast project, the wave load in the slope is reduced and probably the use of a clay revetment is more feasible. Research on the influence of currents on the inner slope is an actual subject of research.

- If a riff or bar is placed as a sort of breakwater before the coast, the load in the slope is lower and maybe a clay revetment is sufficient. (However, a storm is accompanied with high water levels, thus the breakwater should have a high crest height. This is probably from aesthetic point of view not desirable, because the crest could still be above water level.)

Part 5

Conclusions and recommendations

13 Conclusions and Recommendations

13.1 Conclusions

It can be concluded from the literature study undertaken that further research into the behaviour of clay under wave loading is necessary, before the use of clay in dikes could be investigated. This is because little is known on the behaviour of clay under wave loading and the factors that influence this behaviour.

All large scale experiments performed which involved clay under wave loading were investigated and compared in this research. The overview and comparison of these large scale experiments resulted in a broader understanding of the behaviour of clay under wave loading and highlighted different useful conclusions. Comparison of experimental results with the erosion behaviour of sand and observations of outer slope damage during actual storms provided additional conclusions and further validated conclusions from the experiments.
A semi quantitative model was developed as part of this research on the basis of the results of the analysed large scale experiments on clay. This model could be extended in the future if more influencing factors on the erosion process are quantified.

Conclusions on the behaviour of clay under wave loading and preliminary modelling of this behaviour are given in sections 13.1.1 and 13.1.2. Conclusions on the use of clay as revetment in dikes are presented in 13.1.3.

The conclusions drawn in this paragraph are based on all available, but limited amounts of data.

13.1.1 Conclusions on the behaviour of clay under wave loading

Research into the development of erosion

- The clay type, expressed as a sand percentage of the clay, is not the major influencing factor on the erosion rate of clay under wave loading. However, the influence of the clay condition on the strength of the clay under a wave load is significant. Unstructured clay has a significant lower erosion rate than structured clay.
  The clay condition is the amount of soil structure development in the clay, which develops over time and in depth based on several influencing factors such as the sand content, compaction, vegetation growth and animal digging.
  The clay condition can be divided into three conditions: structured clay, moderately structured clay and unstructured clay. The difference in erosion of structured and unstructured clay is significant. Moderately structured clay forms a transitional condition.

- Based on this subdivision in the clay conditions, the clay type and the wave conditions are of influence on the development of erosion. A higher sand percentage and a higher wave height result roughly in a higher erosion rate. The clay type is however more of influence than the wave characteristics.
  Unstructured clay with a low sand percentage has a very high erosion resistance.

- The breaker type, and thus also the slope angle, are probably of influence on the erosion process. A different breaker type results in a different load on the slope and therefore a different erosion mechanism.

- Clay has a significantly lower erosion rate than sand. Structured clay behaves more similarly to sand than unstructured clay.
  The erosion is measured perpendicular and horizontal to the slope, both give a different erosion development over time. The development of the erosion of clay and sand horizontally measured over time are similar, but the amount of erosion of clay is lower. For unstructured clay the horizontal erosion development over time is different and therefore the comparability of the determined horizontal erosion rate of unstructured clay is debatable.
    - The erosion of clay measured perpendicular to the slope is approximately a factor of 2-10 lower than that of sand.

- Horizontally measured, the erosion of structured clay and moderately structured clay is approximately a factor of 1.5-3.5 lower than that of sand.

- The experiment results indicate less erosion development over time than in reality is observed. Observations during the storm surge disaster of 1953 in The Netherlands indicated for dikes, horizontally measured, more erosion than observed in the experiments on sand. Measured perpendicular to the slope, the observed amount of erosion of dikes and dunes was approximately similar to the experiments on sand. The erosion of all dikes was developed through the clay layer into the sand cores of the dikes, but unexpectedly the observed erosion of the dikes was larger than that of the dunes. The erosion data of 1953 are however not accurate and the wave heights were generally higher than those used in the experiments.

- Several factors probably influence the erosion process of clay on dikes and are likely the cause of the difference in results of the experiments and actual damage. These factors include:
  - Weak locations present in a dike.
  - Sand and debris which will be present in the sea water during storm surges.
  - The salt in the sea water.
  - The wave conditions and breaker types in actual storm surges could be different from those used in the experiments.

- A grass cover has a higher erosion resistance against wave loading than clay. The amount of available data on grass under wave loading is however very limited. The erosion rate of unstructured clay with a low sand percentage is approximately similar to the erosion rate of grass.

- The empirical formula of the residual strength of clay under wave loading, which determines the erosion depth over time [WL|Delft Hydraulics, 2006b], fits well to the data it is based on. However, it delivers no good results when compared to the data of the other large scale experiments.

Research into the location of erosion

- The location of erosion is different for clay and sand. The erosion of clay starts below the water level by the development of a hole, approximately at a level between $0.3H_s$ and $1.2H_s$ below the water level. The hole can extend and develop till above the water level, if the erosion process has sufficient time to continue.
  The erosion of sand develops above the water level. This difference is probably the result of different erosion mechanisms of clay and sand.
  These conclusions are confirmed by the observations made in the storm surge disaster of 1953.

- The defined theory of erosion zones by Smith (1994a) does not completely correspond to all experiments.

Research into the shape of the erosion profile

- The erosion profile of clay has a steep slope, the shape of a cliff, if the initial developed hole has enough time to expand.
  In sand a steep cliff develops, which retreats in horizontal direction. The cliffs developed in clay are mutually more different and sometimes gentler than in sand.
  These conclusions are confirmed by the observations made in storm surges in history.

General conclusion on basis of this research

- On the basis of this research, it can be deduced that conclusions based on one or just a few experiments should be interpreted carefully.
  The empirical formula [WL|Delft Hydraulics, 2006b] as well as the theory of the erosion zones [Smith, 1994a] could not completely correspond to all other experiments.

13.1.2 Conclusions on modelling the behaviour of clay under wave loading

- A semi quantitative model to determine the erosion depth over time was developed in this research. This model is based on the determined subdivision of the experiments on clay in the three different clay conditions and a developed basic formula for the soil structure development over time and in depth.
  On the basis of the present available data of the large scale experiments, it is not possible to quantify the influence of the clay type and wave characteristics. Only an approximate influence of the wave height can be defined. The semi quantitative model was developed in a way that it can be extended if the influencing factors on the erosion process of clay are quantified.

- The semi quantitative model determines approximately the average erosion over time in comparison to the developed empirical model to the residual strength of clay by WL|Delft Hydraulics (2006b). The empirical model however appeared not to be applicable for all Delta Flume experiments on clay.

- The design method developed by the engineering firm INFRAM (2003) determines between approximately 3 and 10 hours less erosion than the semi quantitative model. During longer storm durations, the erosion of the design method is larger than the semi quantitative model, as expected.

13.1.3 Conclusions on the use of clay as a revetment on sea dikes

- On the basis of this research it can be concluded that a thick clay layer can possibly form a revetment for a sea dike, however research to date is not sufficient enough. The erosion over time can be determined with the semi quantitative model that was developed in this research. As far as this research demonstrates the wave load should not be severe (H_s=1.5m is maximum in used data). An unstructured clay condition has to be obtained as much as possible, such as by compaction during construction and the use of clay with a low sand percentage.

  However, before actually using a clay revetment, more research is needed into the differences of the experiment results with the observations in made during actual storms.

  A case study of Het Verdronken Land van Saeftinghe was made with the semi quantitative model developed in this research. This study showed that a 100 year old clay layer would erode approximately 0.8m-1.6m in 3 hours, 1.6m-2.2m in 10 hours and 2.0m-2.7m in 30 hours. (The semi quantitative model should not be used in practice.)

- Before actually using a clay revetment in sea dikes, the feasibility and sustainability of this alternative revetment should be elaborated further upon the aspects of technological feasibility, costs, maintenance, availability of material, chances for nature development and obtaining a social basis. It is also possible to determine if a thick clay layer can be applied in other locations, for example river dikes.

- Clay cover layers as a revetment for sea dikes are occasionally used in Zeeuws Vlaanderen, Zeeland, The Netherlands, if a high foreland is present in front of the dike.
  The concept of a clay revetment was monitored for a few years at two test sections at Het Verdronken Land van Saeftinghe. During this period no erosion took place and the vegetation in the clay was well developed. However, also no storms occurred during the monitoring. Based on the monitoring and due to a lack of more data, the use of a clay layer was considered by Rijkswaterstaat to be a suitable revetment for sea dikes if a high foreland is present in front of the dike. This alternative revetment therefore, is also applied at other locations in Zeeuws Vlaanderen where a high foreland is present.

- A design method for clay revetments was developed by the engineering firm INFRAM (2003), which formed the basis of the current design manual used by Projectbureau Zeeweringen. Both methods are considered to be unsure.

13.2 Recommendations

13.2.1 Recommendations on further research into the behaviour of clay under wave loading

In order to make a better description of the strength of a clay layer under wave loading, more accurate conclusions are needed. The conclusions drawn in this research need to be validated and a more accurate model of the erosion development over time and depth needs to be developed.

- More research is necessary in order to make a better description of the strength of a clay layer under wave loading. More information on the differences between the erosion behaviour during experiments and during actual storms should be obtained.
    - The data from the experiments on clay and especially the data pertaining to erosion during storms are very limited. More data are necessary, which can be obtained by performing experiments. Large scale experiments in a wave flume can give useful results, but an experiment on a real dike would be even more valuable. A realistic soil structure development especially is difficult to obtain in a laboratory.
    - Results for higher wave heights than used in the Delta Flume experiments (H_s>1.5m) need to be obtained. These results should also be integrated in the semi quantitative model. In actual storms the wave heights are generally higher than 1.5m.

- The influence of the different influencing parameters on the erosion process needs to be quantified.
  This research has determined that erosion is dependent on several influencing factors, but quantification of this influence is not possible on basis of the available data. Therefore research into the specific influence of parameters such as the wave height, wave period, slope angle, the breaker parameter, clay condition and clay type on the erosion process should be performed. This can be done by experiments where one parameter varies, while keeping the other parameters constant.
  The results can then be integrated into the semi quantitative model.

- More information on soil structure development in clay over time needs to be obtained.
  This concerns more information on the influencing factors on the soil structure development and the quantification of these influencing factors. If a more accurate description and a quantification of this development are obtained, the developed basic formula for the soil structure development over time and in depth, developed in this research, can be adjusted.

- The wave characteristics and breaker type probably have a large influence on the erosion process and therefore, this needs further research.
  A different breaker type, thus a different Iribarren number, could result in a different load being applied to the slope, and therefore into a different erosion mechanism.
  If the waves are of a plunging breaker type a cliff shape develops. Most experiments are performed with plunging breakers. This breaker type results in the highest wave load on the slope, a wave impact. If the load is not an impact load, but more a run up and run down on the slope, the erosion mechanism could be different, for example be more like a scour effect. The critical shear stress could form the start of erosion.
  More research should be performed on erosion due to loading due to currents on the slope. The experiments performed by Van Meerendonk (1984) could be a part of this recommended research. The wave loads of the Delta Flume experiments on wave loading, used in this research, could be rewritten to maximum flow velocities on the slope and compared to the results of loading by currents.

- The distribution of weak locations along the slope and the properties of a weak location in relation to the strength of the rest of a clay layer should be researched.
  For an accurate description of the erosion process it is important to know the influence of weak locations, because the erosion of a clay layer will start at the weakest location on a dike slope.

- The influence of sand and debris in the water during storms will influence the erosion process of the clay. The erosion will probably be larger than for clean, fresh water, because the particles and debris can induce forces on the clay slope and cause scouring. This could increase the erosion process or initiate erosion by the development of holes. It also could function as bulkhead after deposition in front of the dike. Research is necessary to determine this influence on the erosion process.

- The influence of salt in the sea water on the erosion process of clay should be studied. Clay consists of minerals such as quartz, iron, aluminium compounds and chalk, which could make chemical bonds with the salt ions in the sea water (main salt ions are chloride and sodium). The Delta Flume experiments are performed with fresh water and could therefore cause a different erosion process.

- Research on the influence of oblique incoming waves is needed. In the Delta Flume the waves are approaching the dike perpendicular, in actual storm surges the waves will often approach the dike at an angle. This could result in a different wave load in the slope. The influence of oblique incoming waves on the erosion development of clay should be further studied.

- The influence of water pressures on the dike during wave loading could be of influence on the erosion process and therefore need research. Analysis by Hofmann (1993, 1995) gave no satisfying results.

- More research should be performed on the erosion resistance of vegetation under wave loading and the strength of the root system. This research shows that grass has a high erosion resistance. Vegetation always will be present on top of a clay layer and could therefore deliver additional strength to the revetment. If more information on the erosion development of grass is obtained, this can be combined with the erosion development of clay in the semi quantitative model (Included as the first step in the erosion process).

13.2.2 Recommendations on the use of clay as a revetment

- Before using clay as a revetment, more research should be performed on the behaviour of clay under wave loading.
  Apart from this technological aspect, also the feasibility and sustainability of this alternative revetment needs further research. Based on the following listed aspects this alternative revetment of a thick clay layer should be compared to traditional revetments:
  - Technological: More accurate information should be obtained on the strength and stability of clay under wave loading and its stability if used as a revetment.
  - Material: The availability of clay.
  - Nature: More information on the possibility of vegetation growth in the clay revetment should be performed.
  - Costs: In a costs benefit analysis both construction costs and maintenance costs should be included.
  - Social basis: A social basis for the use of this revetment should be gained.

- Other applications of the use of a thick clay layer as a revetment can be investigated. For example its use in river banks, where the hydraulic loads are less severe.

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Appendices

I. The history of flood defence in The Netherlands

The Netherlands has a large tradition of living with water. This owes us a great part of our prosperity, but also many flood disasters have passed. Till the Middle Ages the population conformed themselves to living with water and nature. They lived on the higher dune areas, natural bank walls of rivers or terps. The regular floods did not have many negative effects, even a positive effect, because due to of the deposition of silt, the land was able to keep pace with the naturally rising sea level.

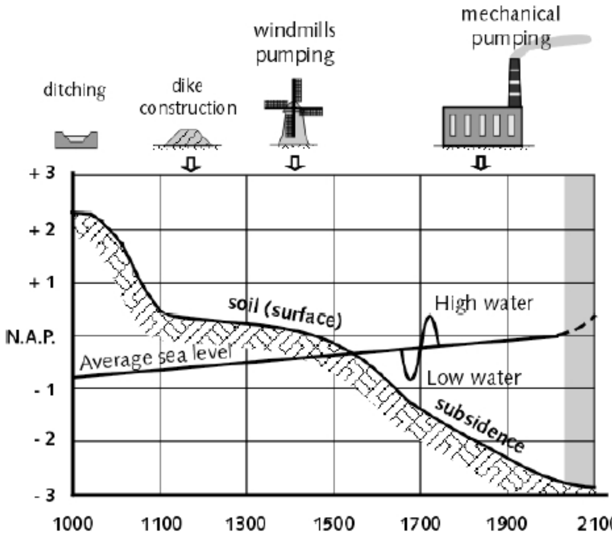

Figure I.1 The subsidence of peat areas in the Netherlands [TAW GvW, 1998]

In the course of time an increasing population growth resulted in an intensifying use of the lower lying areas. These areas were partly drained for agricultural use and partly excavated for peat, which was used as a fuel. As a consequence, the surface subsided and the areas became more vulnerable to flooding. Therefore dikes were constructed, at first only as an addition to the height of existing higher lying areas, but intensive use of the lower areas and the development of drainage techniques resulted in the building of more dikes. The extreme water levels increased along the rivers and the deltas, because the water retaining areas disappeared. The relative sea level rose, and the combination of natural sea level rise and the subsidence of the ground, lead to the fact that large parts of the Netherlands became lying under the sea level. The higher water levels in combination with an increasing population growth resulted in increasing risks of flooding, which emphasizes the increasing importance of the water defences, including the dikes. (See also Figure)

The vulnerability of the Netherlands was painfully emphasized during the storm surge disaster of February 1953, when the western part of the Netherlands was flooded due to dike breaches. This lead to many deaths, casualties and damages. Before 1953, the dikes were raised after every flooding, making the dikes a bit higher than the highest known water level. After the flooding of 1953, questions were raised about how to deal with the safety against flooding and a different design philosophy was developed by the Delta Committee (in 1960).

The Delta Committee introduced requirements based on a design water level with a certain exceedance frequency. From an economic analysis for Central Holland it was determined that a water level with an exceedance frequency of 1/10,000 per year had to be completely retained. This water level is the design water level, the Normative High Water (MHW) level. This was considered to be true when a maximum of 2% of the waves running up the dike exceed the crest. This safety standard, the design water level and the accompanying exceedance frequency, varies for different areas depending on their economic value and form since the core of the safety requirements concerning flooding.

The River Dikes Committee (Becht Committee) applied the same approach in 1977 for the upper river area. They recommended an exceedance frequency for the upper river areas of the design discharge of 1/1250 per year, because inundation by fresh water results in smaller damage and the loss of landscape, nature and cultural heritage (LNC-values) played a role. Later, in 1993, the same standard was recommended by the Boertien Committee (but with a different design discharge of the river Rhine).

These standards, based on a combination of economic factors and the loss of LNC-values are laid down by the government in 1995 in the Flood Defence Act (‘Wet op de Waterkering’). The exceedance frequencies are ranged from 1/10,000 for the densely populated areas in the West, 1/4000 for the other parts of the coast, 1/1250 for the areas along the main rivers and 1/2000 for the intermediate areas. See also Figure. In the course of years with increasing technological knowledge, also safety requirements including testing methods, of other failure mechanisms of the flood defences have been developed, also for revetments. [TAW GvW, 1998]

Results of studies by the Ministry of Transport, Public Works and Water Management as an extension of the work of the Delta Committee are that a new safety approach should be based on the probability of inundation of a dike ring area instead of the probability of exceedance of a certain water level. A dike ring area is an area protected against floods by a series of water defences and/or high grounds. This method implies the fact that the consequences of an inundation (casualties and damage) and thus the flood risks, differ from place to place and that this must be expressed in the dimensions of the dike ring elements. This safety method is not only based on overtopping, but all failure mechanisms of the water defences are taken into account. To come to this more uniform judgement method for the water defence structures the ‘Marsroute’ was started, which has continued into the project ‘Veiligheid Nederland in Kaart’ (VNK). The current safety assessments are based on this approach. [TAW GvW, 1998, TAW vOnO, 2000]

At the moment sea level rise and its consequences for the capacity of the water defences are an important topic of research. Also new insights about the hydraulic loads on the coast, increasing storm frequencies and larger wave periods, imply heavier loads on the water defences than was known so far. This needs adaptation of the present knowledge and adjustments of the design methods.

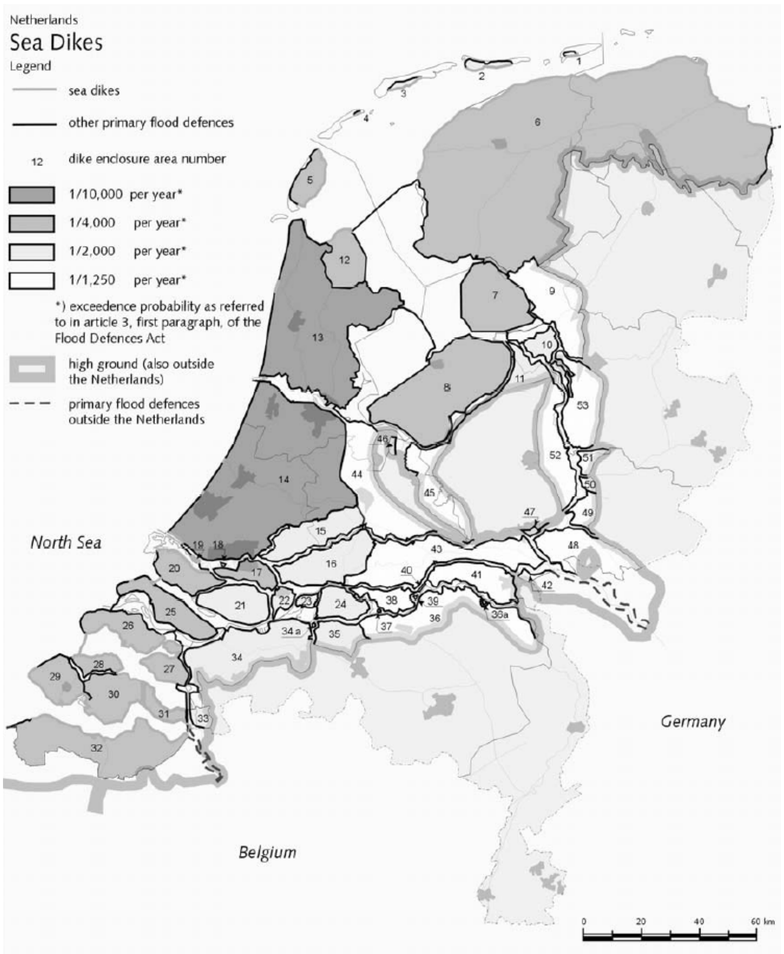

Figure I.2 Sea dikes in The Netherlands and the safety standard per dike ring area [TAW LZM, 1999]

II. Modelling the erosion of grass

Empirical behaviour and damage model

On basis of the large scale model experiments of the Delta Flume in the 1992, DF1992G, and the Schelde basin, 1994, a temporary empirical formula is determined between the wave height and the erosion velocity of the grass sods. On basis of theoretical grounds is assumed that the erosion velocity increases with the square of the wave height. The quality of the grass sod is expressed by a proportion constant, $c_{E.}$.

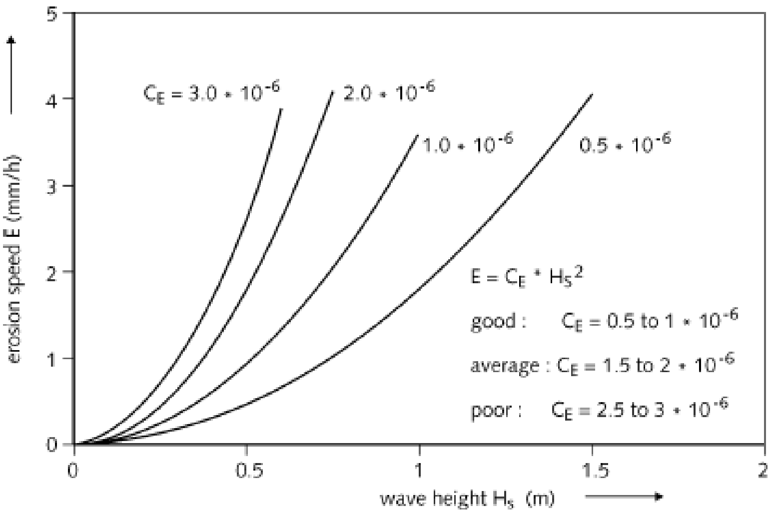

Figure II.1 Rate of erosion as a function of the wave height [TAW Erg, 1998]

$$E_{grass} = c_E H_s^2$$

E_{grass} erosion rate (m/s)
c_E grass erosion coefficient ($m^{-1}s^{-1}$)
H_s significant wave height (m)

The values for the grass erosion coefficient are obtained from the results of the large scale experiments.

Table II.1 Expected values for the grass erosion coefficient (c_E)

Erosion-resistant grassland	Expected values for c_E ($m^{-1}s^{-1}$)
Good	0.5 to 1.5* 10^{-6}
Moderate	1.5 to 2.5* 10^{-6}
Poor	2.5 to 3.5* 10^{-6}

This relationship can be translated into a permitted duration of the wave load for a certain wave height, thickness and erodibility of the grass sod.

$$t_{\max} = \frac{d}{\gamma c_E H_s^2}$$

t_{max} maximum permitted duration of loading (s)
d thickness of the sod (m)
γ safety coefficient (-)

This relationship is supported in by tests by Hewlett et al. (1987) to the loading of the grass by overflowing water.

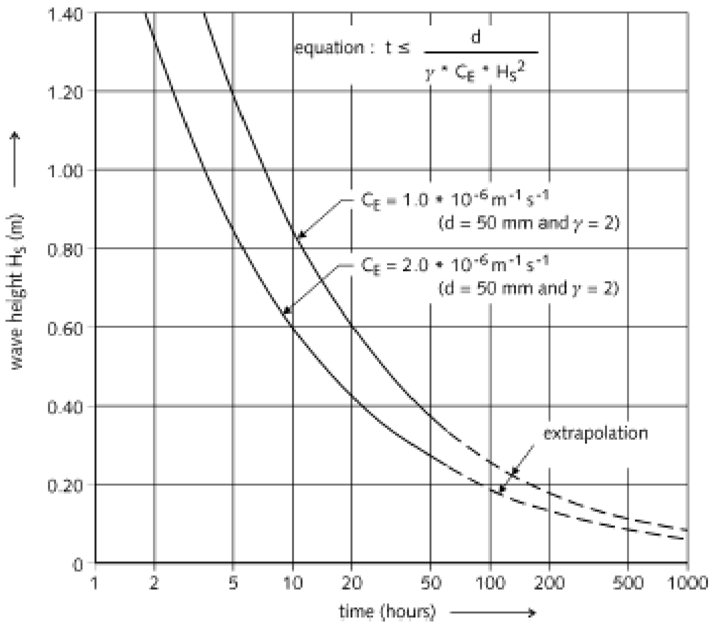

Figure II.2 Maximum permissible duration of wave attack [TAW Erg, 1998]

There are however certain remarks to be made on this formula:

- It is tried to relate the grass erosion coefficient to the quality of the rood system and the soil between the roots, which makes the coefficient dependent on the density of the roots, the roots length and the amount of parts which can be elutriated. No unambiguous result is jet obtained.
- Including the wave period and the slope angle in the model was not succeeded, because the difference in the experiment results was small.
- The relation with the results of the small scale erosion tests in the field with spray head and erosion centrifuge tests is not clear. There is however a difference in the results of the erosion centrifuge tests on bemest hooiland with high root density and bemest weiland with low root density.

Set-up of improvements
An improved model will be obtained if within the modelled erosion process a distinction is made in the separate constituent processes:

- Washing away of the loose particles
- Failure of the roots
- Sudden, or gradual, cracking of the sod.

In this case disconnection of the load and the strength is possible and the effects of the density and the strength of the roots, soil, slope and wave loading parameters can be modelled independently. From these parameters the influence on the erosion rate can be determined. It is important to include in the model the fact that roots can bind the aggregates dependent on the intensity of the wave load. There is also a difference between the strength of the individual roots and the total strength of the grass sod.
[TAW Erg, 1998]

Loading model by Hewlett (1987)

Up till now little attention is paid to a load model. In 1987, by Hewlett et al. the water velocities are assumed as the load parameter. After measurements at the large scale experiments by Delft Hydraulics is however not possible to translate the irregular breaking waves into water velocities or shear stresses.

For the wave run-up a reduction coefficient (r) can be used for the roughness of the slope, 0.55 to 0.65 for small waves to 0.9 to 1.0 for larger waves. For waves smaller than 0.75m this reduction coefficient is dependent on the wave height, because the wave layer thickness on the slope is smaller, which implies a larger hydraulic roughness.

$$r = 1{,}15 H_s^{0{,}5}$$

r reduction coefficient
H_s significant wave height (m)

[TAW Erg, 1998]

Maintenance and management

The density of the vegetation and the root system of a grass cover are influenced by the method of management, with the amount of fertilizer and level of maintenance as the critical factors. According to research by Sprangers (1996) the best sod quality for a dike cover is obtained during the following extensive (in Dutch 'extensief') management methods:

- Hay-making without fertilisation (hydraulic management and natural management)

This method produces the highest root density, till 0.15m deep a closed root package with thin and thick roots, and a reasonably closed vegetation (see Figure II.3). The natural value of the grassland becomes the highest with this method.

- Sheep grazing and light fertilisation (max. 75 kg N/ha) (adapted farming management)

With this method a good closed grass cover with a closed root system develops, however it consists mainly of thin roots in the upper 0.10m of the soil (see Figure II.3 C). This method requires an accurate management plan adapted to the local situation, because the sheep grazing is not aimed at sheep rearing but on obtaining an erosion resistant grass sod. The animal density and grazing periods must agree exactly with the rate of crop production.

Intensive fertilisation results in vegetation with open patches and a low root density and also leaving the cutting after rough mowing twice a year results in open patches and thus a low erosion resistance. Lawn management, very regularly mowing through which the amount of mowed vegetation is smaller, results in less open patches, but also results in a shallow root system concentrated in the upper 5cm of the soil.
[Sprangers, 1996; TAW ErG, 1998; TAW Grasmat, 1999]

In practice it is very difficult to obtain a good quality grass cover, even with the best extensive management methods. Weak locations in the sod are mainly placed just above the berm of the outer slope and at these locations the test criteria of the Voorschrift Toetsen op Veiligheid (TAW, 2004) are rarely fulfilled. This extensive management is also difficult to execute, because large dike parts, mainly in Zeeland, are leased out to farmers. The farmers consider the dike more as an extension of their pasture than as a water defence. Therefore it is difficult to put them up to maintenance. In Friesland better results are obtained, but there the dikes are not leased out to farmers and the maintenance is executed by the Water Board itself.

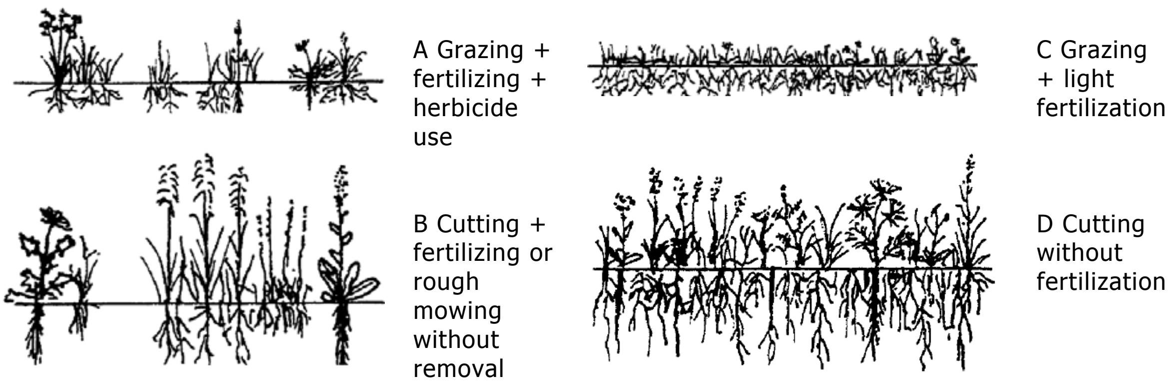

Figure II.3 Type of grass land during different management methods [TAW Grasmat, 1998]

III. Overview experiments on grass covers

An overview of large scale experiments on grass covers with the accompanying references is given below.

1) Flow resistance of a grass cover, 1970
Jong, de, *Stroombestendigheid grasmat*, WL, M1065, aug. 1970

2) Oostelijk Flevoland (Ijsselmeer dike), June – July 1970
Dike experiment: flow over the dike, also with a hole in the grass cover.
Laboratory test: flow on smaller samples and residual strength of the clay layer at flowing water.

Jong, de, *Stroombestendigheid van een grasmat op de dijk van oostelijk Flevoland.* Waterloopkundig Laboratorium, R603, 1970a

3) Lith, July - August 1983
Simulated wave runup

Van Meerendonk, *Erosiebestendigheid van gras op klei taluds. Verslag modelonderzoek.* Waterloopkundig Laboratorium en Grondmechnica Delft, M1930, CO265412/15, 1984

4) Delta Flume Delft Hydraulics, October and November 1983
Strength of the outer slope of a grass cover during a storm.
Erosion resistance of clay, after making holes in grass cover.

Burger, A.M., *Sterkte van het buitenbeloop van een " groene" dijk tijdens een superstormvloed.* Waterloopkundig Laboratorium, M1980, 1984.

5) Delta Flume Delft Hydraulics 1992 (Seadike from Friesland)
Erosion, overflow and residual strength during high and low waves.
Including grass cover.

Smith,G.M., *Grasdijken, graserosie, reststerkte en golfoverslag, Meetverslag grootschalig modelonderzoek.* Waterloopkundig Laboratorium, H1565, 1994.
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6) Schelde Basin Delft Hydraulics, maart –juni 1994
Low waves (Hs=0.3m)

Verheij, H.J, D.G. Meijer, G.A.M. Kruse, G.M. Smith, M. Vesseur, *Onderzoek naar de sterkte van graszoden van rivierdijken.* Waterloopkundig Laboratorium, Q1878, 1995.

IV. Overview of clay properties

Introduction

In NEN 5104 clay is defined as a natural soil with a composition based on the mass percentages lutum, silt and sand. Natural soil consists of erosion and decomposition products of natural stones which are assembled by natural processes. The fraction borders for the different components are:

Table IV.1 Fraction border clay.

Fraction	Equivalent grain size
Sand	2 mm - 63 μm
Silt	63 μm - 2 μm
Lutum	≤ 2 μm

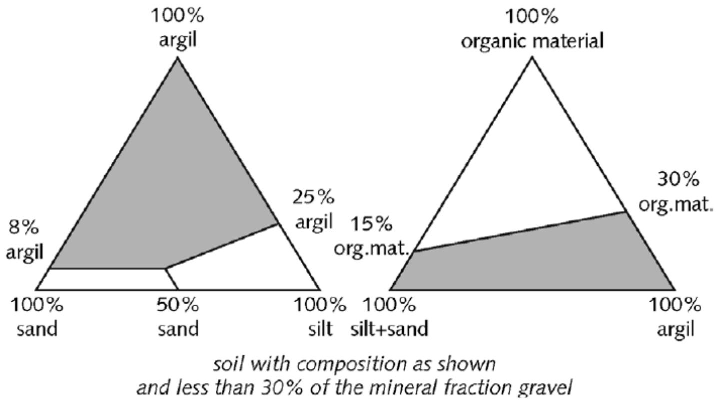


Figure IV.1 Classification of clay, according to NEN 510 (Argil=lutum). [TAW Klei, 1996]

The civil engineering properties of clay, like the permeability and shape retaining can differ in different qualities of the clay, while the grain size distribution can be around the same. The civil engineering properties of the clay change under environmental influences, like the change of ground water level, dehydration, etc. The properties of just excavated plastic grey or blue clay differ from the stiff, lumps of agricultural soil which can originate from this.

In the description if the clay properties a distinction can be made in the properties of the material as a mineral soil component and the properties of clay as a natural soil.

Mineral soil component

The aspects of clay on microscopic and sub-microscopic scale are or of interest for a description of clay as mineral soil component. The properties are obtained from chemical and associated physical phenomena.

Composition

Clay consists of particles solid soil, water with solutions and gasses. The characteristic properties of clay are obtained by the fine fraction of the solid soil particles and the water. The fine particles consists of different minerals and organic materials. In the Netherlands these minerals consists almost always for a great part of clay minerals, thin sheet shaped particles (thinner than 10^{-8} m) and other minerals like quartz, certain irons, aluminium connections and limestone. The organic materials are present in the form of vegetative and animal organisms, fibres, active bacteria, fungus and organic molecules.

The composition of the fine soil changes under the influences of the environment. The grey/bleu clay changes in yellow/brown clay by changes of the mineralogical composition (change of iron-, manganese connections.) Due to this variation in relative amounts of solid soil particles, different properties of the clay develop, like the water retaining capacity and the strength of the connections between the solid soil particles. The mineral composition has thus an influence on the civil engineering properties of the clay, like the shape retaining and erodibility.

The nature and amount of organic matter also changes. The amount of rough and fine organic remainders decrease by the change by micro-organisms. This change is depended on the amount of air which can reach the soil and the temperature. If it is sealed airtight and the temperature is less than 10º to 15º this process develops slowly. Because of this the organic remains which develop in the upper layer as a result of roots, fungus and compost are decomposed within several years. Just below the grass sod develops a dynamic balance due to the supply and change of organic matter, with a total amount of organic matter of 3 to 5%. The amount of organic matter has influence on different properties of the clay, like bulk density, water retaining capacity and changing properties.

Next to this is also the specific surface of influence on the properties. The specific surface is the total surface on the outer side of the solid particles. This is large due to the composition of small particles, in the Netherlands this is between 40-120m^2 per gram soil. (For sand this is <1m^2 per gram soil.) There is thus a coherence between the specific surface of the soil and the grain size distribution.

Water-retention capacity

Many properties of clay depend on the water content and its changes. The amount of water which is held by the clay depends on the physical-chemical properties of the water and the attraction of the environment.

The surface of the clay particles has affinity for water and therefore connect the water particles themselves to the surface. The affinity is dependent of the size and composition of the solid soil particles and of the matter which is soluted in the water and adsorbed to the solid particles. During long lasting dry periods only some water molecules are distributed over the surface. The presence of salt results in a higher water retention capacity. Water is also retained b surface tensions, which is a result of the attraction between the water molecules and the surface of the solid soil. Due to this water retention can also be done by fine pores and small corners.

The amount of water which can be retained by the clay under different circumstances depends on the water retention capacity and the suction pressure. The suction pressure is the water pressure in the clay above the freatic plane and is most of the times negative in relation to the atmospheric pressure. (The water pressure can only become locally positive as a result of precipitation or infiltration. As a result of this negative pressure, the clay can suck water from the freatic plane. The suction pressure is determined by the dynamic balance between the gravity force potential, the location above the freatic plane and the shape and size of the pores. Also evaporation to the atmosphere, directly and via vegetation plays a role. This evaporation is amongst others dependent on the relative humidity.

For a surface of a dike the suction pressure in summer is more than 100m water column due to a relative high temperature and suction force of the vegetation. However, due to precipitation and changes in temperature this can differ heavily. In winter the circumstances are wetter, and therefore the suction pressure lower. The largest variations are present in the root layer by the different water extraction, precipitation and large temperature differences.
A representative value for the suction pressure for grass sods in the Netherlands in summer is 10m. This also counts for clay under stone revetments, however less variation occurs in suction pressure. In the core of the dike variations are a result of a change of the position of the freatic plane and by atmospheric influences. These variations are slow and limited in size, because the atmospheric influences are limited. The result of the difference in suction pressure is a continuing transport of moist through the dike body.

Table IV.2 Suction pressure in clay at different circumstances, in meters water column.

Part of the dike	Summer	Winter
Surface (outer 1-2 m)	Large variation, >100m tot < 5m (during rain), average 10m	<1m
Core	0-5m	0-5m

The physical-chemical properties of the water and the solid soil particles are important for the water retention capacity of the clay.
The aquiferous capacity of clay at suction force larger than 5m water column is mainly stipulated by the properties of the area of the fixed substance. The size and the form of the pores are then relative unimportantly and there are in that case thus a consistency with the Atterbergse limits (flow and plastic limits). These limits are a measure for plastic properties of the soil.

The flow limit is the transition between behaviour of clay as a stirred cohesive soil to a plastic state. When the water content is lowered of the saturated clay, the solidity of the clay increases, this is because the clay particles come closer to each other. The clay will thus behave more like a solid. The water content when this transition takes place, the flow limit, depends on the binding of water to the clay which is dependent don the composition of ions and molecules in the clay. Because civil engineering properties of clay are directly linked to the physical-chemical properties, the flow limit is a useful classification tool.
The plastic limit stands for the water content at which the clay can just plastically deform. (the plastic limit is relatively low in soils which particles are smaller than 10-20 µm.)
The difference between the flow limit and the plastic limit, gives insight into the amount by which the clay is sensitive in differences in water content, the plasticity index (I_p).
$I_p = w_l - w_p$
(Clay with a high sand percentage has a low flow limit, and most of the times also a low plasticity index)

The maximum quantity of water which can be held by clay can be estimated using the consistency index(I_c). This index indicates the proportion of the water content in clay of the flow limit and the plasticity.

$$I_c = \frac{w_l - w_n}{w_l - w_p} = \frac{w_l - w_n}{I_p}$$

The consistency index is a good approximation for the workability of clay. (A water content in the vicinity of the plastic limit indicate low plastic possibility of deformation. The consistency index is then high. A low consistency index corresponds to a relatively high water content and large possibility of deformation).
From observations becomes clear that the following can be used:
Maximum water content of clay at a suction force of 10m water column, lining dike in summer: I_c = 0.75 maximum water content of clay in core of dike, largely above freatic area: I_c = 0.6.

Cohesion
The cohesion of the clay is determined by the affinity of clay for water and the adherence of the water molecules to the dissolved matter. Due to the agility of the water molecules the cohesion remains, as a result of this adherence. For clay types with a high plastic limit with respect to the flow limit and thus a relative low plasticity index, a large water content does not contribute to the cohesion of the soil. For a decrease of the water amount and thus an enlargement of the plasticity, enlargement of the cohesion, because the particles of slid soil become lying more closely. Further dehydration results in a decrease of the plasticity.

Next to this the cementation connections, the direct connection of minerals and organic matter, contribute to the cohesion. These connections develop after several hours to years by mineral and organic matter which are connected to the surface of the soil particles. The results in relatively strong connections, which makes the soil stiff to firm, but which can easily be broken.

Natural soil

When considering the properties of clay as a natural soil, this concerns mainly civil engineering properties such as, water permeability, erosion resistance and shape retention. These properties can vary a great deal after application of the clay.

Soil structure
By changing the water content in clay it shrinks and expands. This is caused by suction force. Clay in the unsaturated zone thereby undergoes volume changes due to changes in suctions pressure. The relative volume changes in this sort of clay is about half the change in water content (expressed in mass percentages).

The shrinking and expanding of soil in the unsaturated zone is linked to the formation of cracks. Crack formation produces a soil tat consists of aggregates of various dimensions. The composition of these crack and aggregates, together with pores and aggregates made by animals, is called the soil structure.

The soil structure can be more or less clearly developed. Such a marked structure develops from continuous movements duet o expending and contracting, or from a single very strong shrinkage, after the large splits occur they are not filled up again. Rapid changes in water content, e.g. due to rainfall , cause many small cranks and therefore a fine soil structure.

Mainly the upper few decimetres (to 0.8m) under the grass sod of a revetment, a dynamic development takes place of continuous formation of new aggregates which are affected.
In the uppermost decimetres under the grass sod on a dike bank the structure is usually very strongly developed and consists of relatively small aggregates with dimensions of millimetres to centimetres.
At greater depths under the grass cover and in clay layers under stone settings , the aggregates are often less clearly recognisable, with relatively large dimensions (10cm), which often still have some cohesion. In clay that becomes wet regularly, under the influence of the tide, such as clay under stone covering below the high-water line, a soil structure is formed only in the uppermost centimetres to decimetres of the clay and the structure is often less visible. After 5 years a recognisable soil structure has been developed under the stone settings, which after 10 to 15 years has developed through the whole area.

When clay with a higher water content is brought above the water table, the water content will decrease until it reaches equilibrium with the suction pressure in the surroundings. The clay will therefore shrink once only together with the formation of large cracks and a decrease of volume. In the upper soil, these crack do not last long due to homogenisation b burrowing animals and root activity. Deeper under ground these cracks will remain.

It should be clear that the presence of soil structure is a major influence in the water permeability of clay layers. A top layer of clay will have a considerably greater permeability than tests based on clay samples have shown. Through different heightening of the dike various layers are created. This increases the permeability also deeper in the dike, which is enhanced by worm tunnels etc. deeper in the dike. A network of pores which through the surface water can infiltrate and a large amount of water can be transported. The soil structure formation is favourable and often even essential for vegetation and other soil life.
Sand inclusions in a clay package lead the same affect as the cracks in the soil structure. The composition of a clay layer package is broken up by sand inclusions, and the permeability is also many times greater than of the clay itself.

Accounted should be that the civil engineering properties of the clay with a soil structure can vary a great deal with the properties of individual aggregates of the total layer. This also applies for the results out of the clay samples.

Micro structure
The microstructure is the spatial composition of individual small and large soil particles, the water content is of influence on the composition. By decreasing the water content of blue/grey clay or mineral dredging slurry the particles come to lie more densely to each other. By this dense packing, the binding between the particles is very large, which results in the fact the clay is strengthened. This impact is also reinforced by the presence of cementing substances. By a repeated change of water content the composition of the particles becomes very dense and clay becomes very firm. After some years the shear strength of multiple MPa in dike linings (c_u-value with laboratory paddle test) in the unsaturated area of the core this can run up to more than 100 kPa. Finely grained mineral dredging slurry can become within 2 up to 4 years to firm clay transformed by putting in contact it with the atmosphere. The changes which occur in the mineral dredging slurry are often called 'ripening'.

Permeability
As mentioned earlier the permeability increases strongly by cracks and worm tunnels, and it decreases by compacting of the layers. The permeability of well compacted clay covering on a dike was less than 10^{-6}m/s, but after about months the permeability increases to 10^{-4} en 10^{-5} m/s. This value account for the greater part of the dikes.

Shape Retention
The shape retention is manly of importance for the clay that is worked in thick packages in the core of a dike. The firmness of a package of clay directly after application depends very closely on the water content and the amount that the package is compacted. The firmness increases in most cases after application, except for dry clay which is compacted too much. Also a present soil structure in the core can determine the amount of shape retention.
The deformation properties, immediately after application dependent on may factors. There are no clear relationships between classification test results and the deformation properties of clay. It is not very likely that the change of salt content in the water has an influence on the firmness of good compacted clay.

Workability

The workability of clay is of influence on the manner in which the clay is applied and compacted. It influences therefore to a large extent the functioning of a clay package in a dike.
The firmness and stickiness of clay are closely related tot the water content, as also is the workability. The workability is often indicated by the water content in relation to the Atterberg limits and the consistency index I_c. In general, clay is best desired packing density for working with when the water content lies close to the plasticity limit, which is between the plastic and the flow limit ($I_c \geq 0.6$).
The workability can strongly and negatively be influenced by an excess of water on the edge of the lumps and at the contact surface between the tools and the clay.

[TAW Klei, 1998]

V. Design method for clay dikes INFRAM

Engineering firm INFRAM (2003) has developed a design method for cover layers of clay by order of Dienst Weg- en Waterbouwkunde, Rijkswaterstaat. Below an overview of this method is given, based on this report:

Conditions

- H_s< 2m.
- Application of clay from MHW – 0.5m. (Good quality grass cover from MHW + 1.0m. [TAW, 1999])
- Yearly checks and regularly maintenance are required.

Assumptions

- Erosion durability decreases in the zone where the clay does not stay dry all the time as a result of the formation of the soil structure: from MHW + 0.5m.
  The rate of soil structure formation is around 10 mm/year.

Background design method

The use of clay as a cover layer asks for a layer thickness of several meters and a slope as gentle as possible. The cover layer is constructed in three layers:

- Top layer
  The top layer of 0.5m thick prevents dehydration and the formation of a soil structure in the protecting clay layer by covering it. It is as much as possible covered by original local bottom material with a view to realising the original vegetation. This layer is thus not included in the determinations of the strength of the dike.
- Intermediate layer
  In the intermediate layer the rest of the formation of the soil structure takes place, because the top layer will not be sufficient. By not including this layer in the strength determinations of the dike, a conservative approach in obtained. This layer meets the storm damage that occurs before the design storm. A thickness of 0.5m will last 50 years (soil structure 10mm/year).
- Lower layer
  This layer must be strong enough to resist the design wave load, the thickness is therefore determined by the damage during these conditions. The slope of the dike is most of the times determined by the original shape or the present fore land (Usually 1:6 or gentler).
  The first visual erosion will take place by breaking waves on the slope from a water level around 0.5m above the foreshore. In the worst case, by a rising water level a hole could develop till the underlying clay layer between 0.5 and 1m above the foreshore. From that moment the real clay erosion starts. By rising water levels the point of maximum erosion depth will rise as well.

Erosion due to flow, wave run-up and run-down will have a minor contribution to the maximum erosion depth in comparison to the wave attack.

The damage development is considered to be similar to the test with residual strength of the clay layer under a failing top layer of stones. Therefore the determination of the residual strength of clay covers from the Leidraad Zee- en Meerdijken [TAW, 1999, Chapter B6.9] is taken as a starting point.

Design method

- Hydraulic boundary conditions needs to be determined in accordance with the method used for dimensioning stone covers.

- The clay quality of the intermediate and lower layer are category 1. For the top layer are no demands.

- Slope conform original situation or connection to foreshore. (Around 1:6 or gentler)

- The normative erosion takes place due to wave attack, therefore the normative duration of this

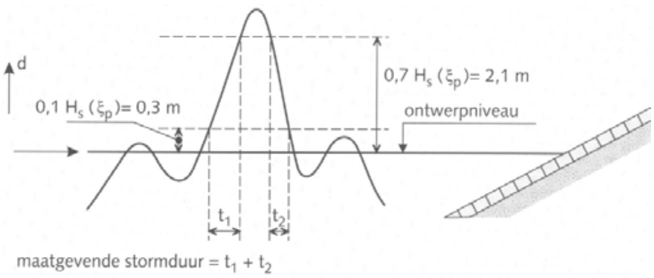


Figure V.1 Determination of the normative duration of the load.

load is determined from the Leidraad Zee- en Meerdijken [TAW LZM, 1999, Chapter B6.9]:

- o Consider the water level course during the design storm as a function of the time.
- o Determine for design level the duration for which concerns:

  $$0.1H_s\xi_p < d < 0.7H_s\xi_p$$

  See also Figure V.1 (d = water level above design level)

Figure shows the minimum demanded layer thickness at a specific duration of the load. This is extrapolated from table B6.9.1 [TAW, 1999], and will therefore be conservative. The calculation will be made for several points of the slope to gain the point of maximum erosion depth. This will be done with help of the spreadsheet. A minimum thickness of 1.2m is recommended.

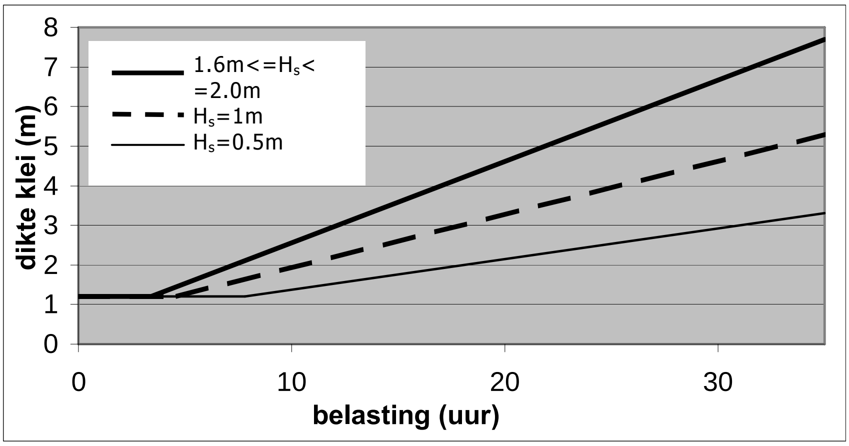


Figure V.2 Design method INFRAM (2003), minimum required thickness of a clay layer for a certain load.

- The calculated minimum layer thickness of the lower layer needs to be increased by a 10mm/year for every year in the design lifetime, to create an intermediate layer for the formation of a soil structure. This can be covered by a top layer of 0.5m thick.

- If the slope is 1:5 or steeper, then dimensioning to sliding needs also to be done.

- Transitional construction for connection to other cover types of 25m.

- Measures during construction and maintenance are part of the conditions for a good result:
  - o Damage to the top layer does not need repair. Damage to the intermediate layer needs repair before the storm season starts.
  - o If the foreshore starts to erode, protecting measure have to be taken.
  - o If the soil structure formation is different than 10mm/year this new value has to be used in the calculations.

If the foreshore starts to erode, protecting measures have to be taken.

If the soil structure formation is different than 10mm/year this new value has to be used in the calculations.

[INFRAM, 2003]

VI. Load

Wave impact

Regular waves

For regular waves, the wave impact is represented by the wave impact factor which represents the force of the impact. This factor is stochastic distributed. (van Vledder, 1990)

$$q = \frac{p_{\max}}{\rho g H}$$

q dimensionless wave impact factor (-)
p maximum pressure (N/m^2)
ρ water density (kg/m^2)
H wave height (m)

The wave impact factor of a breaking wave on a slope is depended on:

- Slope angle tan α
- Wave steepness H/L_0
- Wave height H
- Other factors

Klein Breteler (1992) formulated on basis of wave tests on asphalt slopes with wave heights of 1.5m the following expression for the impact factor:

$q_{90} = 90H/L_0$, for $H/L_0 \leq 0.03$

And for the location of the impact:

$$z/H = -0.0085\left(\frac{H}{gT^2}\right)^{-0.75} = -0.0337\left(\frac{H}{L_0}\right)^{-0.75}$$

z location of the impact with respect to the still water level

Irregular waves

In reality waves on the sea are irregular. In literature are hardly any formulas for wave impacts of irregular waves. Grune (1988) derived from experiments

$$q = \frac{p_{\max}}{\rho g H_s} = 5 \div 7$$

$$\frac{p_{\max,2\%}}{\rho g H_s} = 3.1$$

[Verheij, 1998]

Storm duration

In order to determines by how many wave impacts a cover layer is loaded, the tide difference and the storm duration needs to be determined.
According to the method of Den Heijer (1996), a storm set-up is super posited on the tidal curve for different location along the Dutch coast to give an indication of the water levels during a design storm. The tidal curve is schematised as a sinusoidal shape and the design storm duration is different for different locations along the coast, see table VI.1.

Table VI.1 Design storm duration.

Location	Design storm duration (hours)
Waddenzee	45
Noordzee	35
Westerschelde	35
Oosterschelde	Different storm set-up because of storm surge barrier

Representative situations for each location result from a combination of the water level differences and the accompanying duration. A long duration of the load on a small strip of the dike is unfavourable, because many wave impacts hit than the same location on the dike. This results in the following representative situations according to the Technisch Rapport Asfalt voor Waterkeringen [TAW Asfalt, 2002], see Table VI.2. This is for example for the Westerscheldt, a water level difference of 0.5m during 15.4h, which means that the still water level during 15.4h on a vertical measured strip of 0.5m the slope of a dike stays.

Table VI.2 Representative storm situations. [TAW Asfalt, 2002]

Location	Water level difference (m)	Duration (hours)
Waddenzee	0.5	17
Noordzee	0.5	15.4
Westerschelde	0.5	15.4
Oosterschelde	2.5	35

The average wave period can be determined with the following relation: [TAW, 2002, Asfalt]

$$T_{av} = 3.5\sqrt{H_s}$$

[TAW Asfalt, 2002]

TAW, Technisch rapport Asfalt voor Waterkeren, Technische Adviescommissie voor de Waterkeringen, 2002.

VII. Delta Flume 1984 (DF1984)

The erosion development of the bare clay under loading with regular waves, of the Delta Flume experiment to the residual strength clay a clay layer under a stone revetments, is given in Figure VII.1.

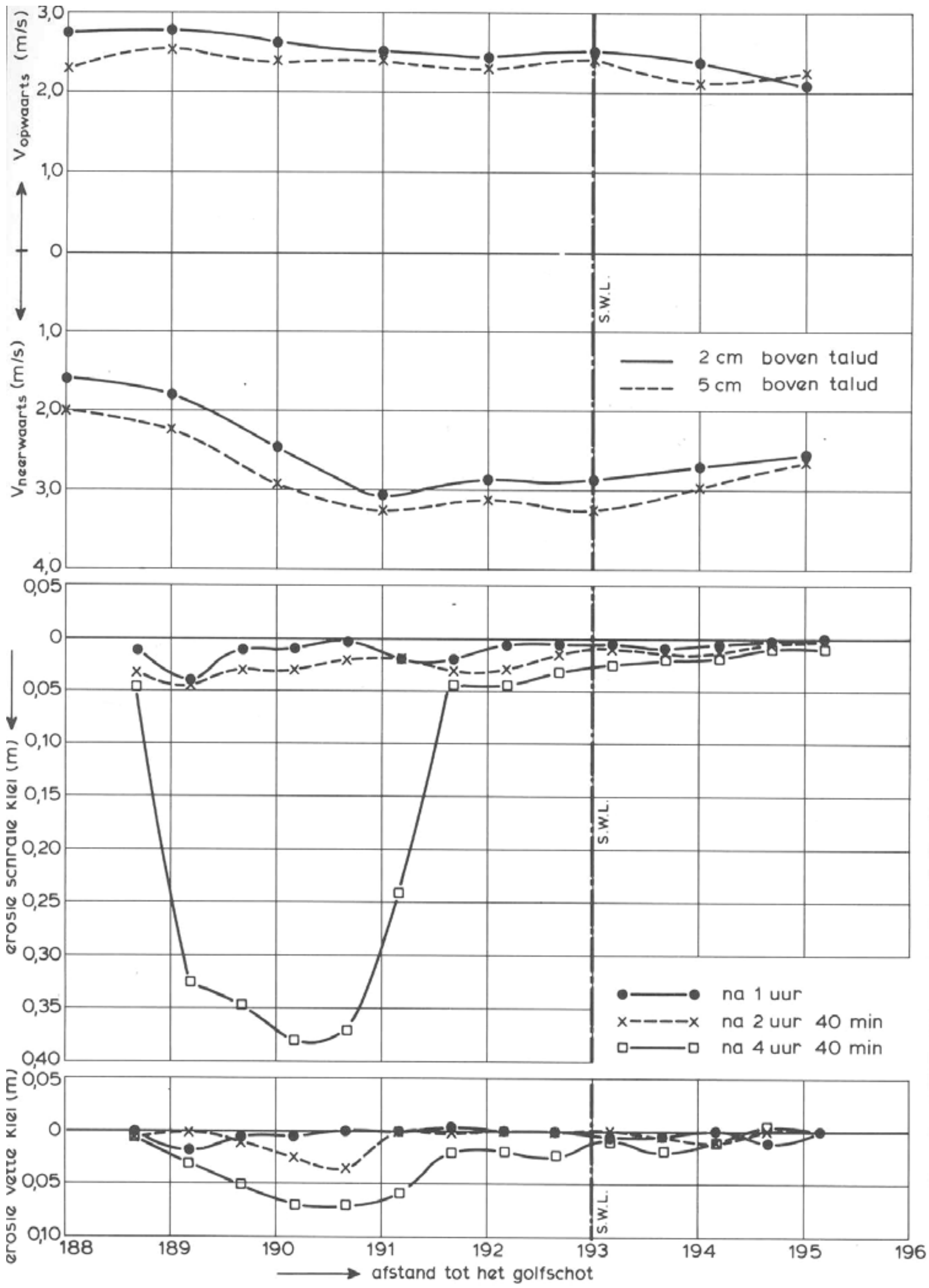


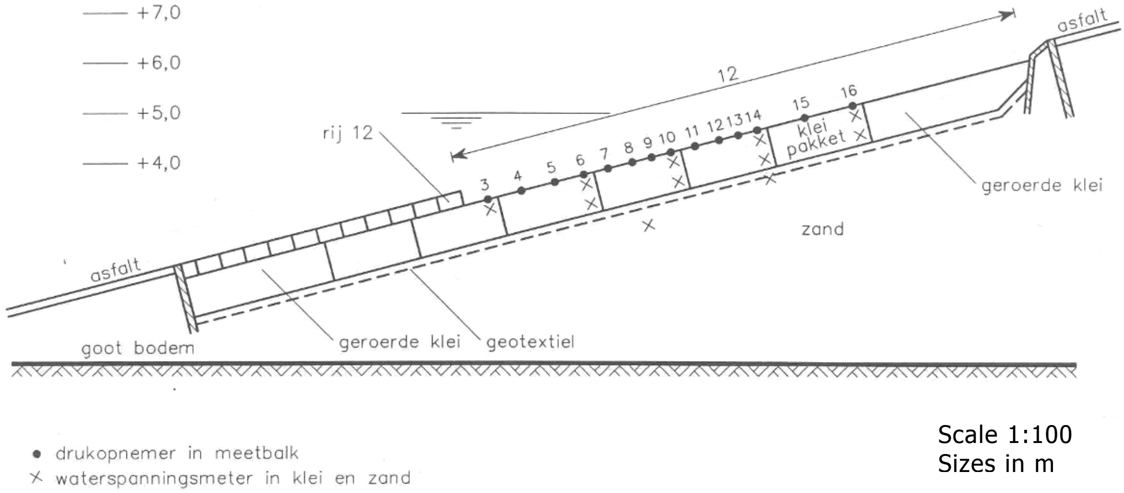
Figure VII.1 The water velocities and the erosion of clay on the bare clay. [Burger, 1985]


Figure VII.2 Cross pattern pressed into the clay to induce erosion. [Burger, 1985]

Table VII.1 Results classification clay. [Burger, 1985]

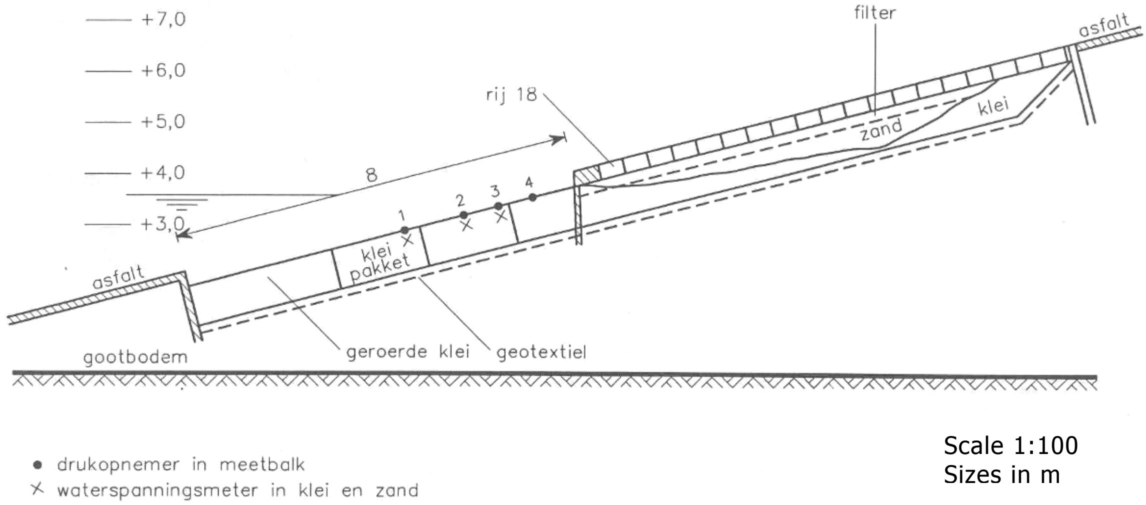
omschrijving	eenheid	"goede" klei	"slechte" klei
soortelijk gewicht	kg/m^3	$1,78 * 10^3$	$1,75/1,72 * 10^3$
watergehalte	%	36,8	3,43
droog soortelijk gewicht	kg/m^3	$1,30 * 10^3$	$1,2 * 10^3$
gew. % droge stof 2 μm	%	36	20
gew. % droge stof 16 μm	%	63	47
gew. % droge stof 63 μm	%	10	32
vloeigrens	%	56,3	62,8
uitrolgrens	%	26,0	29,5
plasticiteit in de x	-	30,3	33,3
max. proctor dichtheid	kg/m^3	$1,61 * 10^3$	$1,49 * 10^3$
optimum watergehalte	%	20	24

VIII. Delta Flume 1992 (DF1992S)



Scale 1:100
Sizes in m

Figure VIII.1 Model setup for experiment K, the high water experiment H_s=1.47m, T_p=4.9s, h=5.0m [Wouters, 1993]



Scale 1:100
Sizes in m

Figure VIII.2 Model setup for experiment L, the low water experiment H_s=1.0m, T_p=4.2s, h=3.5m [Wouters, 1993]

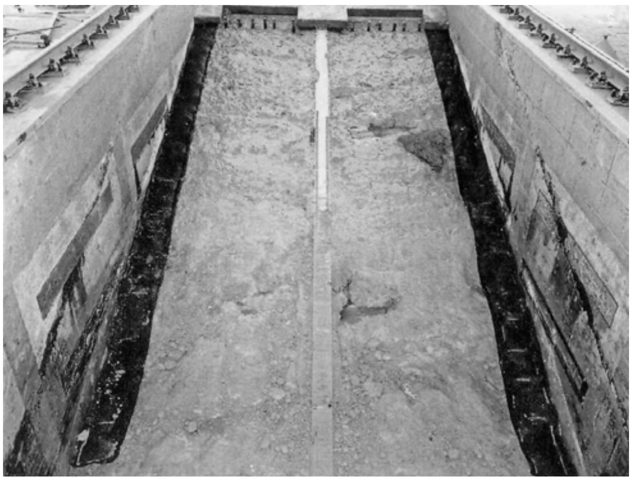
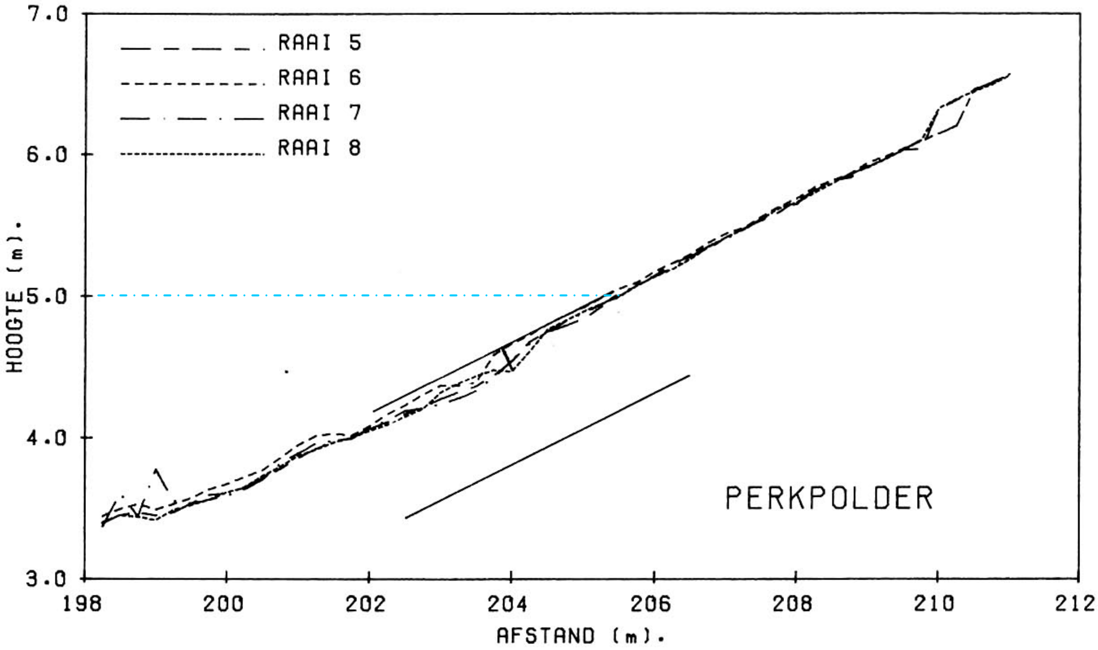
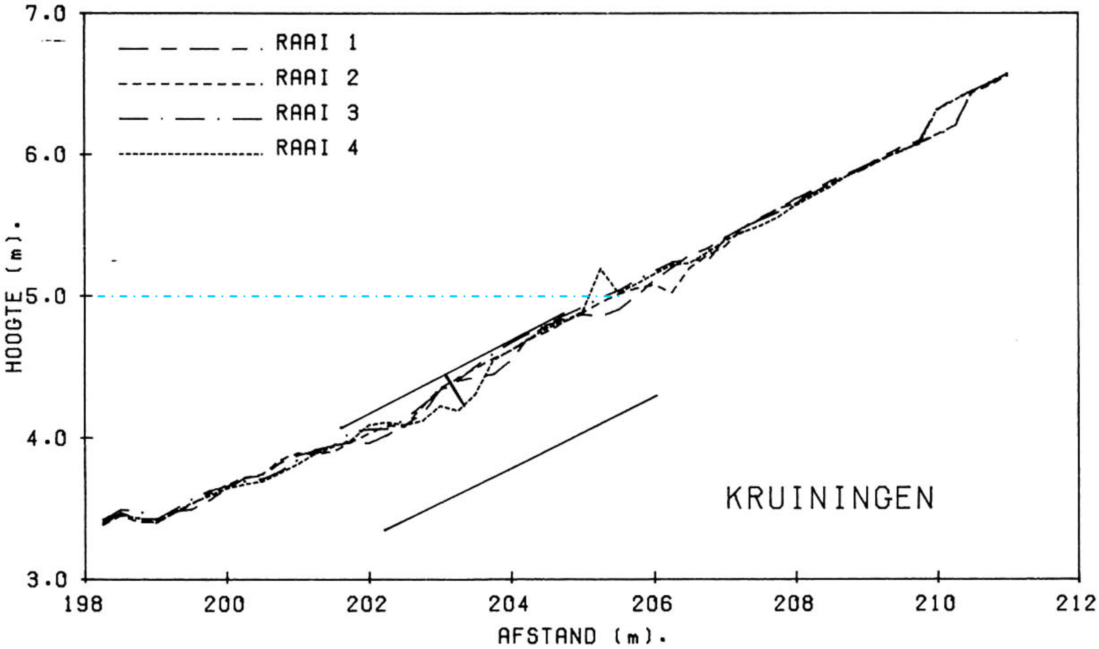


Figure VIII.3 First erosion development (after K2)

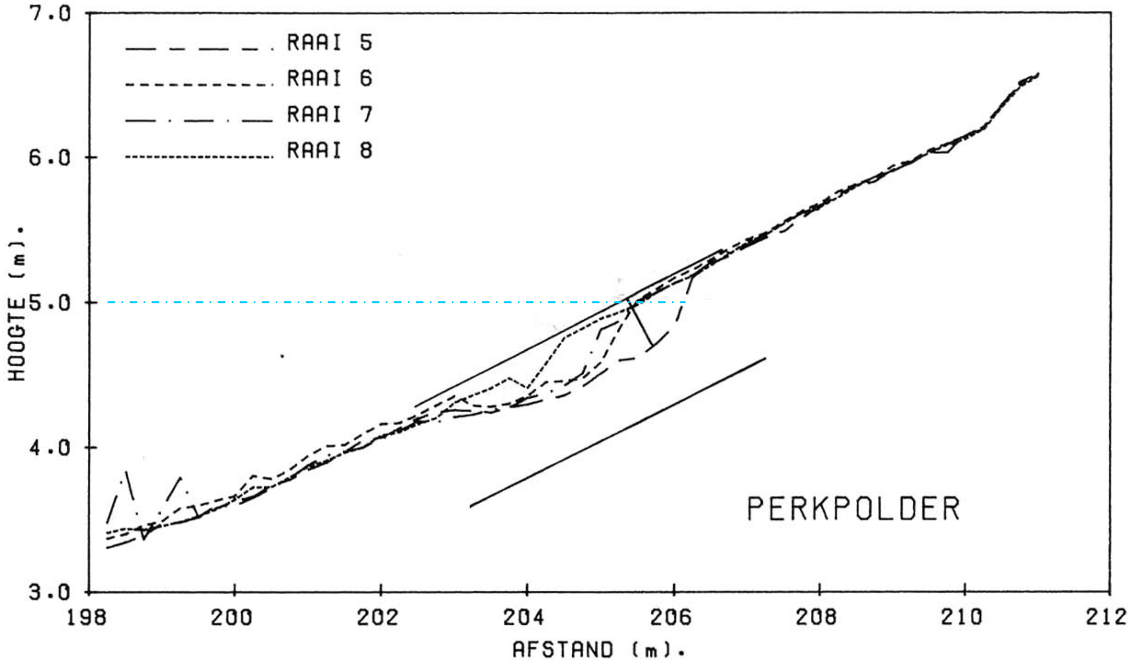
Erosion profiles

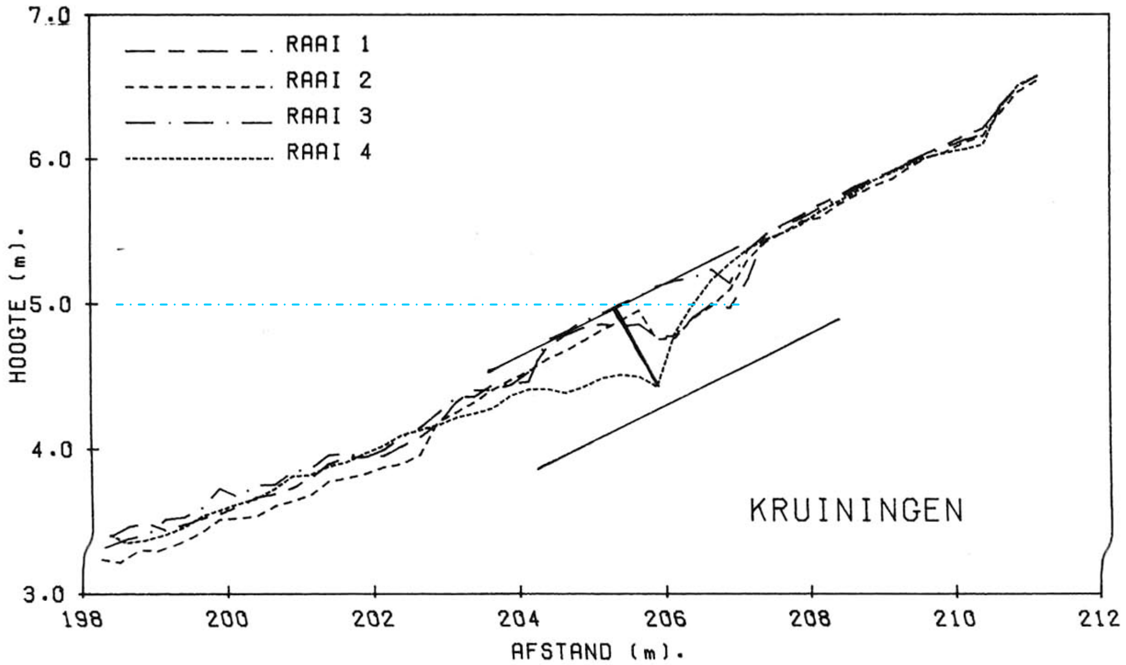
High water experiment



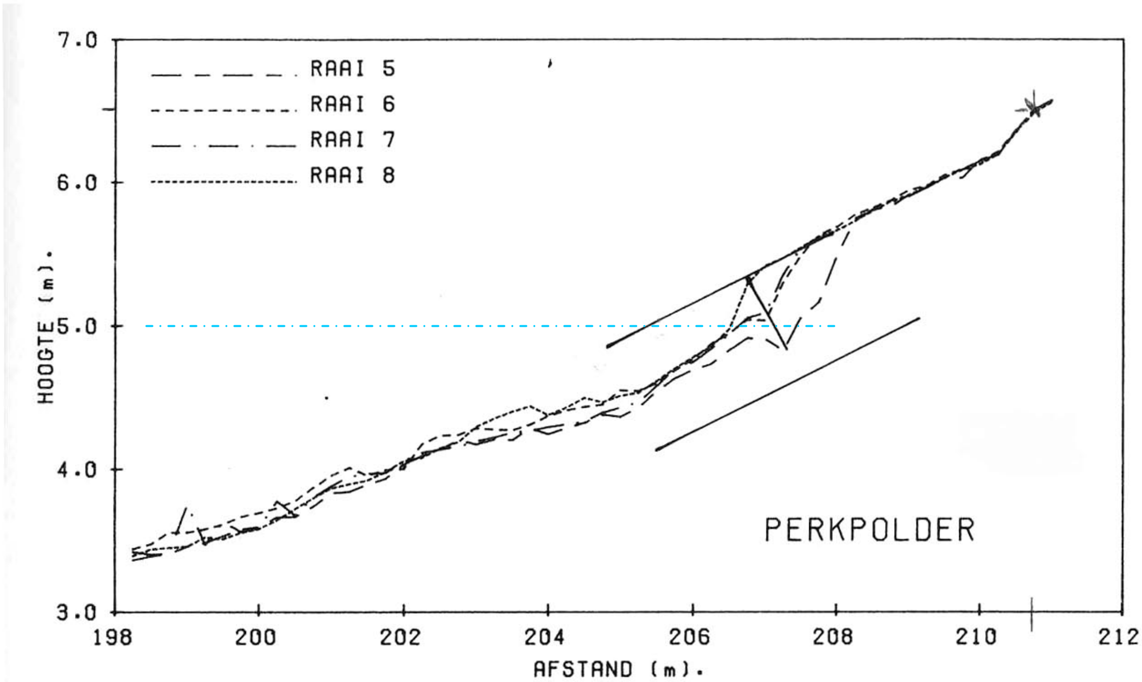


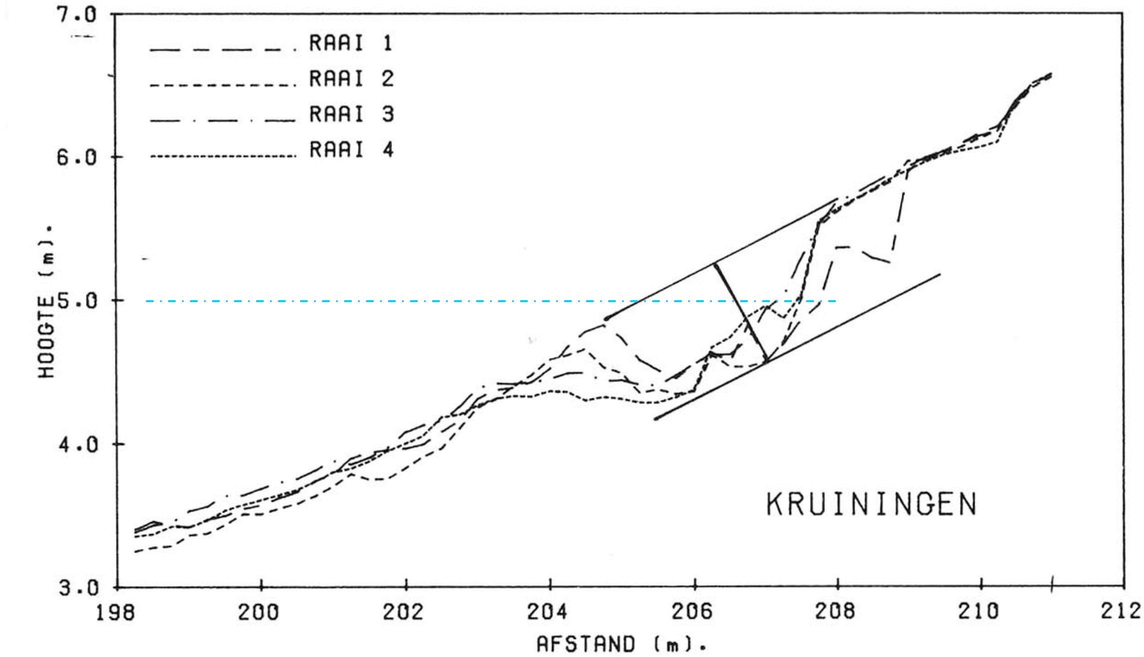
K2 After 1570s = 0.44h





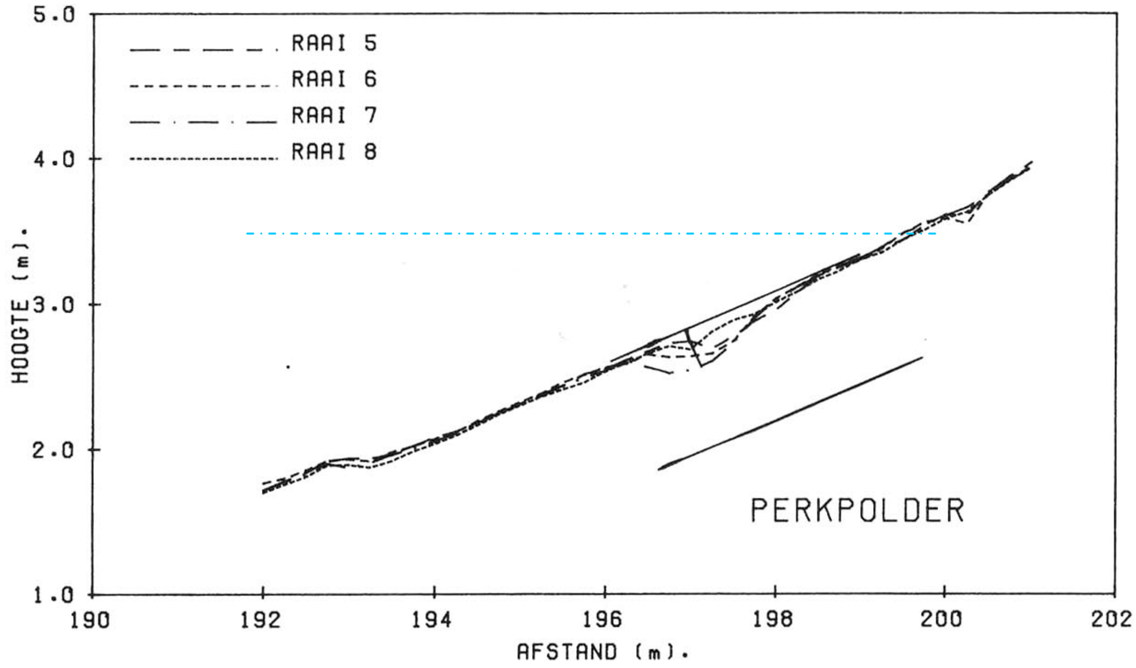
K3 After 4200s = 1.17h

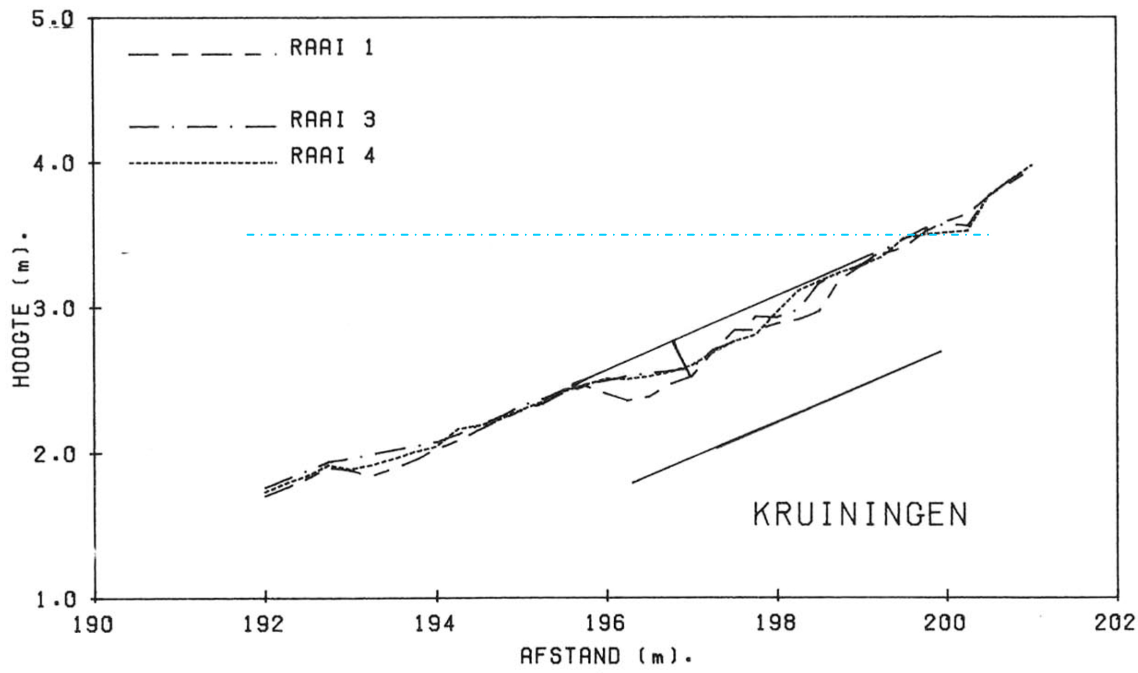




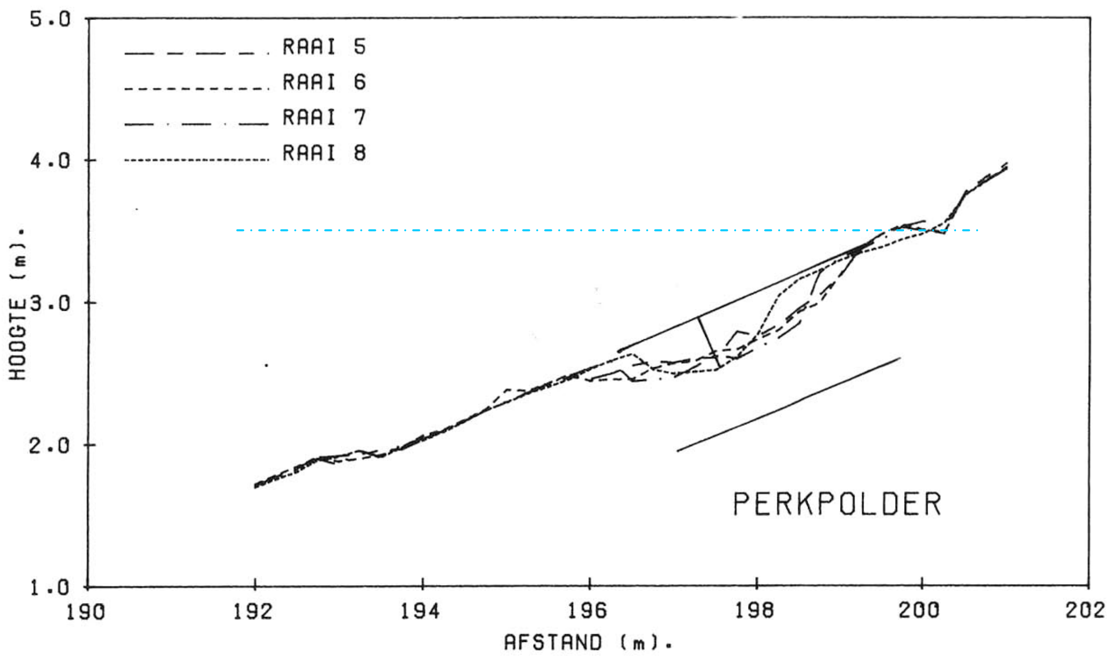
K4 After 7190s = 2.00h
Experiment was stopped, final profiles

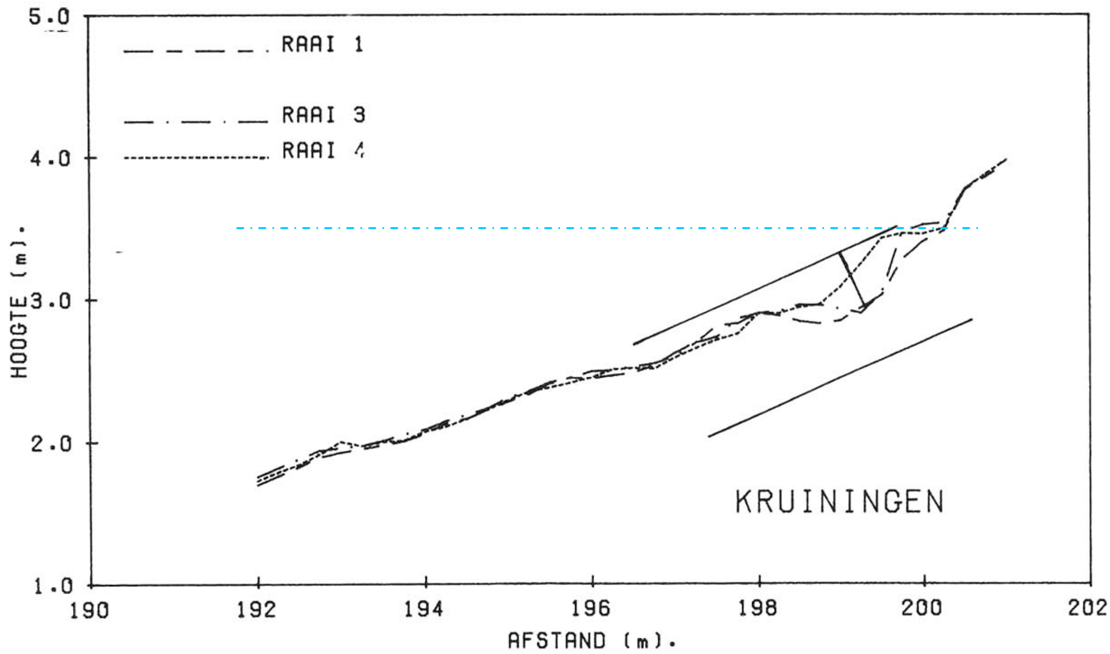
Low water experiment



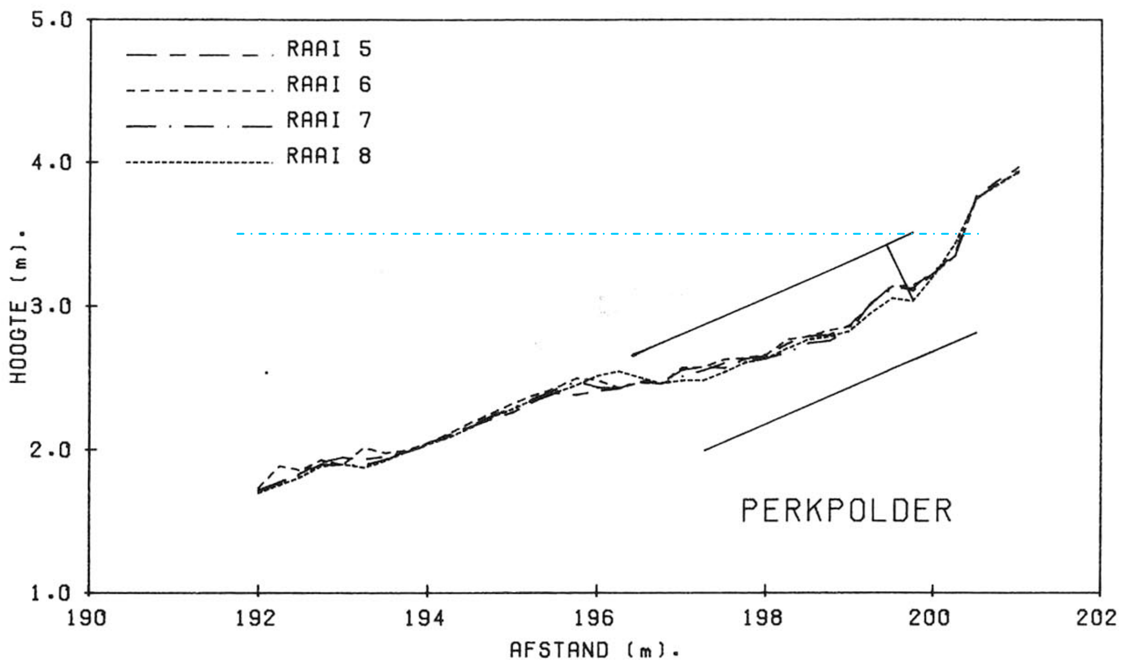


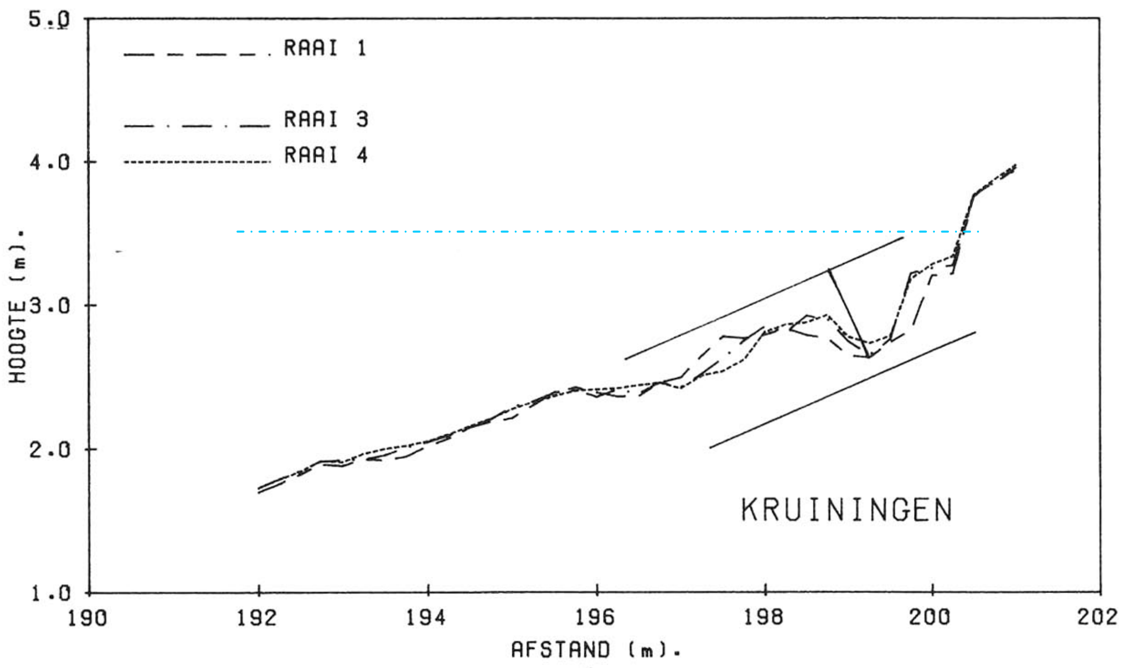
L2 After 2899s = 0.81h



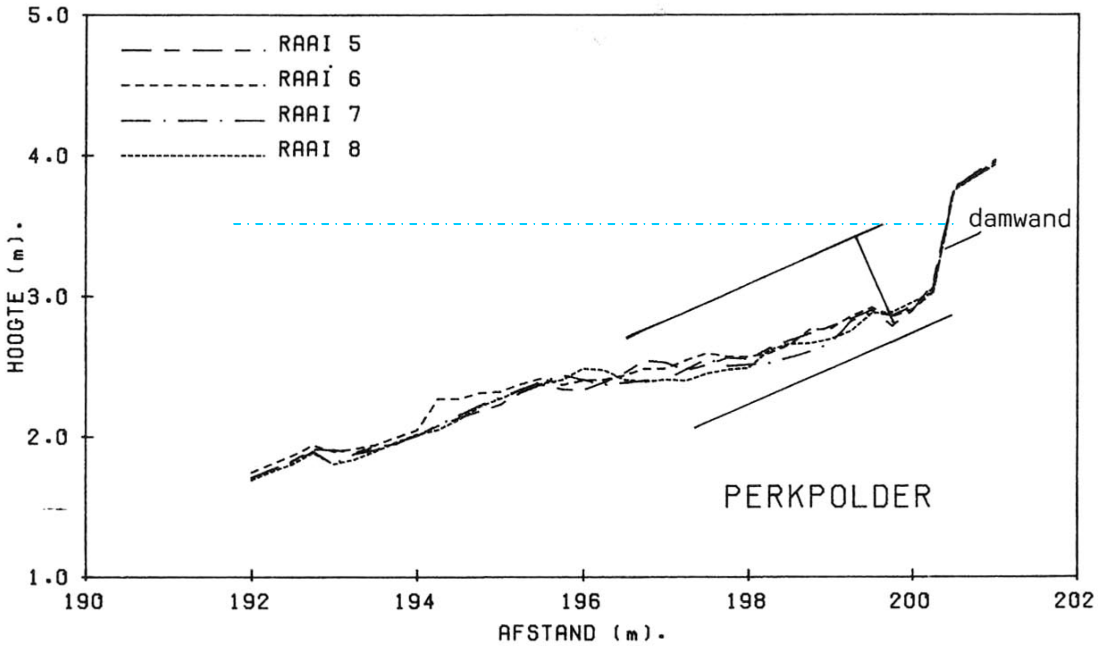


L3 After 6499s = 1.81h

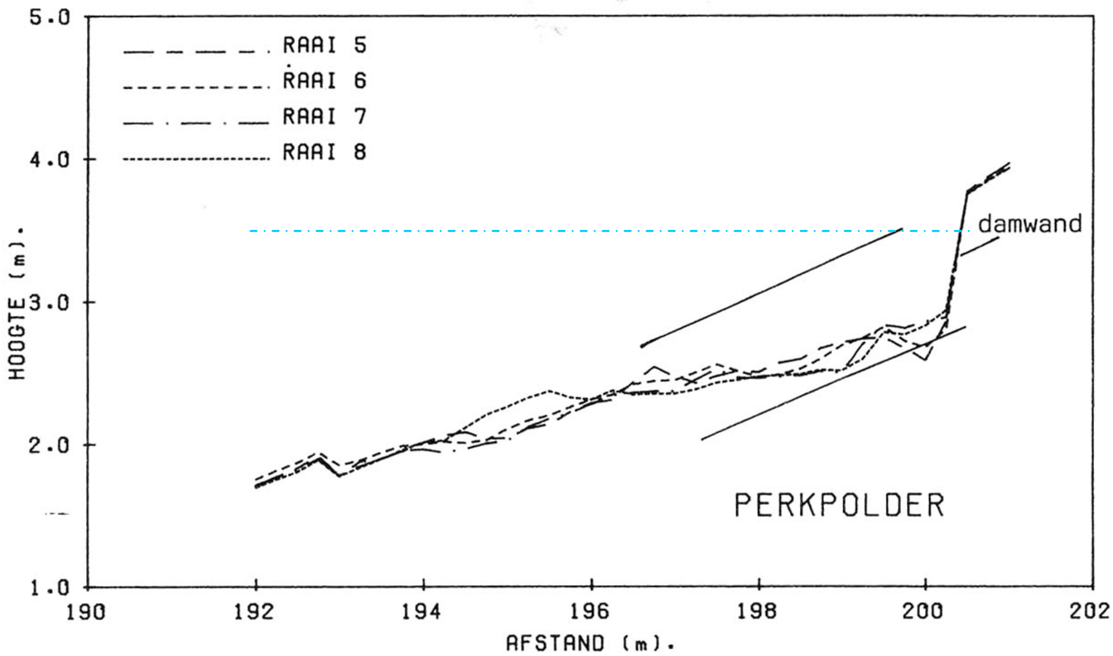




L5 After 12854s = 3.57h
Experiment on Kruiningen clay was stopped, final profile Kruiningen clay



L6 After 36414s = 10.12h



L8 After 53654s = 14.90h
Experiment was stopped, final profile Perkpolder clay

Table VIII.1 Results classification clay

LOCATIE	lutum <2u	zand >63u	WL	WP	PI	Wn	Ic	kalk	OM
	[in gew.% behalve Ic]								
KRUININGEN VEERHAVEN Oostelijke dijk (zie Bijlage II.1 voor locatie monsternummers):									
boven vloedmerk:									
toplaag no. 4	32.3	38.7	35.9	18.6	17.3	24.0	0.69	4.1	1.9
toplaag no. 5	36.8	29.2	45.0	18.8	26.2	27.8	0.66	2.9	2.3
onderlaag no.15	42.2	17.9	74.2	34.4	39.8	39.9	0.86	7.0	0.6
onder vloedmerk: onderlaag no.14	49.6	1.4	74.6	29.0	45.5	30.0	0.98	6.0	0.1
monster uit Deltagoot 0.2 m diepte .	31.9	18.1	43.3	22.8	20.5	24.2	0.93	4.3	0.5
PERKPOLDER									
proefvak 4 nabij dp 279									
bij vloedmerk no.8	28.5	28.6	39.0	22.2	16.8	21.7	1.03	9.5	0.9
bij vloedmerk no.9	26.4	31.9	39.3	16.7	22.6	22.9	0.72	10.2	1.0
bovenste rij no.10	26.3	29.2	40.6	17.4	23.2	20.3	0.88	10.5	0.9
bovenste rij no.11	26.3	28.9	37.1	21.7	15.3	22.8	0.93	9.8	1.0
monster uit de Deltagoot 0.2 m diepte	28.4	31.9	38.8	20.0	18.8	22.2	0.88	8.5	0.4
UITVULLAAG									
monster 1	36.7	8.0	71.1	29.2	41.9	35.3	0.85	8.8	2.7
monster 2	47.8	5.4	60.9	26.3	34.6	32.0	0.84	12.4	1.7

(WL = vloeigrens; WP= uitrolgrens; PI= plasticiteitsindex;
Ic = consistentie-index; OM= organisch materiaal)

IX. Delta Flume 1983 (DF1983)

Holes
The erosion development of the holes, made on beforehand in the grass, is given in Table IX.1.

Table IX.1 Development erosion holes [Burger, 1994]

Location	Duration (hours)	Surface (m^2)	Volume (m^3)	Max. depth (m)	Max. length (m)	Max. width (m)
Hole at NAP +4.50m	0	0.10	0.01	0.07	0.20	0.50
	6	2.70	0.75	0.39	2.35	1.45
	7	4.50	1.65	0.44	3.25	1.90
	8	6.30	2.55	0.44	4.00	2.00
Hole at NAP +5.00m	0	0.10	0.01	0.07	0.20	0.50
	6	1.40	0.50	0.48	2.05	1.30 (restricted by border)
	7	2.50	1.05	0.49	2.65	1.60 (restricted by border)
	8	4.60	1.90	0.49	3.60	1.80 (restricted by border)

Grass cover
An experiment was performed on the grass cover of a dike in Friesland during a design storm, with varying water levels, wave heights and wave periods for a duration of 48 hours.
The clay erosion developed in the zone of long lasting moderate wave loading as well in the zone with a heavy wave load in just a short period, both in the zone of 0.5-1.0m under the water level.
In the wave run up zone hardly ay erosion occurred (0.004m). [Burger, 1984]

Table IX.2 Properties Delta Flume 1983, on grass cover

Properties DF1983 grass slope		
Thickness clay layer	d_c	1.0m
Thickness grass sod	d_g	0.5m
Water level	h	2.4-5.5m
Wave height	H_s	0.65-1.85m
Wave period	T_p	5.0-5.9s
Slope		1:8
Duration experiment	t	29h

X. Laustrup

Laustrup et al. (1990) developed a method to determine the strength of a dike expressed as the mean return period for the critical load. This method enables to calculate the strength of dikes of varying qualities. It is based on results of simple tests on prototype dikes and the results have been calibrated with results of recorded failures of the Rejsby dike after a severe storm in 1976. The results of model calculations on 30 Danish dikes are consistent with the known lifetime of the dikes and the storms they have withstood without damage.

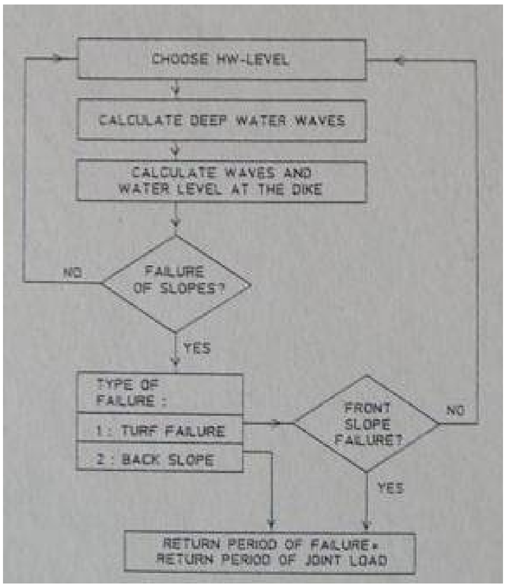


Figure X.1 Method to determine strength of dike [Laustrup, 1990]

Erosion of the turf
A part of this research was performing simple tests on the turf slopes of prototype dikes:

- A series of tests were performed on prototype dikes to obtain the resistance of turf as a function of the dry weight of grass roots per m^2 and the particle velocity in the breaking waves.
- Wave breaking is simulated with a water jet from a 4'' pipe.
- Sixteen tests are performed on turf of four different qualities.
- Each test with a certain velocity was repeated four times.
- The tests indicate that there is a certain critical particle velocity of a breaking wave. This indicates damage of the turf.
- Turf slopes steeper than 1:6 are assumed to fail when the velocity in the plunging breaker exceeds the critical velocity of the turf.

The waves are assumed to hit the slope at the level of mean high water. It is assumed that a plunging breaker falls from a height of $0.78*H_s$ above mean water level. The impact velocity v_A is than $v_A = \sqrt{g\left(h_B + 1.56 H_B\right)}$

v_A impact velocity (m/s)
g acceleration of gravity (m/s^2)
h_B water depth at breaking (m)
H_B wave height at breaking (m)

Fig x shows the results of one location.
Fig x gives the obtained critical velocities as a function of the dry weight of the grass roots in the turf.

Erosion of the core
After failure of the turf the erosion will continue horizontal through the core of the dike. This horizontal erosion is described as follows:

$$l(t) = k_c \cdot k_e \cdot v_A \cdot \frac{t}{T}$$

$l(t)$ horizontal erosion (m)
t time (s)
k_c calibration constant (-)
k_e core material constant (-), see table
v_A impact velocity (m/s)
T wave period (s)

Table X.1 Core material constants, obtained by repeated flushing of the different materials with a well defined velocity. [Laustrup, 1990]

Material	Core material constant k_e (-)
Sand, small amount of silt (10%)	$1.35*10^{-3}$ s/jet
Sand, large amount of silt (17%)	$0.89*10^{-3}$ s/jet
Clay with sand (60%)	$0.059*10^{-3}$ s/jet

This equation was calibrated on failures of the Rejsby dike (Denmark) recorded after the severe storm in 1976.

[Laustrup, 1990]

XI. Unprotected river banks

For cohesive soils no transport formula are available. Several studies have taken place in order to try to adjust the non-cohesive soil approach to cohesive soils. From 1992 to 1996 research was done by CUR (Civieltechnisch Centrum Uitvoering Research and Regelgeving) to investigate erosion of unprotected river banks. Next to flow, also waves from ships act as a load on the banks.

With this research should be kept in mind that the waves from ships are different than waves at the coast. The waves coming from ships are much lower and give therefore a much lower load on the slope than wind waves. Next to that imply wind waves an upper- and under flow, which are not present at ship waves because the wave duration is shorter and that mechanism has no time to develop.

Kamphuis and Wagner

Kamphuis (1987) concludes from studies on peat banks that the same equilibrium profile arises with cohesive soils if the grain sizes of the sand and the sand fractions of the cohesive soils are the same. This implies the possible use of the formula of Vellinga (1986) for cohesive soils. See also chapter 7.

$$z = 0.39 w^{0.44} y^{0.78} = p y^{0.78}$$

z see Figure XI.1 (m)
w fall velocity of the particles (m/s)
y see Figure XI.1(m)

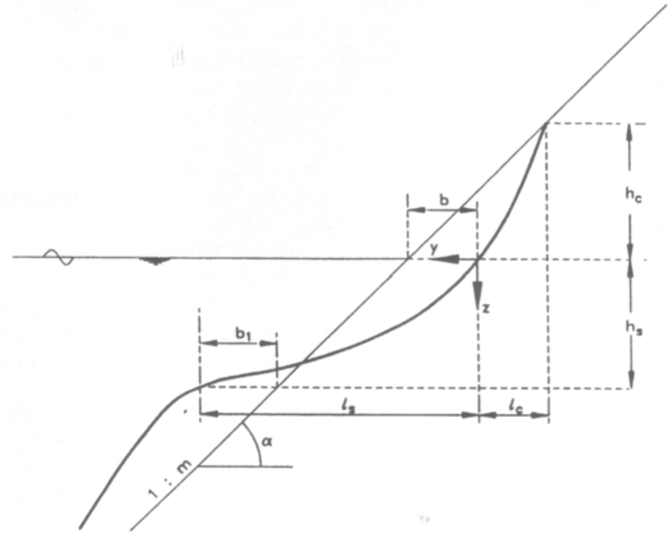


Figure XI.1 Schematised erosion profile [CUR, 1993]

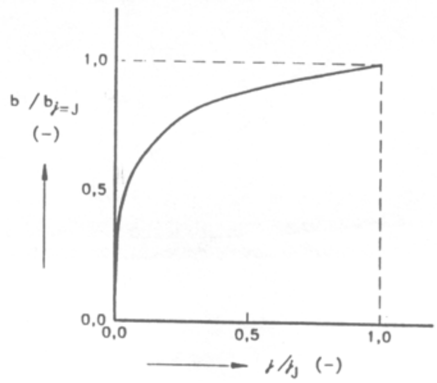


Figure XI.2 Decrease of lateral erosion in time

$b*j \geq b_c$

b lateral erosion around water level (m/s) Figure XI.1
j time (years)
b_c critical value lateral erosion (m)
b_c is dependent on the sliding of the soil and the value the manager accepts.

At banks in areas without a tidal range, the waves cause advancing erosion without the occurrence of an equilibrium profile. From model tests follows however that the amount of erosion decreases in time (Wagner, 1978): (See also Figure XI.2)

$$\frac{b_j}{b_{j=J}} = \left(\frac{j}{j_J}\right)^{0.2}$$

b_j lateral erosion after year j (m)
$b_{j=J}$ lateral erosion after year J (m)
j time (years)
j_J time after J years (years)

The tests were performed with H_s/D_{50} around 500. (Wave height of 0.25m if D_{50} is 0.5mm) The same relation counts for scour holes, but than with exponent 0.4.

Laustrup (1990)
Laustrup (1990) developed a formula to determine the residual strength of dike after erosion of the grass layer, based on measures on Danish dikes:

$$b_i = \frac{ck_e v_i j}{T}$$

c calibration coefficient (no values were given)
k_e Soil parameter (s), see table
v_i Impact velocity of the wave impact (m/s)
j time (s)
T wave period (s)

Table XI.1 Soil parameter k_e

Soiltype	k_e (s/ wave impact)
Sand + 10% silt	$1.35*10^3$
Sand + 17% silt	$0.89*10^3$
Clay + 60% sand	$0.059*10^3$

The factor k_e can be seen as a measure for the velocity of the declination of the shore. An increase of small soil particles (<0.063mm, silt) from 10% to 17% decreases the striding back with a factor 1.5, and an increase to 40% lead to a factor 25 in relation to sand with 10% silt.

DUROSTA-approach
DUROSTA is a transport model made by Delft Hydraulics (1991). It determines the profile development of dunes and beaches in time under wave attack during storms, on basis of time dependent suspended cross shore transport. The process implies a closed sand balance in cross shore direction and also long shore transport is taken into account. The continuity equation for the local bottom changes is:

$$\frac{dZ(y)}{dt} = \frac{-1}{(1-p)} * \left[\frac{dS_y(y)}{dy} + \frac{dS_x(y)}{dx} \right]$$

Z(y) position of the bottom in y (m)
p pore content (-)
$S_y(y)$ local cross shore transport (m^3/s)
$S_x(y)$ local long shore transport (m^3/s)

This model could be used for cohesive soils with some adaptations, like adapting suspended transport to bottom transport. For waves from ships the wave load also needs adaptations, because they are different from wind waves. Next to that imply wind waves a upper- and under flow, which are not present at ship waves because the wave duration is shorter and that mechanism has no time to develop.

CUR (1993) concludes that adapting DUROSTA or for cross shore transport of unprotected river banks under ship loads will take a large effort and will probably lead to uncertain results due to uncertain transport formulas. Despite this conclusion, two test calculations were performed with DUROSTA which led to reasonable results. The first canal consisted of peat, the second of a clay-sand mixture. Good transport formulas are needed in order to obtain a well working model.

Erosion-rate-concept
The erosion-rate-concept forms a relation between the hydraulic load and the local strength parameters. This is in first instance elaborated for the stability of cohesive banks loaded by flowing water. This method determines the erosion velocity, change in bottom width, critical height, angle of failure and the volume of the piece that fails. The formulas are derived for cohesive soils and are valid for critical shear stresses lager than $0.6N/m^2$:

$$\frac{db}{dt} = \frac{R}{\rho g}\left(\frac{\tau}{\tau_c} - 1\right), \text{ with } R = 0.364\tau_c e^{-1.3\tau_c}$$

B horizontal profile parameter (m)
t time (s)

ρ volumic mass (kg/m^3)
g acceleration of gravity (m/s^2)
τ shear stress (N/m^2)
τ_c critical shear stress (N/m^2)
R initial erosion velocity (N/m^2s)

With $\tau = 0.5 c_f \rho g u_b^2$, this can be re-written into:

$$\frac{db}{dt} = \frac{R}{\rho g}\left(\frac{u_b^2}{u_c^2} - 1\right), \text{ with } R = 0.364\tau_c e^{-1.3\tau_c}$$

c_f shear stress coefficient
u_b flow velocity along the bank (m/s)
u_c critical velocity (m/s)

To use this approach also under wave loading, this wave load has to be related to a flow velocity. This is done on basis of conservation of energy:

$$0.5 m v_0^2 = mgRu$$

m mass (kg)
v_0 wave run-up velocity (m/s)
Ru wave run-up (m)

From measurements the upper limit for the wave run-up velocity appears to be:
$v_0 = 1{,}25(gRu)^{0.5}$ with $Ru = \xi H_i \cos\beta$ $0 \le \xi < 2$
$Ru = 2H_i \cos\beta$ $\xi \ge 2$

H_i wave height (m)
ξ breaker parameter
β angle of wave attack (°)

During the passage of a wave the flow velocity increases and decreases due to wave run-up and run-down. This is assumed to have a sinusoidal shape:

$$v_r = v_0 \frac{2\pi t}{T_i}$$

v_r time dependent wave run-up speed (m/s)
t time (s)
T_i period secondary waves (s)

The flow velocity u_b in XX is assumed to be equal to v_r. To obtain the erosion due to one wave, XX is integrated over one wave period T_i:

$$b_{1,wave} = \frac{0.364}{\rho g}\tau_c e^{-1.3\tau_c}\left(\frac{v_0^2}{u_c^2} - 2\right)\frac{T_i}{2}$$

The eroded surface can be determined with the schematizised erosion profile in Figure XI.3 and the assumption that b'=b.

$$A = 0.5b\frac{h_c + h_s}{\sin\alpha}$$

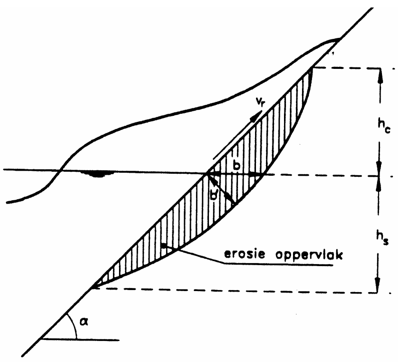


Figure XI.3 Schematizised erosion profile, with b'=b

Values for the critical velocity can be determined from erosion tests or formulas like developed by Mirtskhoulava (1991).

The model above was tested during a study to transport in Sarawak. See Figure XI.4 for the erosion mechanisms. The results from the model corresponded with the field observations.

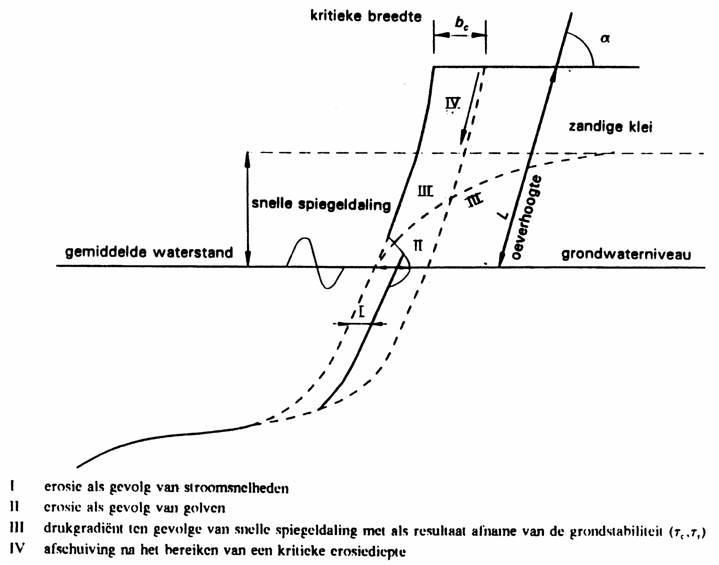


Figure XI.4 Erosion mechanisms Sarawak

[CUR, 1993]

Delft Hydraulics (2000)

Delft Hydraulics has developed a pragmatic bank erosion model based on the erosion-rate-concept. The basic of the model is formed by the following formula for bank erosion:

$$y = \frac{1}{2\mu} \ln\left(2\mu c_E H_0^2 t + 1\right)$$

y	width of the eroded shore (m)
H_o	initial approaching wave height at y=0 (m)
t	time (s)
μ	parameter for wave damping (m^{-1})
c_E	coefficient for the strength of the shore material ($m^{-1}s^{-1}$), see Table XI.2.

With the parameter μ the effect of the reduction on the load due to foreshore or vegetation can be taken into account. Coefficient c_E is related to the strength of the bank, see Table XI.2.

Restrictions of the model, given by Delft Hydraulics:

- Inhomogeneities of the soil, different layers with different properties, are not taken into account.
- The erosion determined is only due to ship waves.
- Varying water levels are not taken into account.

[Booster, 2003]

Table XI.2 Parameter c_E by CUR (1996) [Booster, 2004]

Soil type		c_E [$m^{-1}s^{-1}$]
Grass	Good	$0.01*10^{-4}$
	Moderate	$0.02*10^{-4}$
	Poor	$0.03*10^{-4}$
Clay	Very good	$< 0.5*10^{-4}$
	Good	$0.60*10^{-4} – 0.75*10^{-4}$
	Structured	$1.50*10^{-4} – 2.80*10^{-4}$
	Moderate - Poor	$2.80*10^{-4} – 7.00*10^{-4}$
Sand		$> 10*10^{-4}$

XII. Clay cover layer thickness [Pohl, 2005]

The thickness of the clay layer is determined by the reduction of the infiltration. On the basis of utility the cover layer has to be a size Δd thicker than the crack depth d_c.

$$d \geq d_c + \Delta d \text{ (m)}$$

d needed thickness clay cover layer
d_c crack depth
Δd additional thickness

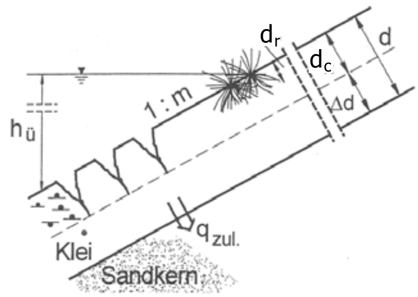


Figure XII.1 Structure clay layer. [Pohl, 2005]

According to Weißmann/Richwien 2004 is the maximum crack depth by the in Germany occurring climate circumstances around the threefold of the shrinkage size.
The shrinkage size V_s refers to the specific inclination of the soil to build up shrinkage cracks. The shrinkage limits according to DIN 18122 enquire the difference between the soil volume at liquid limit water content and the volume after oven drying, related to the volume at liquid limit water content. On the basis of biotic macro pores a minimum root depth of 0.2m has to be assumed.

$$d_c = 3V_s$$

$$d_c \geq d_r = 0.2\text{m}$$

V_s shrinkage size
d_r minimum root depth
...
The additional thickness Δd is needed to reduce the pressure potential of the water level and the cover layer (d). The gradient in potentials leads to the over time and surface unit infiltrated discharge q.

$$q = k_r \frac{h + d_c + \Delta d}{\Delta d} \text{ (m}^3\text{m}^{-2}\text{s}^{-1}\text{)}$$

q infiltrated discharge ($m^3m^{-2}s^{-1}$)
k_r permeability of the soil (m/s)
h water level (m)

For a given discharge this results in:

$$\Delta d \geq \frac{k_r}{q - k_r}\left(h + d_c\right) \text{ (m)}$$

The soil is only suitable for dike building if the permeability of the soil is smaller than the infiltrated discharge, k_r<q. On basis of a parameter study Weißmann/Richwien proposed a permissible infiltration of $1*10^{-5}$ $m^3m^{-2}s^{-1}$ (for not systematically dewatered dike cores). In compliance with this value no significant back water originates within 6 hours in the dike core, so the stability of the dike is maintained.
For construction reasons the lower limit of the additional thickness is determined by: $\Delta d \geq 0.1h$; $\Delta d \geq 0.5$ m [Pohl, -; Pohl, 2005]

The needed additional thickness and the total thickness of the clay cover layer depending of the water level is given in the graph.

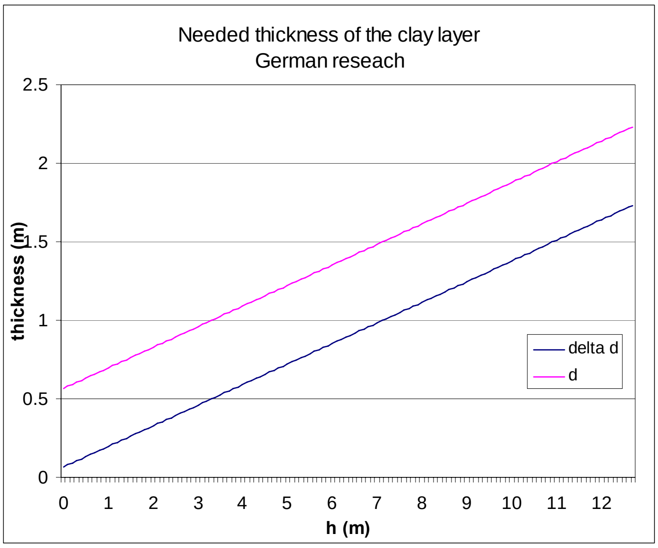


Figure XII.2 The needed additional thickness and the total thickness of the clay cover layer depending of the water level.

Influence of the wave impact

The load of breaking waves can make numerous damages to the outside of the dike cover layer, in form of erosion or sliding. A pressure impact originates due to the shape of the breaking wave, because the water mass forms a fall parable which includes air. The air is compressed till the wave finally bursts open. The released energy E_r is brought into the soil as a pressure impulse over an area of several decimeters within 1/100 seconds. The pressure impact hits the clay cover which is already damaged by cracks or animal digging and clearly is weakened by infiltration.
The pressure peaks are stochastic distributed and amount to a multiple of the hydrostatic load. Gentle slopes (1:m) are favorable, because this reduces the wave impact by the longer length of stay of the decreasing water, which reduces also the pressure maxima.

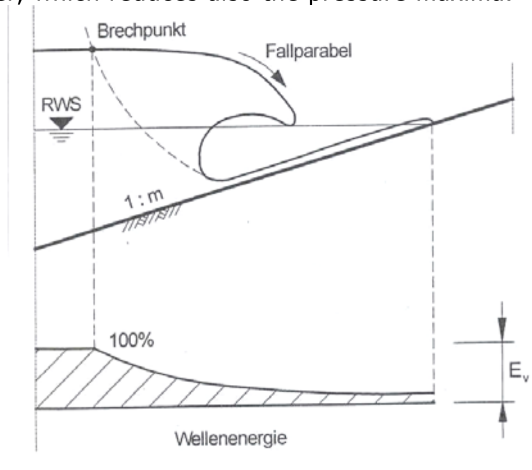


Figure XII.3 Influence wave impact. [Pohl, 2005]

Führböter (1966) proved a linear dependency of the pressure peaks compared to the wave height.

$$p_{\max} = C(i)\rho_w g H_s \text{ [kN/m}^2\text{]}$$

P_{max} maximum pressure peak
H_s wave height
C(i) dimensionless coefficient which varies stochastically with the probability of occurrence(i).

For wave pressures which are to be expected once in 1000 waves, Führböter specified C(i) in relation the slope of the cover layer 1:m to 24/m.

$$p_{\max} = \frac{24}{m}\rho_w g H_s \text{ [kN/m}^2\text{]}$$

Due to compressive stresses on the surface an instationary pore water pressure builds up, which decreases quickly with depth as a result of the small soaking width. Depending on the permeability and the deformation properties of the soil exists a pore water overpressure after the loading, which reduces in time. In this way the rest of the cover layer is used. During the soaking short-lived en local restricted deliquesces can occur.

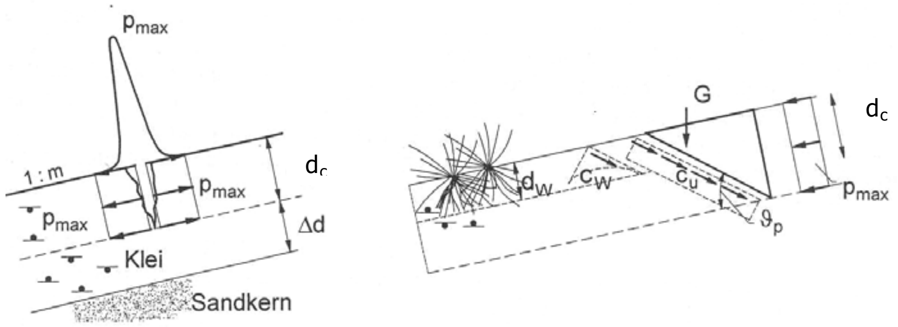


Figure XII.4 Water pressure in clay during wave impact. [Pohl, 2005]

From the equilibrium of forces follows for the assumption of a sliding surface of 45° for the saturated soil and with neglecting the dead load of the soil, follows a required strength of the cover layer (τ) of 50% of the wave pressure.

$\tau \geq 0.5 p_{\max}$

For the undrained case, the angle of repose is assumed to be 0°, which facilitates the case the maximum shear strength of the soil to the sum of the root cohesion (c_w) and the undrained shear strength of the soil (c_u). $\tau = c_w + c_u$ (Coulomb: $\tau = c + \sigma' \tan\varphi$) On the basis of the pressure peak follows a needed strength c_u.

$$c_u \geq \frac{12}{m} \rho_w g H_s - c_w \frac{d_w}{d_R} \text{ [kN/m}^2\text{]} \quad \text{, with } \frac{d_w}{d_R} \leq 1$$

Richwien (1993) proved a logarithmic course for the reduction of strength with the consistency index for a soil consisting of aggregates:

$c_u\left(w_{sat}\right) = c_u\left(w_p\right)^{I_{c,sat}}$ (kN/m^2)

$c_u\left(w_{sat}\right)$ undrained shear strength at saturation

$c_u\left(w_p\right)$ undrained shear strength at a water level at expand limit

$I_c = \frac{w_L - w}{w_L - w_P}$ (-) consistency index

$w_{sat} = n \frac{\rho_W}{\rho_d}$ (-) water level at saturation, which follows from the pore space

According to the rate of compression the saturated soil can have different consistencies and therefore also different strengths. This results in a demand for the pore number, to take up the pressure loads from the wave impact.

$$n \leq \frac{\rho_d}{\rho_W} \left[w_L - \frac{\ln\left(\frac{12}{m} \rho_w g H_s - c_w \frac{d_w}{d_R}\right)}{\ln c_u\left(w_p\right)} * \left(w_L - w_p\right) \right] \text{ (-)}$$

ρ_d dry density; $\rho_d = \rho_s(1-n)$

n pore number

ρ_s grain density

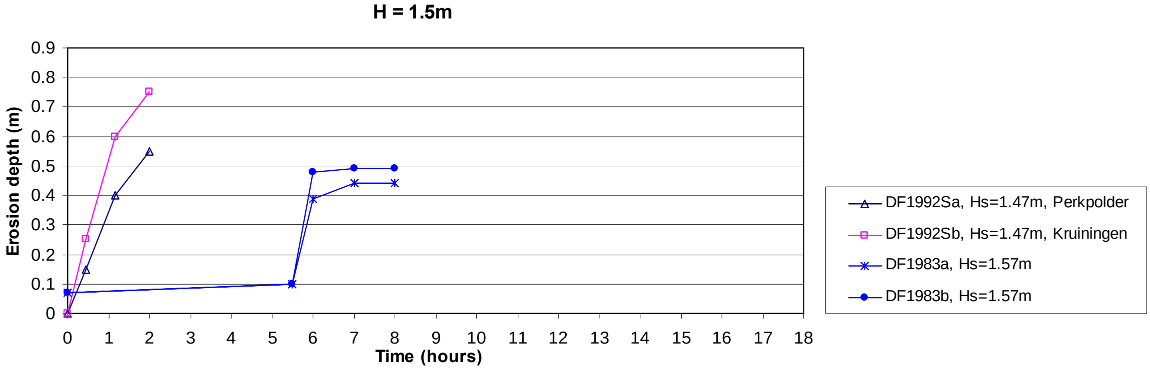
[Pohl, 2005; Pohl, 2006]

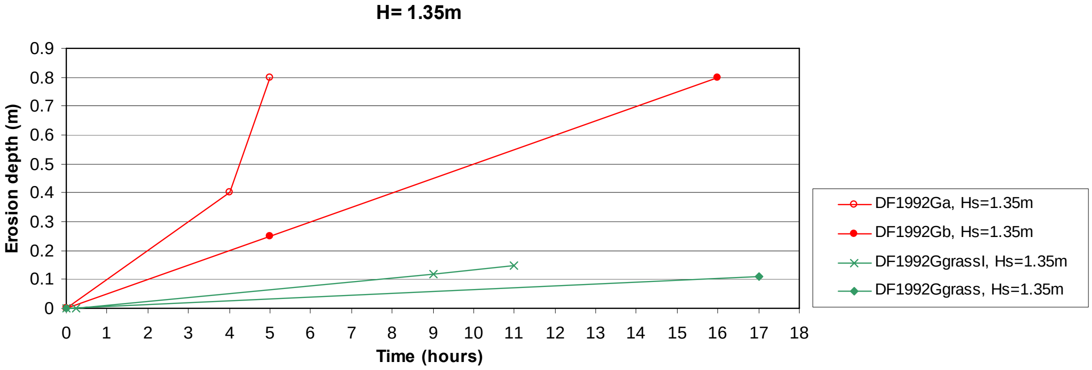
Conclusions

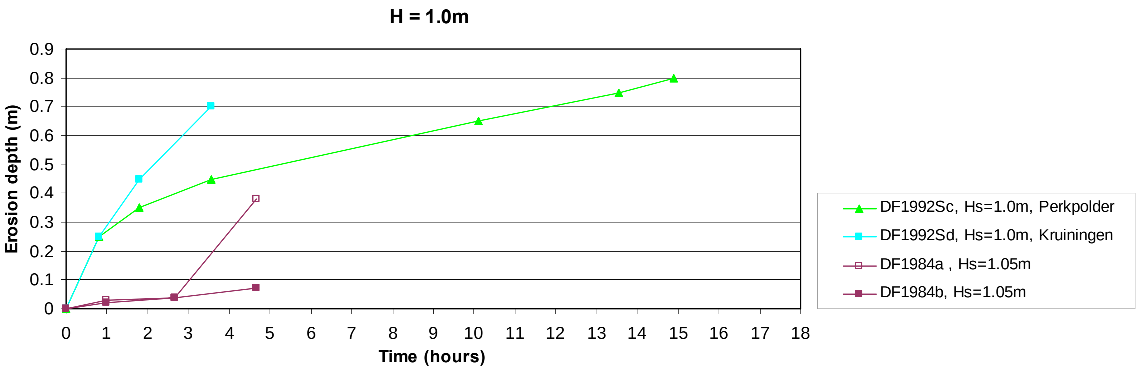
- This method joins with the starting assumption for construction of the cover layer. The cracks depth is based on the shrinkage size v_s, which follows from German standards.
- The clay cover layer will be strong enough to withstand the pressure loads due to wave impact if the pore number does not exceed a certain value. This is thus depending on the installation of the clay.
- The thickness of the clay layer is not directly dependent on the wave height, because the wave height is a variable in the amount of compression needed of the clay, the pore number.
- This method assumes a significant influence of the cohesion due to the roots of vegetation. This joins with the theory that grass gives a significant contribution to the strength of the clay cover layer, but since it is still difficult to quantify this contribution, it will probably also be difficult to quantify the cohesion of the roots.

XIII. Comparison Delta Flume experiments on clay

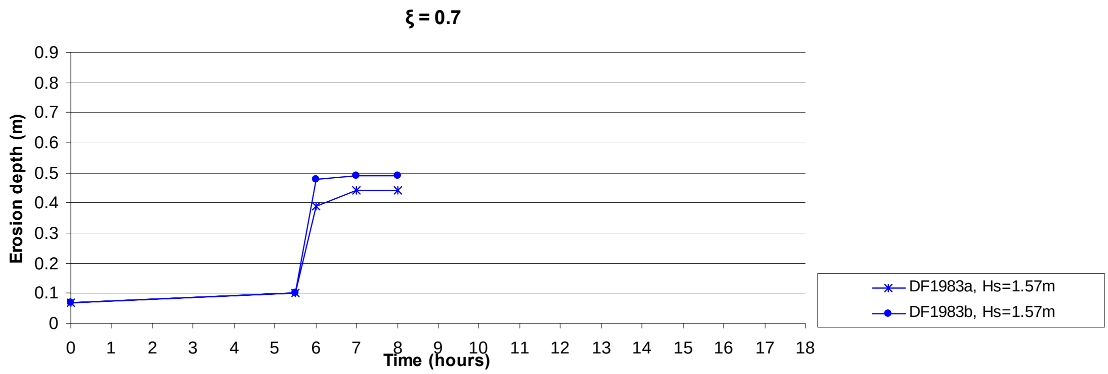
Wave height

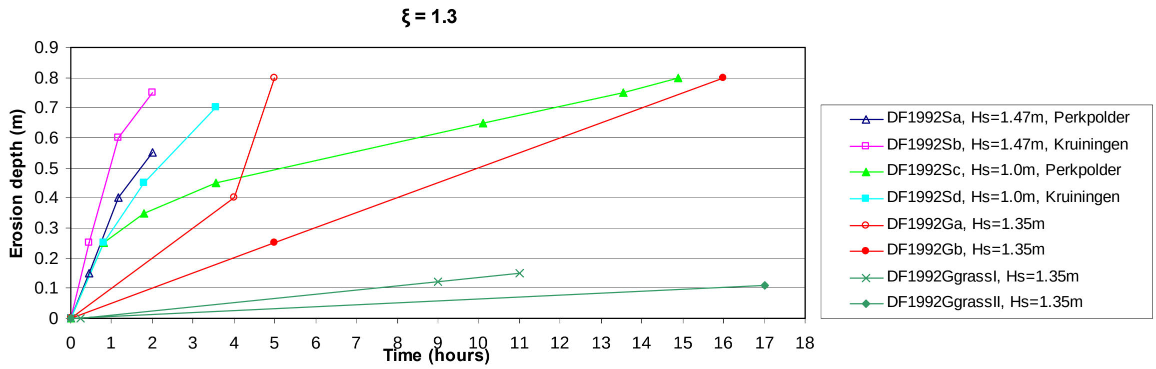


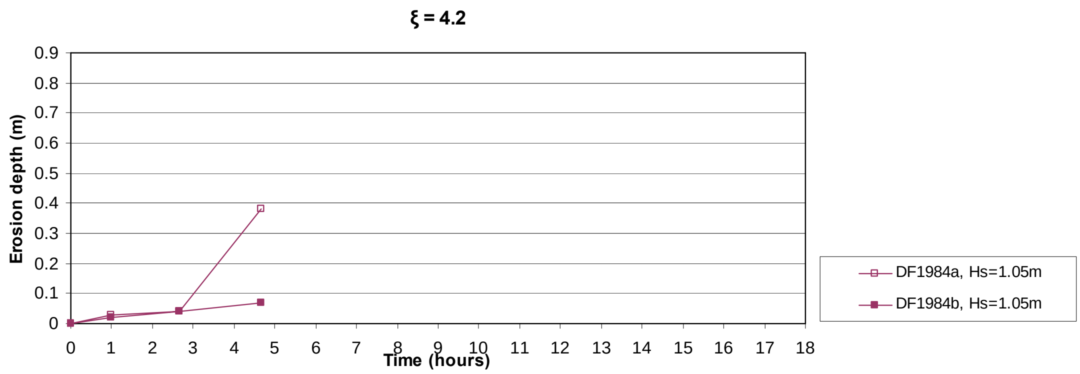




Iribarren number

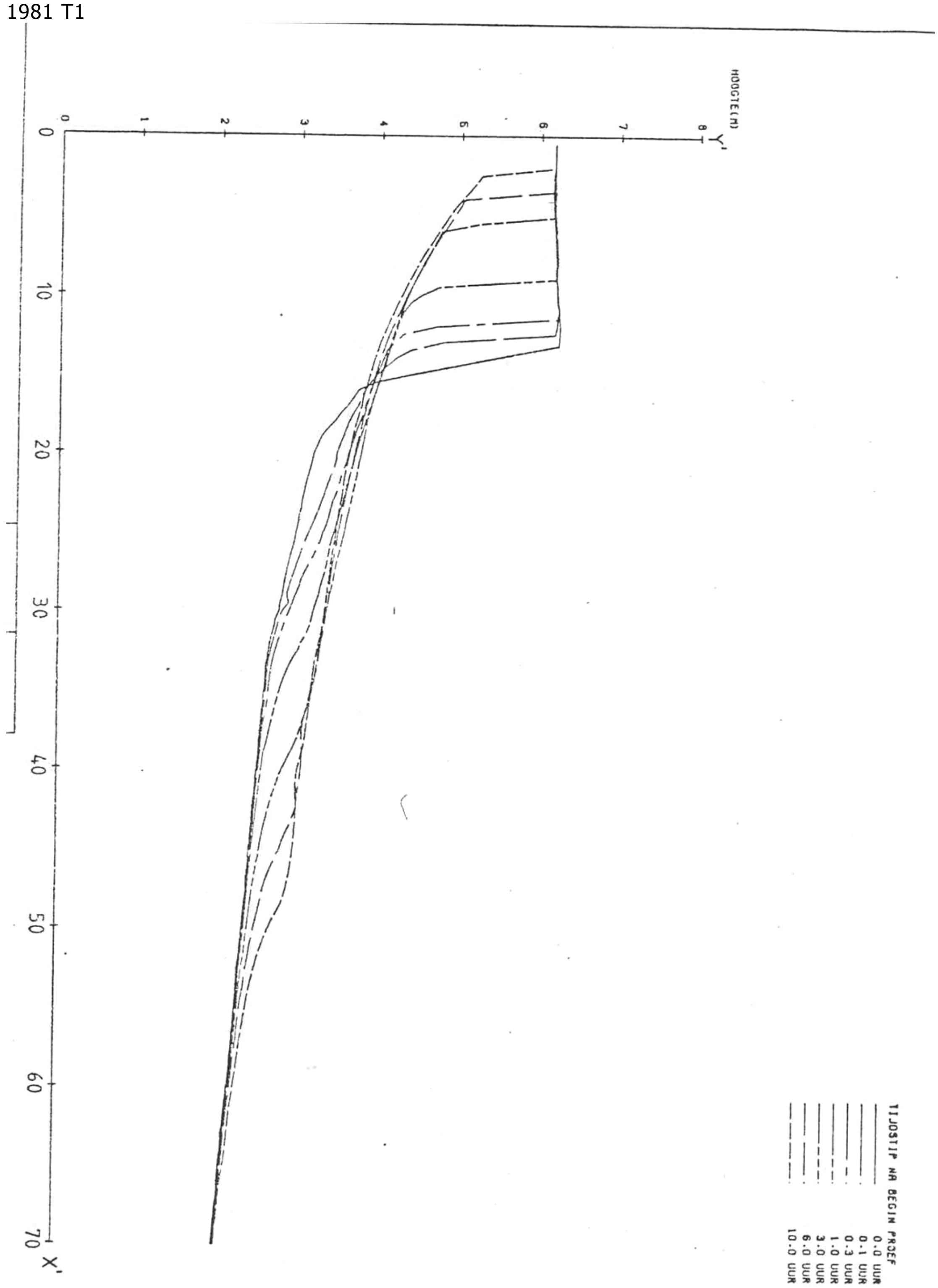




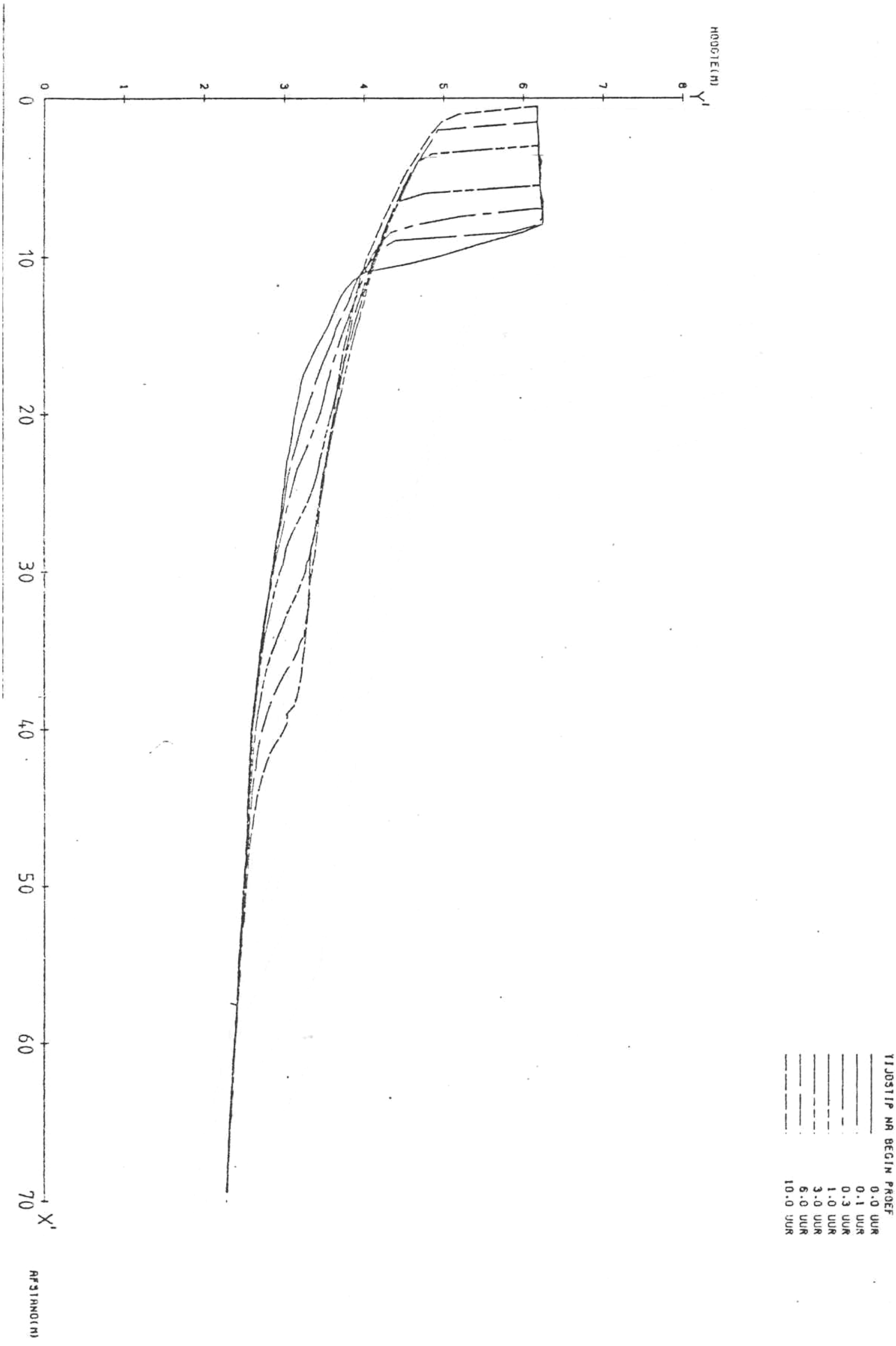


XIV. Delta Flume experiments on sand

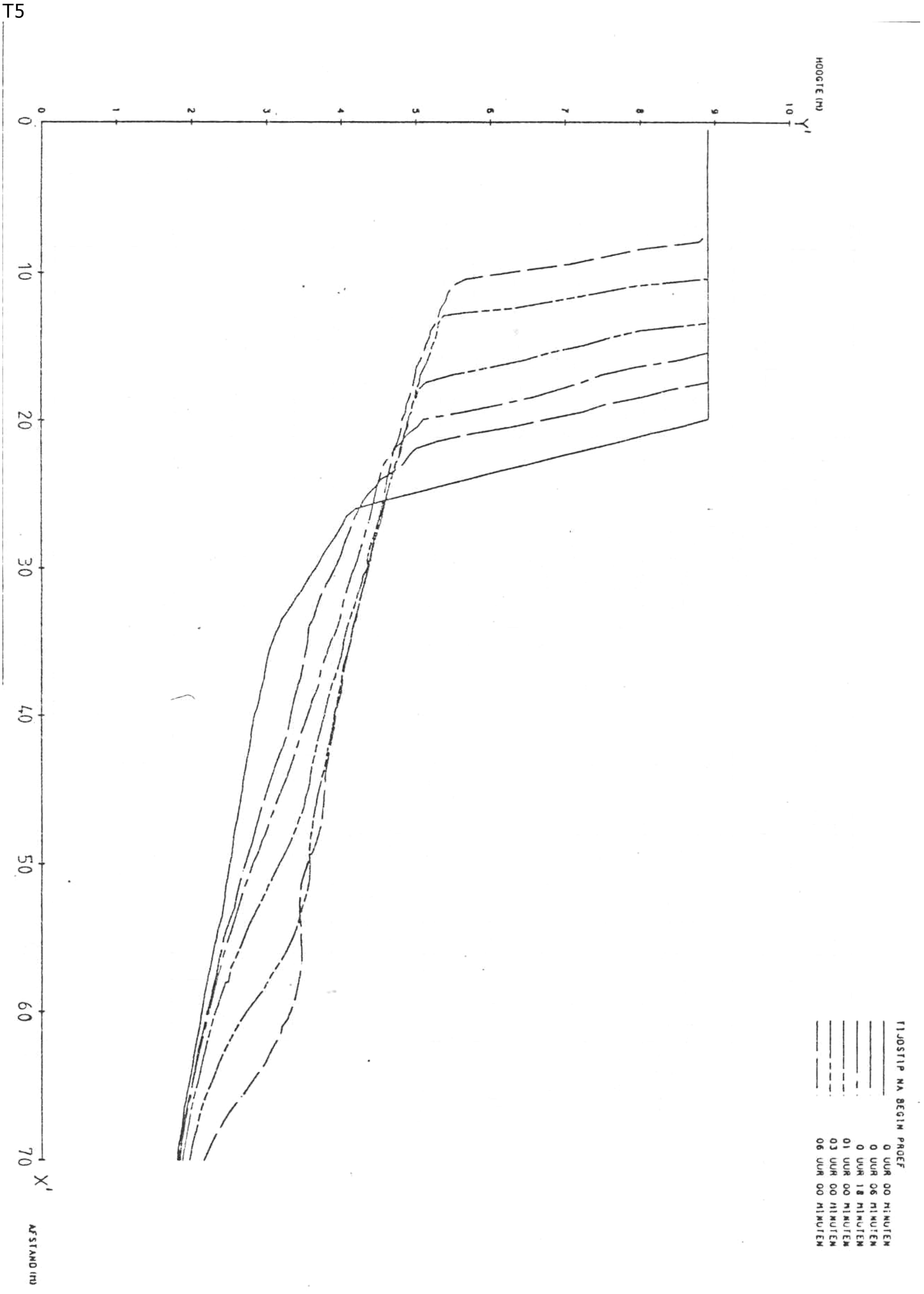
1981 T1



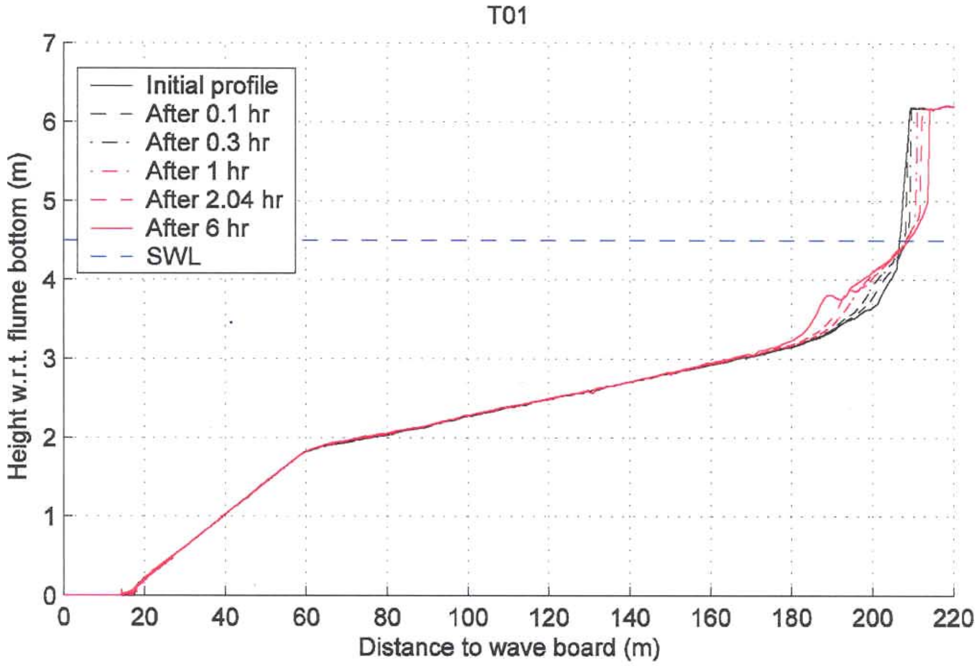
T2

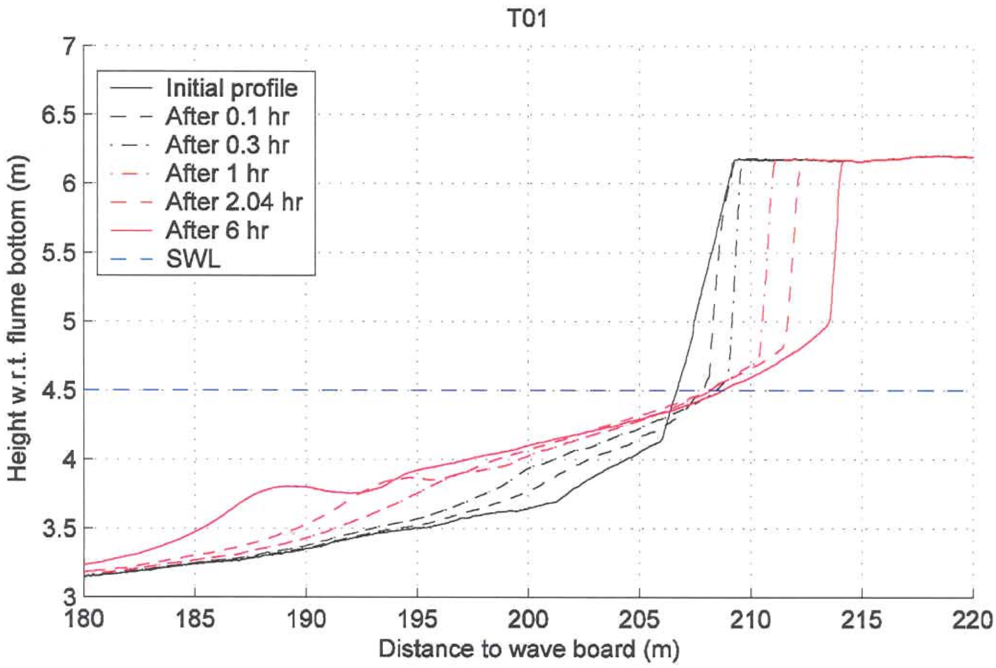

HOOGTE(M)
Y'
0
1
2
3
4
5
6
7
8
0
10
20
30
40
50
60
70
X'
AFSTAND(M)
TIJDSTIP NA BEGIN PROEF
0.0 UUR
0.1 UUR
0.3 UUR
1.0 UUR
3.0 UUR
6.0 UUR
10.0 UUR

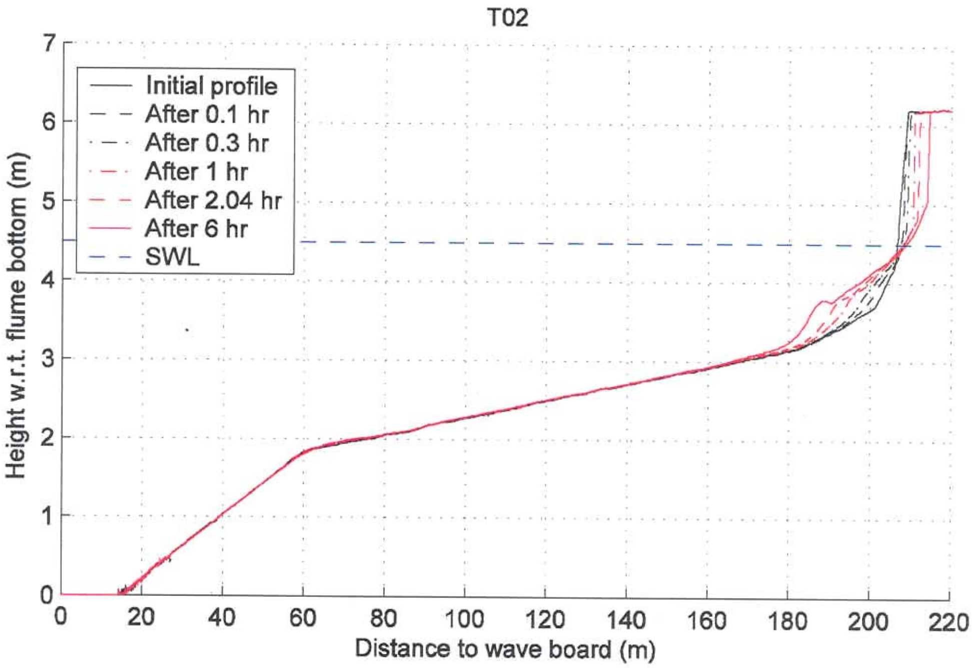
T5

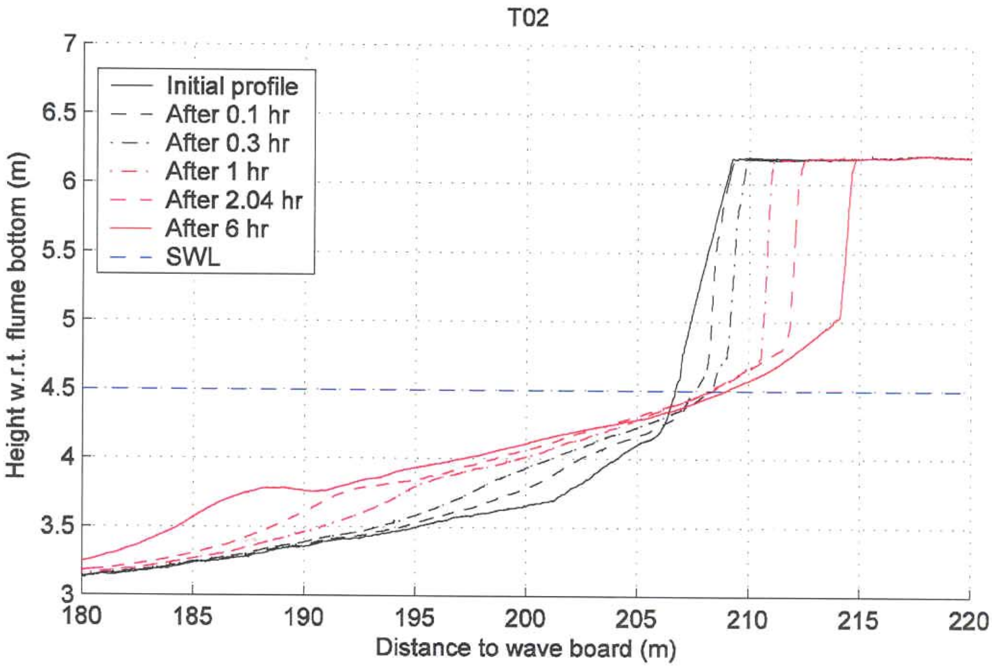

HOOGTE (M)
AFSTAND (M)
TIJDSTIP NA BEGIN PROEF
0 UUR 00 MINUTEN
0 UUR 06 MINUTEN
0 UUR 18 MINUTEN
01 UUR 00 MINUTEN
03 UUR 00 MINUTEN
06 UUR 00 MINUTEN

2006


T01
Initial profile
After 0.1 hr
After 0.3 hr
After 1 hr
After 2.04 hr
After 6 hr
SWL
Height w.r.t. flume bottom (m)
Distance to wave board (m)
0
1
2
3
4
5
6
7
0
20
40
60
80
100
120
140
160
180
200
220


T01
Initial profile
After 0.1 hr
After 0.3 hr
After 1 hr
After 2.04 hr
After 6 hr
SWL
Height w.r.t. flume bottom (m)
Distance to wave board (m)
3
3.5
4
4.5
5
5.5
6
6.5
7
180
185
190
195
200
205
210
215
220


T02
Initial profile
After 0.1 hr
After 0.3 hr
After 1 hr
After 2.04 hr
After 6 hr
SWL
Height w.r.t. flume bottom (m)
Distance to wave board (m)
0
1
2
3
4
5
6
7
0
20
40
60
80
100
120
140
160
180
200
220


T02
Initial profile
After 0.1 hr
After 0.3 hr
After 1 hr
After 2.04 hr
After 6 hr
SWL
Height w.r.t. flume bottom (m)
Distance to wave board (m)
3
3.5
4
4.5
5
5.5
6
6.5
7
180
185
190
195
200
205
210
215
220

XV. Storm surge disaster 1953, The Netherlands

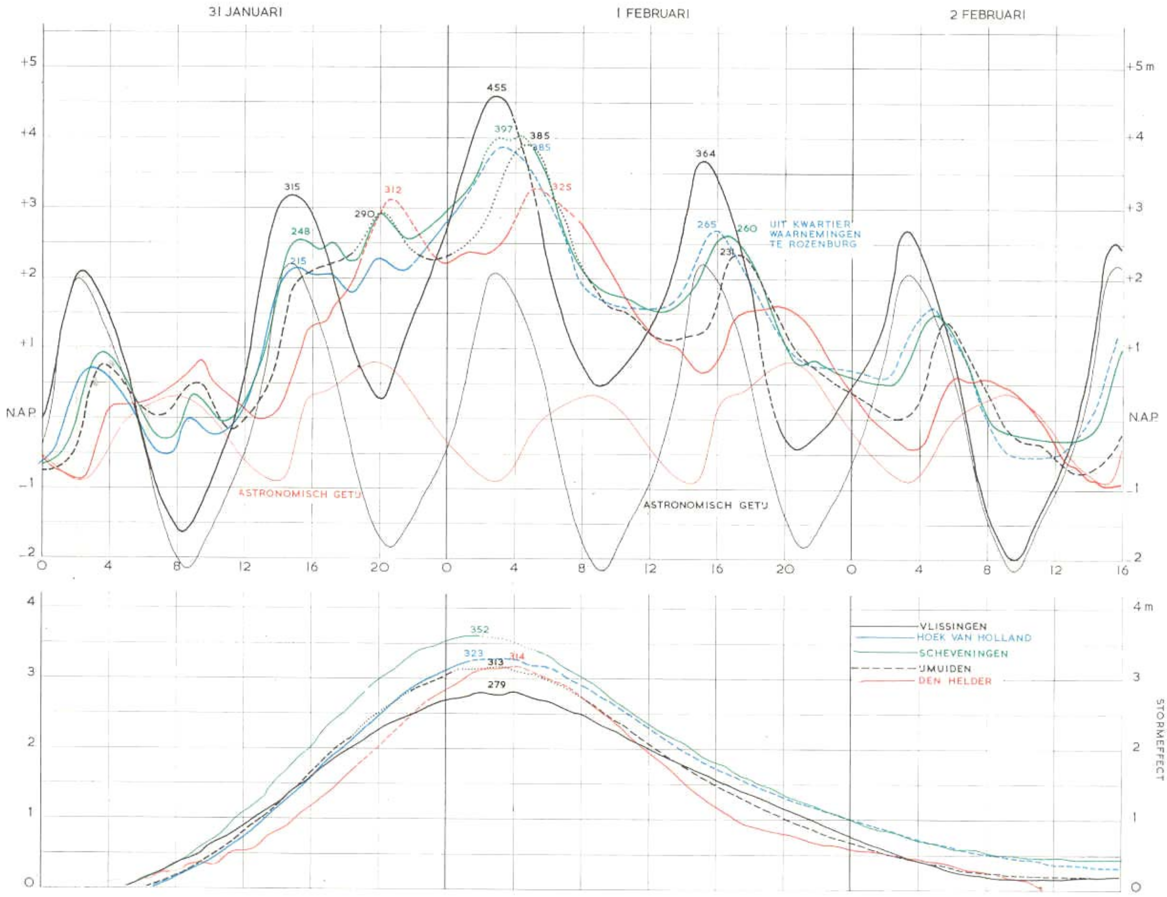
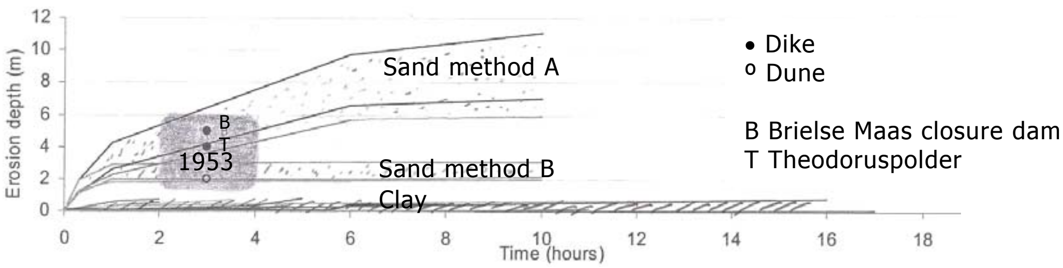


Figure XVI.1 Measured and predicted water levels during the storm surge disaster of 1953 [Rijkswaterstaat, 1961]

XVI. Comparison experiments with actual storms

Maximum Erosion Perpendicular to the slope
Comparison experiments on clay and sand with erosion in actual storm



Maximum Horizontal Erosion
Comparison experiments on clay and sand with erosion in actual storm

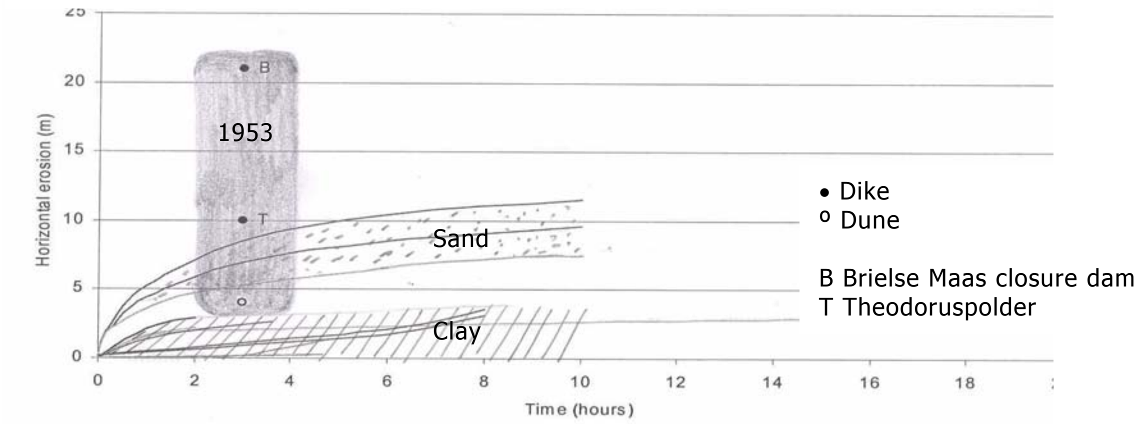


Figure XVI.1 Maximum erosion of the experiments on sand and on clay and the observed erosion after the storm surge of 1953.