Background and Literature review

Wave impact on asphaltic concrete revetments

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Master Thesis Literature Review

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March, 2009

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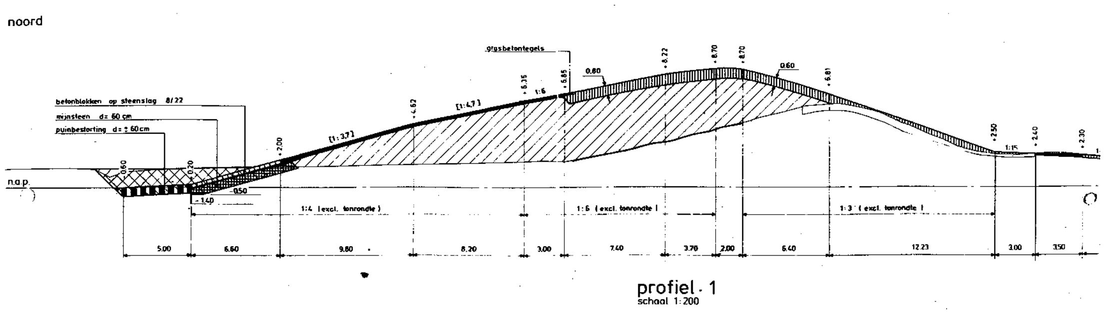
Introduction and problem description

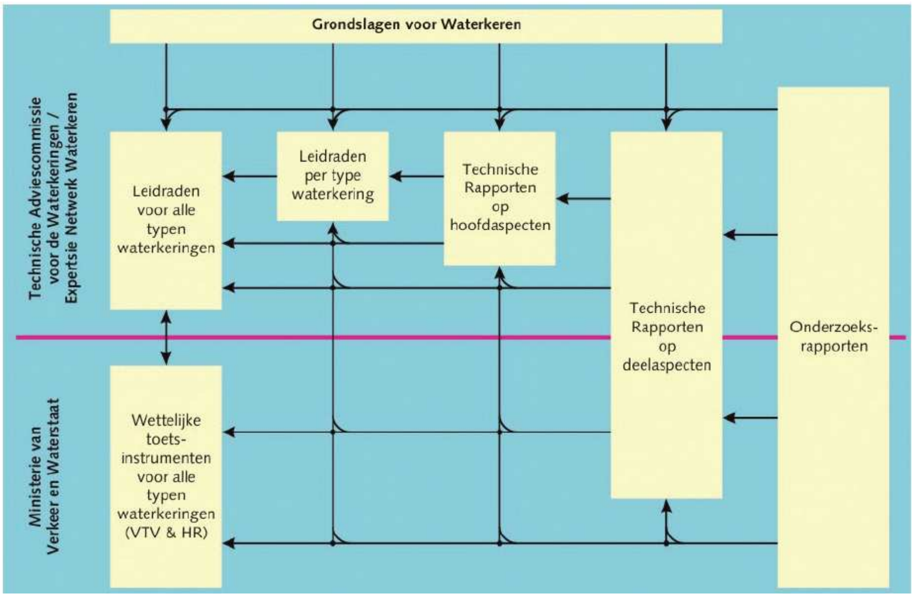
In the Netherlands for about 400 km of dikes are covered with an asphalt revetment. Some of these revetments are already 60 years old! The most of these asphalt revetments are constructed after the flood in 1953. Today when a revetment is older then 30 years a detailed assessment is made to get more knowledge about the quality of the asphalt layer.



Starting in 1996 each five years all the flood defences in the Netherlands have to be checked for safety against flooding. A part of this safety check is about wave impacts on an asphalt revetment. In this study that part will be extensively treated, especially the way the impact is schematized and taken into account.

Study of literature

In this survey a summary is given of the studied literature. The goal of the survey is to summarize the current knowledge about wave impacts. In the Netherlands after the flood in 1953 engineers and the government realized that a new approach was needed to make sure flooding could be prevented in the future. Money was invested in getting more knowledge about the specific failure mechanisms. Also advisory committees where founded who published the results of there findings in different guidelines and reports. These guidelines and reports represent the knowledge until that time, in this survey they are studied and summarized. With studying these guidelines and reports also knowledge is obtained about the development of the theories in time. In the next picture the different publications are summarized.



However to gain more specific knowledge about wave impacts studying only the results of the guidelines and reports gives not enough information. The guidelines and report do give information about the results of investigations only applied for practical use.

To study the development of the theory of wave impacts the literature survey to wave impact on dike slopes of van Vledder in 1990 is used. This survey is ordered by the Technical Advisory Committee to get more insight into wave impacts and its failure mechanism.

To study further development of theories the proceedings done at the International Conferences on Coastal Engineering are studied. Most of the investigations done in the world are published there and these publications are easy to obtain.

Furthermore the basic theory of the program GOLFKLAP, the program to asses the safety of asphalt revetments, is summarized.

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HydroBlock betonzuilen	Greenbanks erosion control, Appeltern, Netherlands
GOLFKLAP	KOAC•NPC, Nieuwegein, Netherlands

Part 1 Introduction

1 History of dikes in the Netherlands

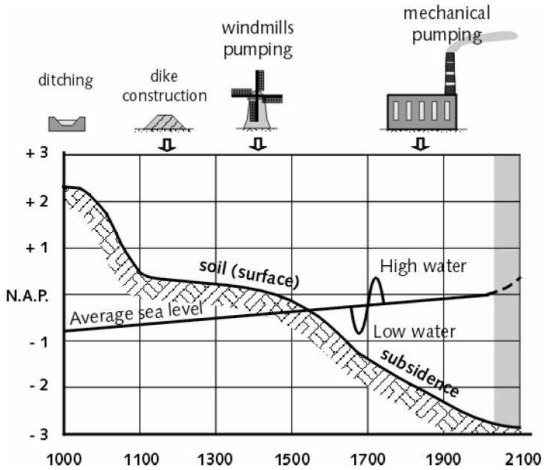
The rivers and the sea together created the Netherlands. When people came into the Netherlands they had to protect themselves and there belongings against the threat of the water. This protection developed over time into the many hydraulic structures as can be seen in the Netherlands nowadays. In this chapter some global background information about the geological structure of the Netherlands and the development of protection against the water is given.

1.1 Geological history

The geological structure of the Netherlands is formed after the last ice age. After this ice age the temperature rose and the ice caps started to melt. This melting resulted in a sea level rise which was the main cause of the way the Netherlands is formed. The North Sea was completely dry, forming a plain that connected England with the rest of Western Europe. With the rising sea level the coastline had to retreat. The sea penetrated the land through the valleys. The heights between the valleys were created by the ice cap, pushing the soil forward into ridges; the Veluwe is an example of these ridges. In the valleys marine sediment could be trapped and also the rivers deposited sand and clay. With the rate of the sea level rise declining the supply of sediment became sufficient to fill in the valleys and basins created by the sea.

1.2 Interference of people

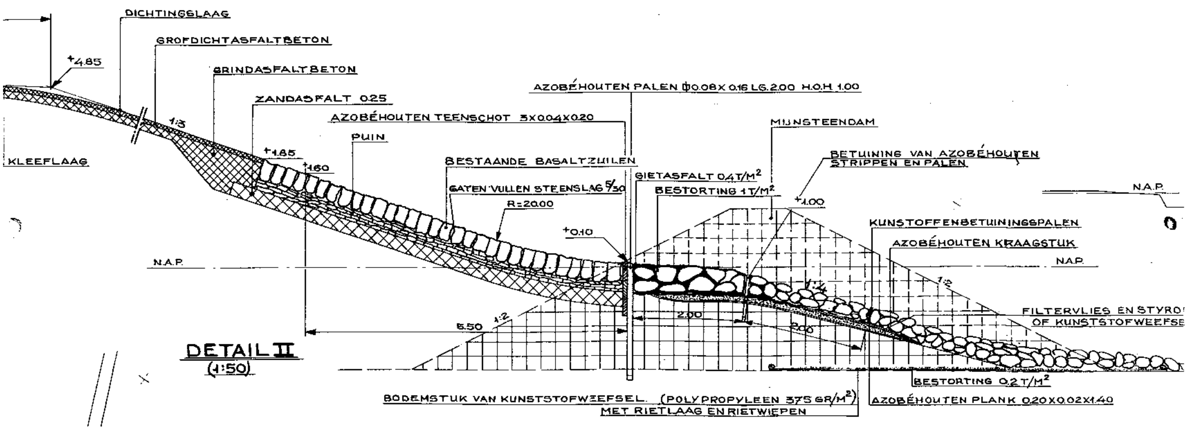
Around 1000 B.C. people start to interfere with the flow of the water and the flow of the sediment. From around this point till now subsidence of the land is an important factor in protecting the land against the sea and the rivers. Also with a growing population this protection became more and more important. At first people lived on high grounds. In the eleventh century the rate of population growth increased. People start to cultivate the land. Between the twelfth and the fourteenth century, the big cultivation began. Large peat areas were cultivated causing more and more subsidence of the land.



At the same time drainage systems were created. This caused even more land subsidence. While sea level rose and the land subsided lakes were formed. The sea and the rivers caused inundations and people start to build small hills to live on. This offered a relatively safe place to dwell but after some time more space was needed. At first dikes protected only the settlements and would have a horseshoe shape. The water flowing down the river would be diverted to a place where it would cause less damage. After some time these horseshoe shaped dikes were connected to each other creating dikes as we know them nowadays.

1.3 Defence

As the value of the properties grows also the interest in protecting these properties grows. People lived with the water and moved away when the high waters came. With creating dikes they staid and tried to withstand the water. Specialized organizations called the water boards got the task to protect the people living in there area. The height of the dikes was determined by the highest known water level.



However in the last century things started to change. With growing knowledge in mathematics and statistics the approach of protection became more and more scientific. People started to think more in a way where a load and strength could be determined. After the disaster in 1953 this way of thinking was implemented in the design and construction methods. Nowadays all our protection systems have to be checked every five years to come to, and maintain, our safety levels determined by our government. Basically there are two threats in the Netherlands. First, storms can lead to inundations. The wind can push up the water in the lake or the sea to such a high level that the dike system fails. Secondly, rainfall makes the discharge in the rivers higher. The dikes along the rivers can fail because of that. Besides high water, waves can be an important threat to our dike systems. In the next chapter more failure mechanisms are discussed.

1.4 References

d'Angremond, K. and Pluim-van der Velden, E.T.J.M. (2001) Introduction Coastal Engineering, *Delft University of Technology*

CUR (1997) CUR 190 "Probabilities in civil engineering, Part 1: Probabilistic design in theory", *CUR, Gouda*

TAW (1998) Grondslagen voor waterkeren, in Dutch (also in English; Fundamentals on Water Defences), *RWS-DWW, Delft*

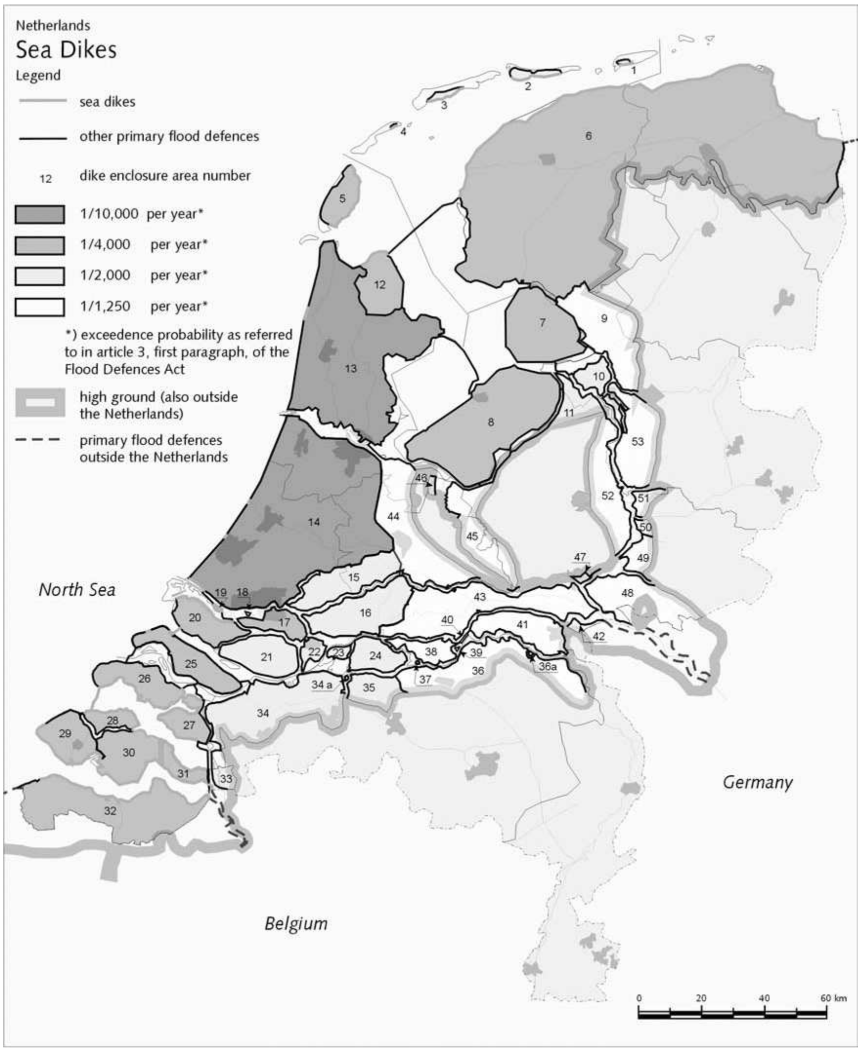
Tonneijck, M. and Weijers, J. (2008) Flood Defences, *Delft University of Technology*

2 Probabilistic design

In this chapter the safety approach used nowadays in the Netherlands, will be discussed briefly, this because the subject of this thesis is part of the safety assessment in the Netherlands. For an extensive study of the safety assessment the references at the end of this chapter are recommended.

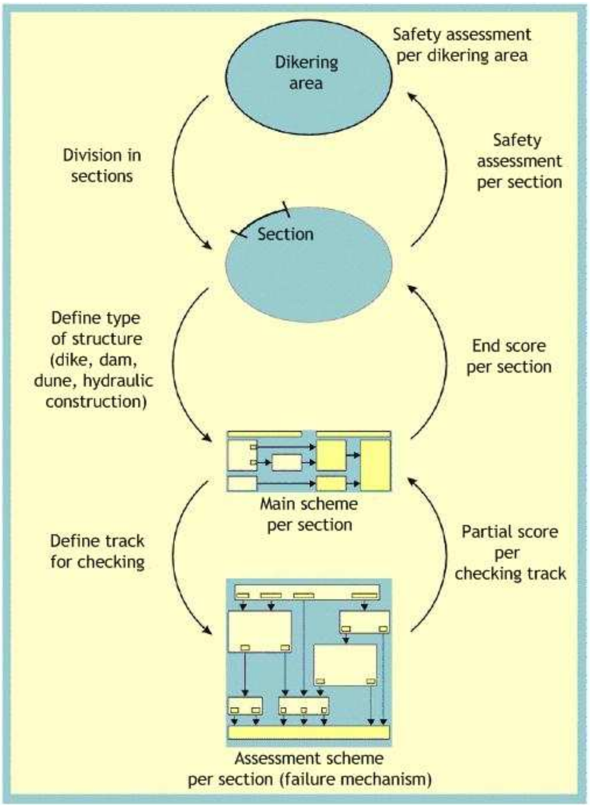
2.1 Safety

When making a safety approach the risk is a factor which is always used. Risk is defined as a probability times the consequences. Because the probability is dimensionless the dimension of the risk, according to this definition, equals that of the consequence. When determining the safety level first the accepted risk should be determined. In the Netherlands the government plays an important role in determining the safety levels. The country is divided into different dike rings each with its own safety level. This safety level is determined looking to the risk of inundation of a dike ring. In this way with everywhere the same acceptable risk the parts where the consequences of inundation will be high the probability of exceedance of the load is low. With the probability of exceedance determined one can start to determine the probability of occurrence of the loads and the probability of failure of the strength of the dike. In the next two sections is explained what approach is used in the Netherlands to calculate these probabilities.



2.2 Threats and loads

Nowadays in the safety assessment of the Dutch water retaining structures use is made of the hydraulic boundary conditions determined by the Dutch government. The Delta Commission started to determine these boundary conditions. An economical view was drawn up for the central part of the Netherlands. The economic most optimal safety level was determined at 1/125.000 years. Because it was difficult to determine a probability of inundation of 1/125.000 years the commission decided that a water level with probability of exceedance of 1/10.000 years should be withstood by the dikes. So for every dike ring this water level was calculated and processed in the safety assessment as boundary conditions. These boundary conditions are thus a collection of statistical descriptions of the hydraulic loads.



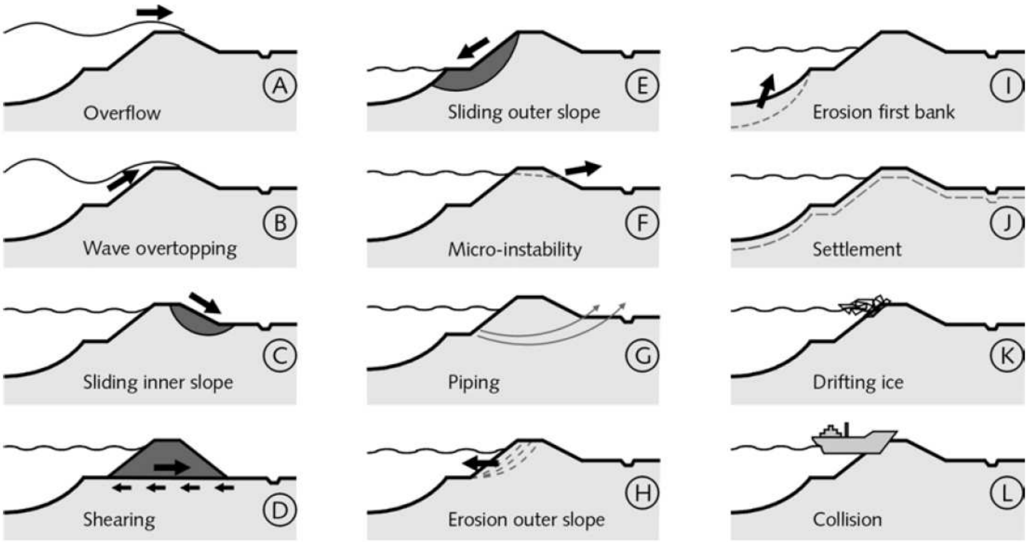
2.3 Failure mechanisms

Thinking about different mechanisms leading to failure started after the flood in 1953, this to determine the probability of failure of the whole system. The Delta Commission started with this approach and in time the assessment method became more sophisticated. It makes sense to make a difference between failure and collapse of the construction. The construction failed when it does not fulfil one or more of its functions. When the construction collapses the total construction is geometrically changed. The construction can fail without collapsing and vice versa.

When looking to failure and collapsing of the construction there are two sides to be considered. One side is the geometrical and material properties of the construction, the other side is the load upon the construction. Different loads can be distinguished.

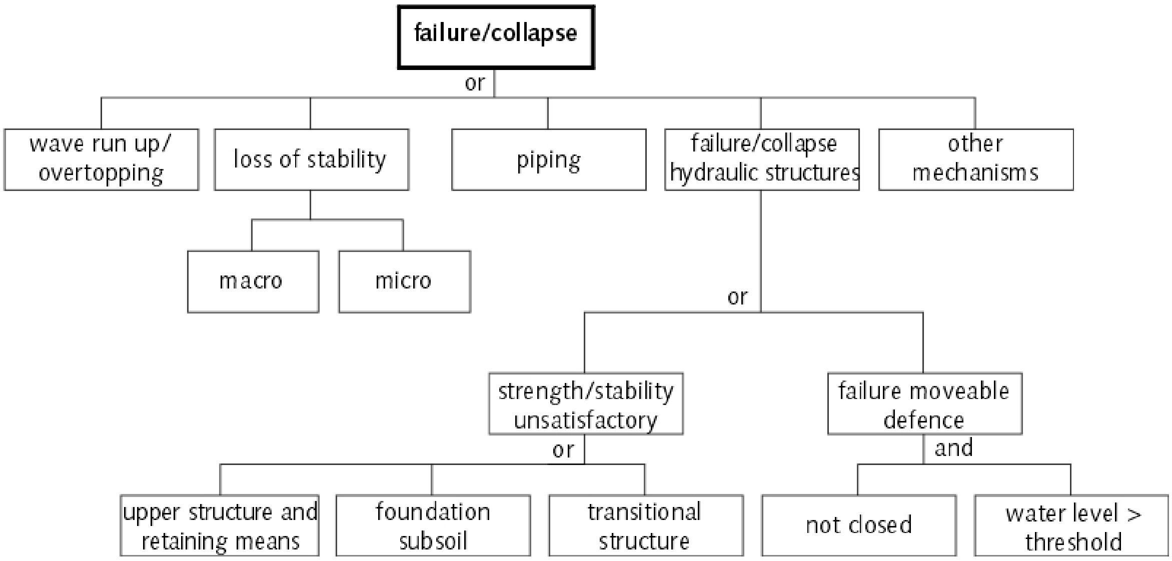
Permanent loads	own weight soil pressure
Variable loads	water level and flows waves rainfall wind temperature traffic flora and fauna
Special loads	collision ice load explosion/ earthquake

The main function of the construction is that it needs to retain the water. The way at which it cannot fulfil this function is called failure mechanism. In the picture below an overview of the different failure mechanisms according to "Grondslagen voor Waterkeren" which is used in the Dutch safety assessment.



- (A) inundation of the dike ring without collapse of the water retaining construction
- (B) erosion of the inner slope by flowing water
- (C) instability of the inner slope or by overflowing water or by high pressures in the dike body
- (D) sliding of the whole dike body, because of high water or high water pressures in/under the dike body
- (E) slide of the outer slope because of quick drop of the water level after high water
- (F) instability of the inner our outer slope because of seepage through the dike body leading to micro instability
- (G) piping, micro instability in or under the dike body
- (H) (I) erosion outer slope or first bank
- (J) settlement of the total dike body
- (K) (L) mechanical threats by ice or shipping

To show the relations between the failure mechanisms, the different mechanisms can be placed in a fault tree.



By assigning a probability of failure to each of the components of the fault tree insight is gained about the probability of failure of the top event. Also the weak spots in the construction become clear in this way of approach. The values of these loads are processed in the hydraulic boundary conditions mentioned earlier.

2.4 Reliability function

To come to a final safety assessment the difference between the strength and the load has to be determined. To test whether a construction is safe two different limit states are defined. The ultimate limit state (ULS) and the serviceability limit state (SLS). When the water retaining function of the primary structure is threatened the ultimate limit state is reached. The serviceability limit state is reached when a part of the primary water retaining structure fails. This means that in the serviceability limit state no inundations occur but that repair works are needed. To determine if a construction reaches the ultimate limit state a function Z is defined as: Z = Strength - Load. For every failure mechanism the function Z can be determined. When Z<0 the construction fails to fulfil its function.

2.5 References

CUR (1997) CUR 190 "Probabilities in civil engineering, Part 1: Probabilistic design in theory", *CUR, Gouda*

RWS (2007) Hydraulische Randvoorwaarden 2006 voor het toetsen van primaire waterkeringen (in Dutch)

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3 Revetments

3.1 Introduction

In this chapter the functions of revetments and the difference between revetments is treated. In chapter two is already dealt with the main function of a revetment. In the past the revetment on the dike is placed to strengthen the dike, this to prevent flooding. Nowadays a revetment can have more functions:

- traffic
- landscape/ ecology
- recreation

For maintenance some parts of the revetment need to be accessible for traffic. In the design of the revetment one has to reckon with the load of the maintenance vehicles, part of the revetment may also be accessible for bicycles. The dike is part of the landscape; the revetment can play an important role in fitting the dike into its environment. Also when this environment is attractive for recreation the revetment has to be accessible. In this report only the main function of the dike is treated.

If a protection system is threatened there are five main options available:

- do nothing

With a threatened system in time the function of protection will be lost.

- take away the cause of the problem

This means that in the ULS function the part with the loads have to be taken away.

- keep repairing the part that is threatened

So in this way the strength in the ULS function stays the same.

- reduce the loads

In the ULS function the part with the loads is reduced.

- increase the strength

In the ULS function the part with the strength is increased.

For every part of the system a choice has to be made, looking to the costs and the benefits (on the long term) with safety as an important factor. In this way our ancestors came up with all kind of ideas to make sure this ultimate limit state was never reached. To increase the strength they placed revetments on the dike. In history all kind of revetments were developed each with its own advantages and disadvantages. The availability of the material was in the past even more an important factor because transportation was more costly, so at different places different revetments were developed.

Today the revetment is that part of the dike that covers the core of the dike. The revetment can consist of one or more layers. In the Netherlands the sub-layer most of the times consist of clay. The sub-layer is defined as the layer with a material which differs from the core material and fulfils another function than that of the top-layer. The function of a revetment for protection can be:

- protection of the underlying soil body
- reduce the wave run up
- contribute to the making the structure impervious
- reduce maintenance
- give the water retaining structure a aesthetic/ natural look

The revetment most of the time has a combination of these functions.

3.2 Different revetments

Slope revetments may be divided into four different main categories.

- natural material (clay and grass)
- loose units (gravel, riprap)
- interlocking units (concrete blocks and mats)
- concrete and asphalt slabs

The difference in resistance against the loads between these categories is in friction, cohesion, weight of the units, friction between the units, interlocking and mechanical strength. In the next table these differences are summarized.

type of coverlayer	critical failure mode	determinant wave loading	strength
clay/grass	• erosion • deformation	• max. velocity • wave impact	• cohesion • grass-roots • quality of clay
rip-rap	• initiation of motion • deformation	• max. velocity • seepage	• weight • friction • permeability of sublayer/ core
gabions/ (sand-, stone-, cement-) mattresses including geotextiles	• initiation of motion • deformation • rocking • abrasion/ corosion of wires • U V light	• max. velocity • wave impact • climate • vandalism	• weight • blocking • wires • large unit • permeability including sublayer
placed blocks including block-mats	• lifting • bending • deformation • sliding	• overpressure • wave impact	• thickness, friction, interlocking • permeability including sublayer/ geotextile • cabling/pins
asphalt	• erosion • deformation • lifting	• max. velocity • wave impact • overpressure	• mechanical strength • weight

In general when designing a dike a specific choice has to be made between the different revetments. Because at each part of the dike another load is present dikes consist of different revetments. The transitions between these revetments are sensitive for damage. When designing a revetment extra attention has to be paid to these transitions. A big difference between revetments is the permeability, when a choice has to be made between revetments this property plays an important role, this in combination with the availability of other materials.

3.2.1 Clay and grass

When wave impacts are small, a grass revetment or vegetation with good cover can be an attractive solution also from an economical point of view. But for a grass revetment, another material is necessary because grass can not live under water permanently, so another material should be applied to the low part of the revetment.

3.2.2 Loose rocks

Below mean water level loose rock is applied on many places in the Netherlands. Loose are easy to apply and maintenance cost are low.

3.2.3 Mattresses

Composite mattresses can be applied on large stretches of bank. Most of the time special equipment is needed to place the mattresses. When this is the case the placement costs are so high that for small scale projects it is not feasible to use it as a revetment. When no special equipment is needed also small stretches re covered with composite mattresses.

3.2.4 Placed blocks

Placed blocks on a slope, when placed well, have a nice regularly appearance, however good appearance needs more maintenance. After a storm some elements may be removed and have to be replaced.

3.2.5 Asphalt

Asphalt revetments are relatively strong and able to suffer heavy wave loads. When wave attack is severe, asphalt is a solution. To place an asphalt revetment special equipment is needed, with this equipment it can be placed very fast. If it is well designed constructed and maintained it is very reliable. This thesis will treat this type of revetment extensively, in chapter four more about the history and in chapters five and six more about the properties of asphalt is treated.

3.3 References

RWS (2007) Hydraulische Randvoorwaarden 2006 voor het toetsen van primaire waterkeringen (in Dutch)

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Part 2 Asphaltic revetments

4 History

In this chapter the development of asphalt as a product used in hydraulic engineering is treated. Nowadays asphalt is used for almost a century as a revetment in Dutch practice and over time more knowledge is obtained and processed in all kind of books and papers.

4.1 History in development

The first time asphalt is used in hydraulic engineering in The Netherlands is around 1935. The first development of this hydraulic use was mainly in the field of waterproof linings. Today bitumen is produced by the refining of crude petroleum oil. In the past people used bitumen which came from surface deposits. These surface deposits were located at places where oil was formed in the crust of the earth. When the oil containing layer came to the surface the lighter fraction of the oil evaporated leaving the heavier non-volatile residue in the limestone or sandstone rock. The bitumen was extracted from the rock by melting down. Then the bitumen was used wherever brickwork was liable to damage by rain or running water. One example of this use is in the embankment of the Tigris. After 3000 years it still fulfils its purpose!

In Europe no surface deposits of bitumen were available so it had to be imported by ships. The dawn of the modern asphalt industry can probably be dated in the early eighteenth century, when efforts were made to develop the use of waterproofing and other purposes of the Neuchatel (Switzerland) rock-asphalt deposits. The first use of rock-asphalt in European cities was in the early nineteenth century. It was used for surfacing streets and footpaths. In 1870 the first synthetic asphalt street surface was laid in de U.S.A.

With the rise of the oil industry, because of developed drilling methods, from 1860 onwards a large new source of bitumen came into existence. Because of the success of applying bitumen in roads, hydraulic purposes were derived. In the late nineteen-twenties bitumen was made subject to systematic investigation as a material for hydraulic construction.

In the Netherlands the use of bitumen ascends to a higher level after the storm in 1953. After this storm a lot of dikes had to be repaired. Because the materials tradionally used became scarce the use of asphalt on dikes became interesting. Also because the use of asphalt was not labour intensive and asphalt was quickly to apply, lots of dikes and dames were covered with an asphalt layer.

4.2 History of the literature

All this triggered the research to and documentation of the use of asphalt in hydraulic engineering. One of the first books about asphalt in hydraulic engineering is made by baron van Asbeck. This book is published in 1955 by Shell International. It covers a wide range of subjects from laboratory tests to practical applications. In 1961 the study group "gesloten dijkbekledingen" (Closed dike revetments) published a report named; "voorlopig rapport" (Temporarily report). The study group was assembled to investigate what factors were important to ensure the state and durability of asphalt revetments. This report covers the subjects:

- calculation of the strength of the material,
- durability of the material,
- design, construction and experience with the material.

Shell International published the second volume of the book Bitumen in Hydraulic Engineering in 1964. It basically is an extension of the first volume, as the name implies. In this version more examples of practical applications are shown. Also more physical properties of bitumen and bituminous mixes are treated.

In 1984 the Technical Advisory Committee for hydraulic structures published the guideline for application of asphalt in hydraulic engineering, this because there had been a lot of new development since 1961. The following subjects are treated in this guideline:

- material properties
- material used in hydraulic engineering
- designing and calculating aspects
- construction
- management and maintenance

In 1999 the Shell Company published the Shell Bitumen Hydraulic Engineering Handbook. In this book al the continued development in the field of bituminous products till then applied in hydraulic engineering is treated.

The Technical Advisory Committee again published a report for constructing hydraulic structures with the use of asphalt. This because users of the guideline of 1984 and other involved parties asked for a new simple and handy book in which the actual knowledge was treated. As an answer to this question in 2002 the technical report, asphalt for water defences is published. Again all the subjects mentioned before are treated. An important subject in this report is the management and maintenance part. This because most projects with the use of asphalt as constructing material are already finished.

The Coastal Engineering Manual briefly discusses the use of asphalt. To quote a part of the manual;
"**Dikes**. Although asphalt is not widely used in the United States for coastal protection structures, the Dutch have made good use of asphalt to protect the slopes of earthen dikes."
Furthermore, the manual mentioned asphalt layer as an adequate revetment to use when wave action is slight. In the manual the main properties of asphalt are treated, about the durability; "Asphalt it not considered to be a durable material because it has low strength in both compression and tension, it is subject to chemical reaction, its stiffness changes with temperature, and it is not resistant to impact or abrasion."
The flexible property of asphalt when exposed to settlements is embraced but the text shown above does not invite the design of an asphalt revetment.

Also in other countries several reports has been published. Some examples of these are:

- the British Standard (1989) 3690, Part 1
- DGGT EAAW 83 (1983) Empfehlungen für die Ausführung von Asphaltarbeiten im Wasserbau (Recommendations for the execution of asphalt works in hydraulic engineering). Deutsche Gesselschaft für Geotechnik, Auflage 1983 (4th edition 1983) reprint 1996
- Kerkhoven, R.E. (1975) Recent developments in asphalt techniques for hydraulic applications in the Netherlands

Lots of other books and papers are published, most of them with a more specific part of application.

4.3 References

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TAW (2002) Technisch Rapport Asfalt voor Waterkeren (in Dutch), *RWS-DWW, Delft*

U.S. Army Corps of Engineers (2001) Coastal Engineering Manual, *Washington DC*

Werkgroep „Gesloten dijkbekledingen" (1961) Voorlopig Rapport 1961 (in Dutch)

5 The material

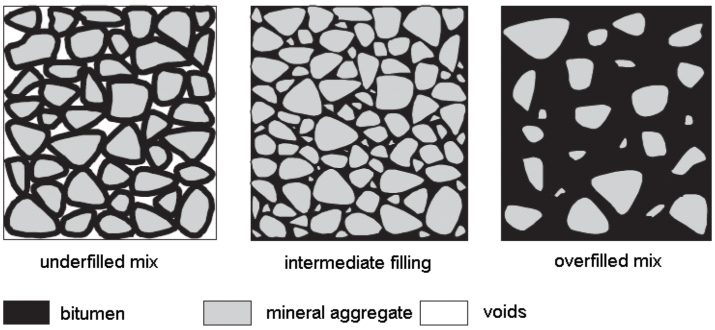
Asphalt is a mix of bitumen and mineral aggregate each with its own properties determining the properties of the asphalt. In this section the properties of the bitumen and mineral aggregate will be explained.

5.1.1 Bitumen

Bitumen is used as a binder of the mineral aggregate. It is refined from crude oil and has to meet certain national specifications for paving, industrial use and other purposes. The definition in the TAW technical report for asphalt in water retaining (see references) is; "a very viscous liquid or solid substance, mainly consisting of hydrocarbon or its derivatives, almost entirely dissolvable in sulphur carbon". Mostly used in hydraulic engineering is bitumen 70/100. This 70/100 is a measure for the penetration of a needle in the bitumen in a standardized test. In the Netherlands the requirement with respect to the binder are stated in EN 12591. To process the bitumen it is heated and mixed. Another possibility is adding an emulsion and water, the emulsion to prevent cluttering of the bitumen.

5.1.2 Mineral aggregate

The mineral aggregate can consist of crushed stone, gravel, round or crushed sand and filler. The skeleton of material provides the bearing strength. The bitumen glues the material together and fills the holes in the skeleton with the smaller aggregates. The holes that are left over and filled with air are very important for the properties like permeability and durability.



5.2 Properties

When choosing what kind of asphalt should be applied the properties of asphalt are important factors. The most important properties are:

- permeability
- mechanical properties
- durability
- process ability

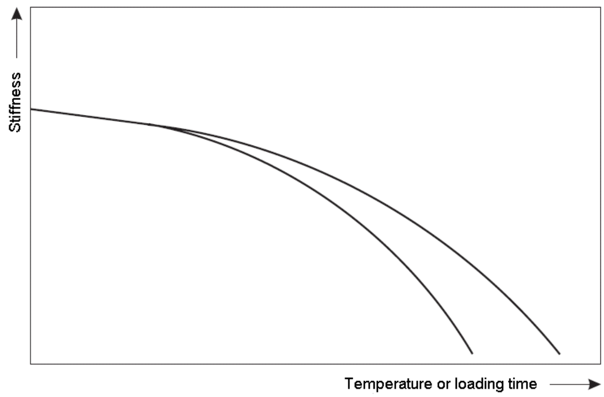
In the next paragraph these properties are treated.

5.2.1 Permeability

As mentioned before the holes left over determine for a large part the permeability. The amount of holes and how well they are connected with each other are a measure for permeability. Most revetments need at least to be sand tight but most of the times also watertight.

5.2.2 Mechanical properties

Stiffness and strength are two very important properties. For asphalt they change with temperature and loading time. In hydraulic engineering both temperature and loading time do vary a lot. In the Netherlands the asphalt temperature can change from some degrees Celsius below zero till fifty degrees above zero. The loading time can vary from 0.1 second (peak loads due to wave impacts) to several years (settlement of the surface). Asphalt is stiff and strong when short loaded and with low temperatures. At high temperatures and with long loading time it behaves as a viscous material thus flexible and weak.



Stiffness is a measure between loading and deformation and is defined as a quotient between tension and relative deformation. The stiffness is also dependent on the amount of voids in the material. Strength is a measure for the maximum load the asphalt can resist. The strength to resist bending is the most important to endure wave impacts. Other mechanical properties are stability and flexibility. Stability poses no problem in Dutch practice when applied on slopes which are not too steep. Also the flexibility is no problem for the mixture of today. Only when large deformations occur the asphalt layer cannot follow the deformation.

5.2.3 Durability

Durability is the extent at which the relevant properties stay at the desired level. Because some asphalt revetments in the Netherlands are already seventy years old checking this property is a hot item at the moment. The factors influencing the durability are hydraulic loads (wave impact, currents), the weather (temperature, temperature differences and UV), the surroundings (oxygen, chemicals and living organisms) and the construction (heat).

5.2.3.1 Hydraulic loads

A short summary of the loads:

- water pressure
- uplift
- air pressure
- wave attack
- current

Also ships and ice forms a threat to an asphalt revetment. Fatigue is something what plays an important role in the designing and testing process of asphalt. In the Netherlands this aspect only is important when the asphalt revetment is exposed to a design storm. Under normal storm conditions there is hardly any wave attack on the asphalt revetment. After the storm the micro cracks in the layer disappear when the layer is heated up by the sun in summer time, this process is called healing.

5.2.3.2 Weathering

In time the bitumen in the asphalt mixture becomes harder. Because of ultra violet light the oxygen take-up is advanced, this results in ageing and hardening. Warmth encourages the evaporation of lighter fractions which also results in ageing and hardening of the bitumen. All this makes the asphalt less flexible what induces forming of cracks. Frost in conditions like in the Netherlands does normally not cause damage to the asphalt layer.

5.2.3.3 Surroundings

Obviously the surroundings influence the bitumen; some of them are already mentioned. Damage by chemicals also can occur. The chemical itself has to contain a higher concentration of hydrocarbon then the bitumen. In most cases the concentration is lower so damage because of this hardly occurs; only at places where oil is treated this can cause problems. Living organisms like plants or marine pocks can cause damage when the layer of asphalt is not thick enough. The layer of revetment should be at least 12 to 15 cm thick to prevent the roots to grow through the layer. Even then good maintenance is needed to prevent reed and other woody plants to damage the construction.

5.2.4 Process ability

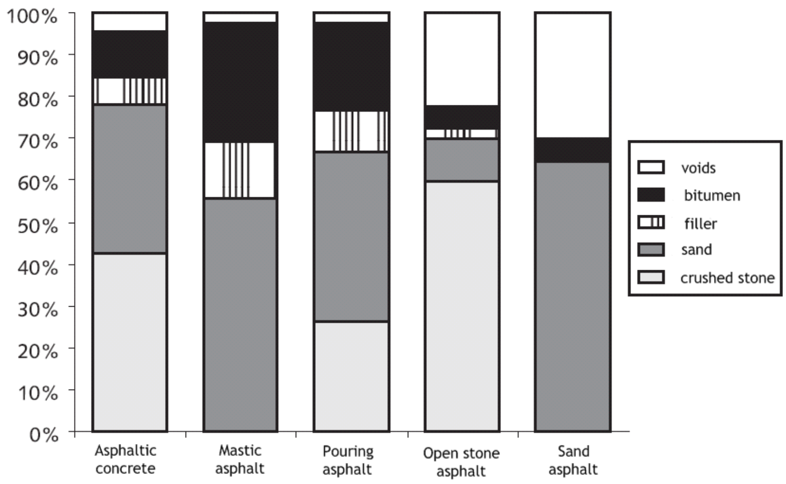
The asphalt is heated to process it into a layer on a dike. For every type of asphalt there is an optimum temperature for processing. In the technical report for asphalt structures these temperatures are stated. Most of the times an earth moving machine brings the asphalt on the slope and a hydraulic crane is used to spread the asphalt over the slope. After spreading, a roller is used to compact the layer, this to increase stability.

5.3 Different types

The different types of asphalt revetments in the Netherlands are:

- asphaltic concrete - hydraulic type
- mastic Asphalt
- dense stone asphalt
- open stone asphalt mats (with or without enforcements)
- open stone asphalt
- sand asphalt
- broken stone, penetrated with asphalt
- brick stone/ concrete bricks/ basalt penetrated with asphalt

In the next picture the percentage of components per volume are given.

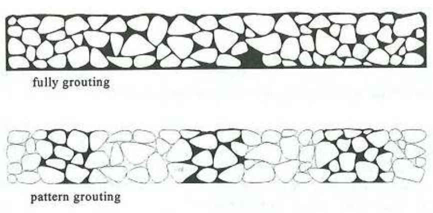


Asphaltic concrete is a 'nearly filled' mix of crushed stone, sand, filler and bitumen. The bitumen content on 100% mineral is 6.5% (mass). Asphaltic concrete is considered to be sufficiently watertight when the voids ratio is less than 8%. To obtain sufficient durability the maximum void ratio may not exceed 6%. The asphalt has to be compacted mechanically and can therefore not be placed in the tidal zone or under water. After construction the revetment is covered with a seal coat consisting of a sprayed layer of bitumen emulsion blinded with chipping.

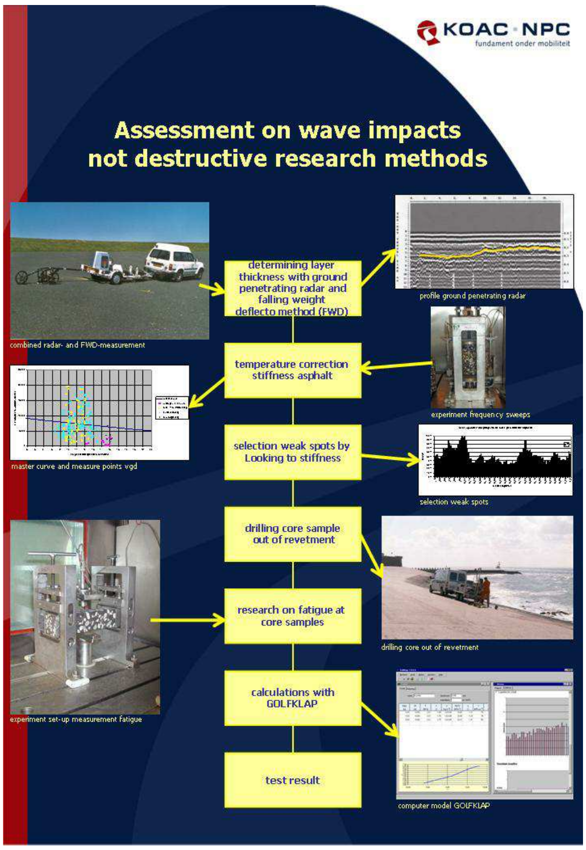
Generally in bank and dike protection the revetment is placed in one layer. The reason for this is that a proper cohesion between layers might be obstructed by sand blown in between the layers while constructing the revetment.

Asphalt mastic is an overfilled mix of sand, filler and bitumen. It has the properties of a highly viscous fluid. The mix design is therefore based to obtain the right viscosity. Requirements for the mix design are that the mix has to be workable, which means pourable, during execution and after cooling down that the flow is kept within certain limits. Mastic is used in asphalt slabs above and under water, as lining and bed- and toe protection.

Grouting mortar is a mastic to which gravel is added to control the penetration behaviour. Grouting mortars are used to grouted stone revetments above or below the water level. Grouting can be applied in different ways.




Open stone asphalt is mixed in two stages. First a mastic is produced which is then mixed with crushed stone (limestone 20/40 mm or 16/22 mm). A new development is that fibres are added to the mastic to increase the resistance against sagging of the mastic. A very open mixture is obtained which is water but also sand permeable. A filter construction is therefore necessary.



Sand asphalt is a mixture of sand and 4 to 5 % bitumen. It is water permeable but sand impermeable. Lean sand asphalt is used for filter layers, temporary slope revetments and as a core material.

5.4 Assessment method

To test whether the asphalt properties are as required several test are available. In the Dutch guidelines (*RWS* (2007)), is made reference to a report which describes how the checks are done. This report will be shortly summarized in this section. The whole report is directed towards the assessment of safety against wave impact.

In this assessment the following subdivision is made:

- preparation
- perform fieldwork
- analyze data fieldwork
- perform laboratory investigation
- assessment of the revetment using the program GOLFKLAP

In the preparation phase as much information as possible about the state of the revetment is gathered. Also information about construction, the used material, visual inspections and geometrical information is gathered. When this is done a plan for measuring is made. Advised is to measure with a grid distance of fifty meters when two lines of directions are used. The fieldwork consists of the following activities:

- perform ground penetrating radar (gpr) scans to determine the layer thickness of the revetment
- perform falling weight deflector (fwd) to determine the stiffness of the top layer and the sub grade
- measure the temperature of the asphalt during fwd measurements
- drilling cores to calibrate gpr data and to determine the relation between stiffness and temperature by performing laboratory tests
- analyzing data, determine weak spots of revetment
- drill cores out of the revetment on weak spots
- compare the radar scan with the measured layer thickness
- performing fatigue tests
- determining the resistance against wave impacts with the use of GOLFKLAP

5.5 References

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6 Relation between stress and deformation

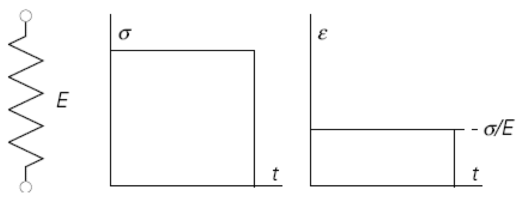
This chapter deals with more specific behaviour of asphalt. Chapter six of the book; "*Van der Vegt, A.K. and Govaert, L.E.* (2005) Polymeren, van keten tot kunststof" is summarized in the first four sections. In the first three sections the viscous-elastic behaviour of asphalt is treated. The procedure taken when asphalt is tested is already mentioned in section 5.4, at the end of this chapter the way the parameters are obtained for this testing are discussed but first the viscous-elastic behaviour of asphalt is explained.

6.1 Linear elastic behaviour

The linear-elastic behaviour of an ideal solid mass can be described by Hooke's law:

$$\sigma = E\varepsilon$$

where σ is the stress, ε the strain and E the modulus of elasticity. The modulus of elasticity is a material constant dependent on the material temperature. The material can be schematized by a spring. The response of a deformation follows immediately.

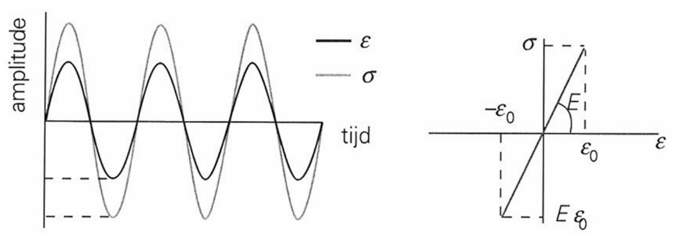


When the material is dynamically deformed with:

$$\varepsilon(t) = \varepsilon_0 \sin(\omega t)$$

The response of the material will be:

$$\sigma(t) = E\varepsilon_0 \sin(\omega t)$$



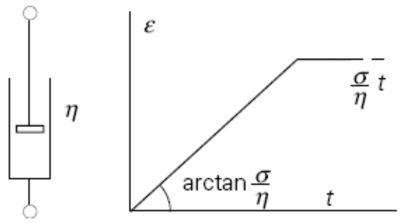
This means there is no phase difference between the deformation and the stress. No energy is dissipated.

6.2 Viscous behaviour

Viscous behaviour for an ideal liquid can be described with Newton's law:

$$\sigma = \eta \frac{d\varepsilon}{dt}$$

The stress is linear dependent on the dynamic viscosity ($\eta = \rho\upsilon$) and the rate of deformation. This can be represented by a dashpot which is characterized by η.

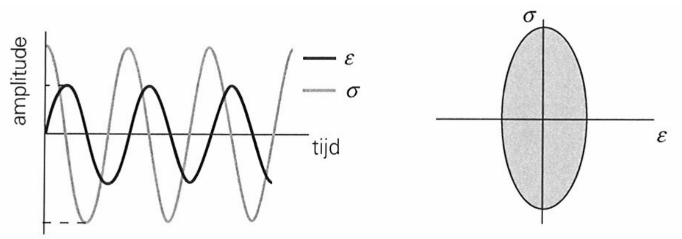


When the material is dynamically deformed as:

$$\varepsilon(\mathrm{t}) = \varepsilon_0 \sin(\omega \mathrm{t})$$

the response of the stress will be:

$$\sigma(\mathrm{t}) = \eta\omega\varepsilon_0 \sin(\omega \mathrm{t} + \frac{\pi}{2})$$



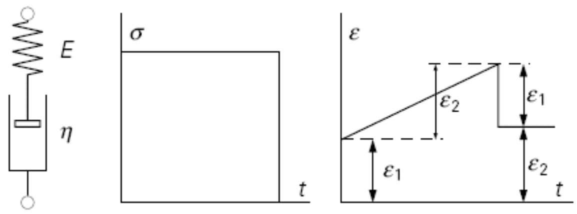
The deformation and the stress are out of phase by π/2 and all the energy is dissipated

6.3 Linear viscous-elastic theory

To describe the time dependence response of a viscous elastic material a mathematical method is available; the classical linear viscous elasticity theory. Two main assumptions are made:

- proportionality
- superposition

Proportionality means the response of a viscous material is always proportional with the deformation. This means when the stress is doubled also the deformation is doubled.
Superposition means that the stress that follows after several deformations is the sum of the stresses for each individual deformation. Maxwell developed a model based on these assumptions to simulate viscous elastic behaviour.



$$\varepsilon = \varepsilon_1 + \varepsilon_2 = \frac{\sigma}{\mathrm{E}} + \frac{\sigma}{\eta}\mathrm{t} = \sigma[\frac{1}{\mathrm{E}} + \frac{\mathrm{t}}{\eta}]$$

This formula represents creep in the material that occurs when the material is constantly loaded. What also has to be considered is stress relaxation which occurs when the deformation is kept constant. Now the case that the stress in the string and the dashpot are the same is considered:

$$\sigma_1 = E\varepsilon_1 = \sigma_2 = \eta\frac{d\varepsilon_2}{dt} = \eta\frac{d(\varepsilon-\varepsilon_1)}{dt} = -\eta\frac{d\varepsilon_1}{dt} \text{ With: } \varepsilon_1 + \varepsilon_2 = \varepsilon$$

Gives:

$$-\eta\frac{d\varepsilon_1}{dt} = E\varepsilon_1$$

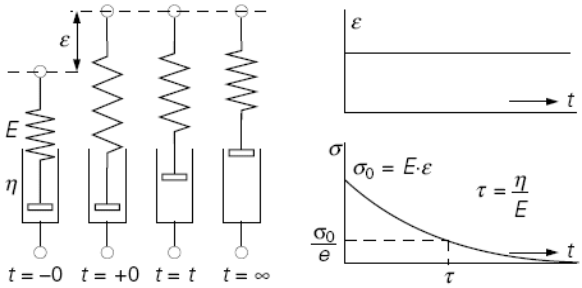
$$\frac{d\varepsilon_1}{\varepsilon_1} = -\frac{E}{\eta}dt$$

$$\ln\varepsilon_1 = -\frac{E}{\eta}t + c$$

$$\varepsilon_1 = \exp(-\frac{E}{\eta}t)\cdot c'$$

$$\sigma = E\varepsilon_1 = E\cdot\exp(-\frac{E}{\eta}t)\cdot c'$$

At t=0 $\sigma=E\varepsilon$, so $\sigma(t)=E\varepsilon \exp((-E/\eta)t) = E\varepsilon \exp(-t/\tau)$ in which $\tau=\eta/E$, the relaxation time.



Another model which described creep is the Kelvin-Voigt model. This model does not allow an instantaneous deformation and it does not show stress relaxation. At time t the stress in the spring is:

$$\sigma_1 = E\varepsilon(t)$$

In the dashpot:

$$\sigma_2 = \eta\frac{d\varepsilon(t)}{dt}$$

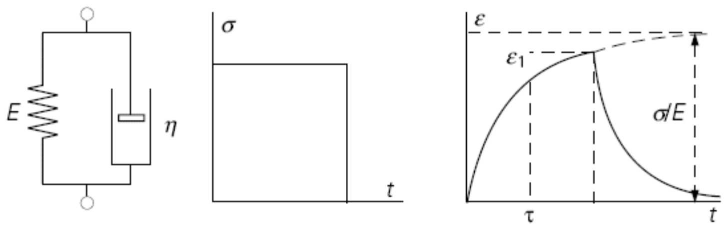
The total stress $\sigma_1 + \sigma_2 = \sigma$ is constant.

$$\sigma = E\varepsilon + \eta\frac{d\varepsilon}{dt}$$

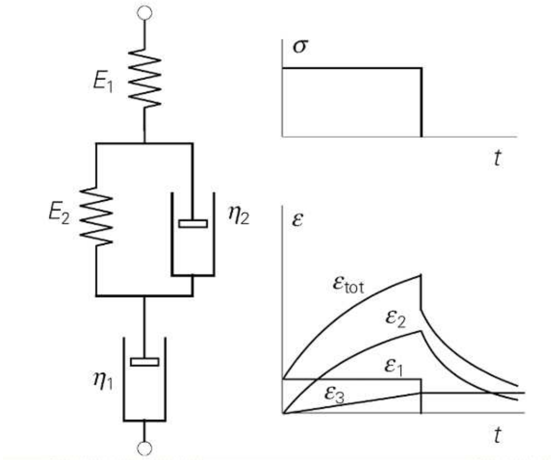
$$\eta\frac{d\varepsilon}{dt} = -E\varepsilon + \sigma$$

The solution of this differential equation is:

$$\varepsilon(t) = \frac{\sigma}{E}[1-\exp(\frac{-t}{\tau})] \text{ With } \tau = \frac{\eta}{E}$$



Both models do not fully describe the viscous-elastic behaviour. The first model fails to describe creep and the second model fails to describe the instantaneous deformation.
Based on these theories Burgers came up with another model to describe viscous elastic behaviour.



With this model also the behaviour of asphalt is described. When a deformation is imposed first the springs represents the elastic behaviour of the material, after some time the damper will take over representing the viscous behaviour of the material. The first spring represents the pure linear elastic behaviour. The second spring with damper represents the linear elastic behaviour with the possibility of storing the energy and giving the energy back. The last damper represents the behaviour in the long run with energy dissipation (creep). This can be represented in formulas:

$$\varepsilon_1 = \frac{\sigma}{E_1}$$

$$\varepsilon_2 = \frac{\sigma}{E_2}[1 - \exp(\frac{E_2 t}{\eta_2})]$$

$$\varepsilon_3 = \sigma(\frac{t}{\eta_1})$$

With these formulas the total deformation is:

$$\varepsilon = \sigma\left[[\frac{1}{E_1} + \frac{1}{E_2}(1 - \exp(\frac{E_2 t}{\eta_2})) + (\frac{t}{\eta_1})] \right]$$

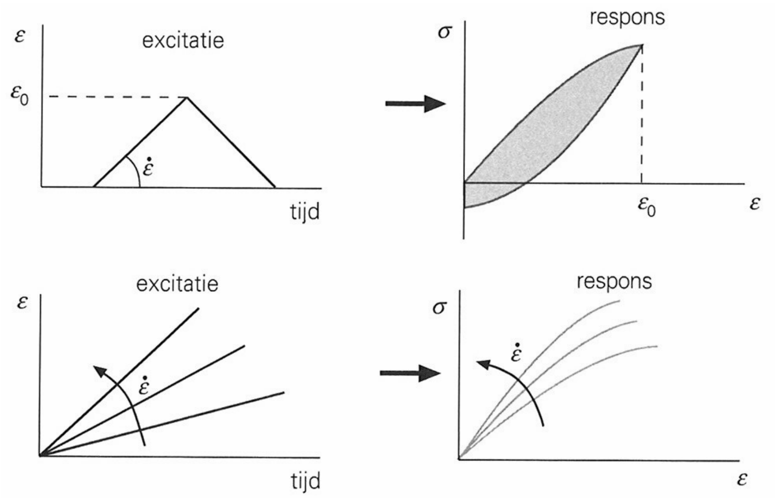
6.4 Dynamic mechanical behaviour

Asphalt is a material that shows viscous-elastic behaviour. This means the relation between deformation and stress can be described with two different properties. Again the material is deformed with a sine deformation:

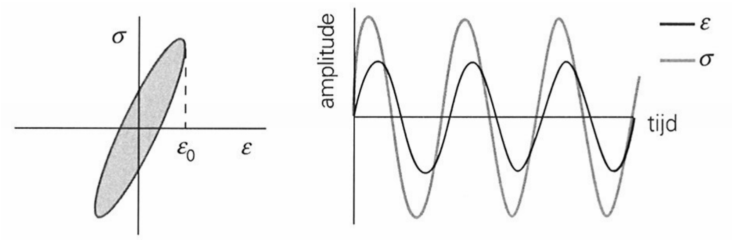
$$\varepsilon(t) = \varepsilon_0 \sin(\omega t + \delta(\omega)) \ .$$

This gives the following response:

$$\sigma(t) = E(\omega)\varepsilon_0 \sin(\omega t + \delta(\omega)) \ .$$



This means the value of the stress has a sine shape with the same frequency as the deformation but there is a phase difference δ. The value of this δ is always between 0 (elastic material) and π/2 (liquid). Both δ and E are dependent on the frequency, and for asphalt also on temperature. A part of the energy is dissipated.



To better understand the behaviour of this material the stress response is rewritten (with *sin(a + B) = cos(a) sin(B) + sin(a) cos(B)*):

$$\sigma(t) = E\varepsilon_0 \sin(\omega t + \delta) = \varepsilon_0[E\cos(\delta)\sin(\omega t) + E\sin(\delta)\cos(\omega t)]$$

The equation shows the existence of two components of the stress. The first one is in phase and the seconds one is out of phase with π/2. With:

$$E' = E\cos(\delta) \ ; \ E'' = E\sin(\delta)$$

Follows:

$$\sigma(t) = \varepsilon_0[E'\sin(\omega t) + E''\cos(\omega t)] \text{ With } \tan\delta = E''/E'$$

To determine the property of the used parameter, the energy dissipation in one cycle is obtained:

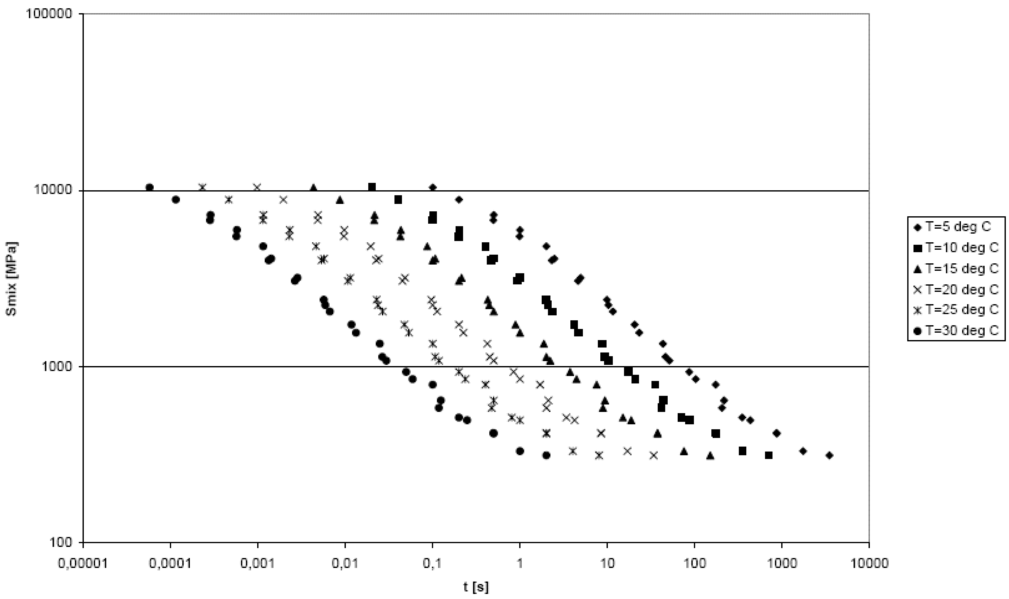
$$W = \int \sigma \, d\varepsilon = \int_0^{2\pi/\omega} \varepsilon_0[E'\sin(\omega t) + E''\cos(\omega t)] \, d\varepsilon_0 \sin(\omega t)$$

$$W = \pi\varepsilon_0^2 E''$$

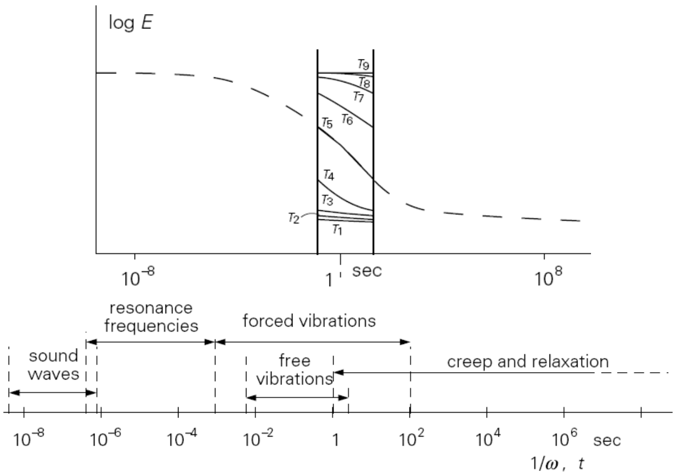
This means the dissipated energy is linearly dependent on E". This parameter represents the viscous behaviour of the material and is called loss modulus. The parameter E' represents the elastic energy that is stored in the material, so it represents the linear elastic behaviour of the material and is called the storage modulus.

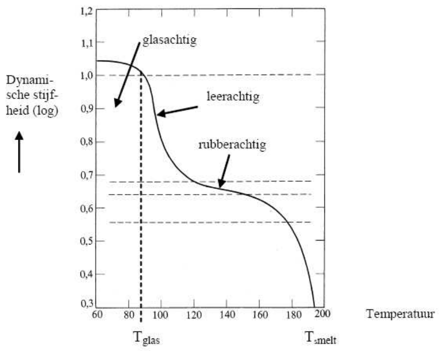
6.5 The influence of temperature

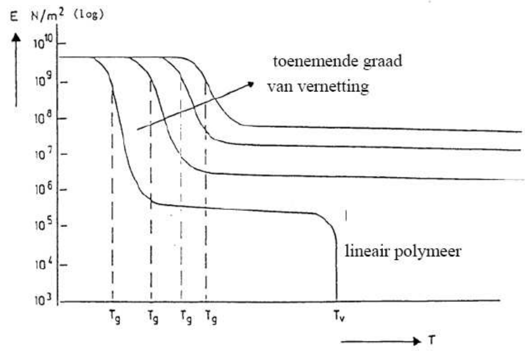
The relation between the modulus of elasticity and the temperature can be expressed in a master curve. To determine the master curve, a dynamic loading can be applied at different temperatures leading to different deformations. Also the frequency of the dynamic loading can be changed. From this, the modulus of elasticity at different temperatures can be determined. The modulus of elasticity is expressed as: $S_{mix} = \frac{\sigma}{\varepsilon}$.



Every type of asphalt mix has its own properties. In general when properties of a certain frequency range is obtained these properties can be extrapolated to the whole range of frequencies; this because the general properties of a viscous-elastic material is known.

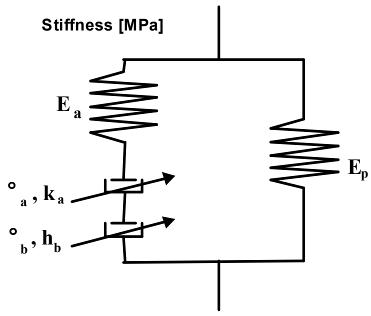






6.6 Practical approach

In practice distinction is made between an elastic and a viscous approach. When the duration of the loading is short and the temperature is low (e.g. the behaviour of an asphaltic revetment under wave impacts) the calculation method is concentrated to an elastic approach. With long duration of loading and high temperatures or bitumen content the approach is to treat asphalt as a viscose material.



In a wide range of temperatures and frequencies a viscose-elastic approach is needed. In practice master-curves are estimated with the use of the Huet-Sayegh model, a viscous-elastic model similar to the Burgers model e.g. to normalize the result of a falling weight deflectometer to one temperature.

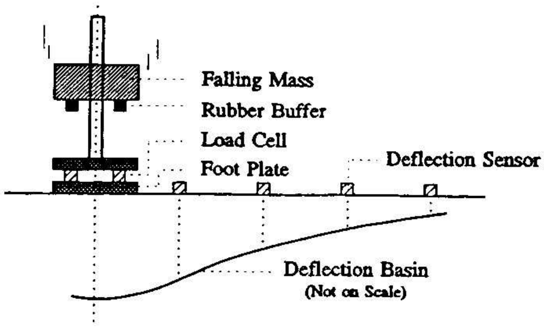
6.7 Test methods

6.7.1 Falling weight deflectometer

With a falling weight deflectometer, data about the relation between load and deformation is obtained. This with the combined data of the layer thickness, from radar, the modulus of elasticity of both the top layer and sub grade is determined. The advantages of the falling weight deflector test are:

- it is a fast method
- the load is a good simulation of a wave impact

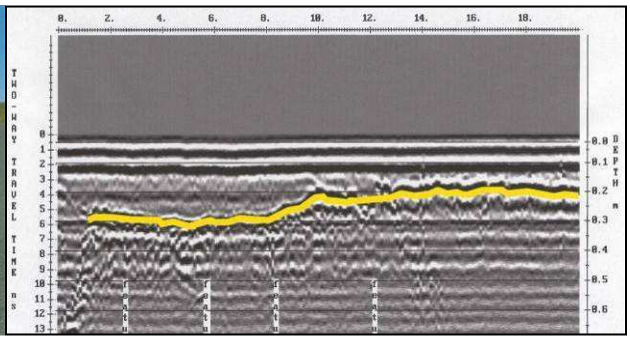
This test is used to select the weak spots of the revetment and to gather data for the safety assessment.



To analyze a specific asphalt layer tests are done on a core which is drilled out of the revetment. With this test more parameters which describe the behaviour of the layer are determined. In this section some of the test and the parameters obtained are described.

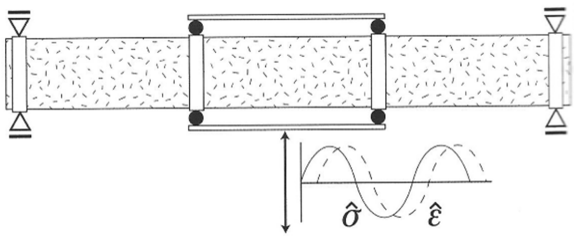
6.7.2 Ground penetrating radar test

With ground penetrating radar more information about the sub layers is obtained. The disturbance of the radar signal indicates a different material. In this way the thickness of the revetment is determined and calibrated with drilling cores out of the revetment.

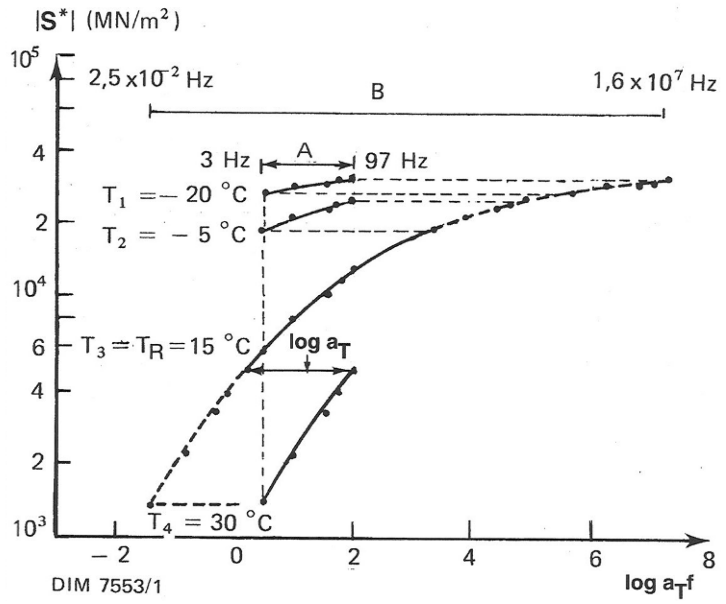


6.7.3 Frequency sweep test

With the use of the frequency sweep test the modulus of elasticity and the phase difference as a function of temperature and frequency can be determined. In the Netherlands the 4-point bending test is used.

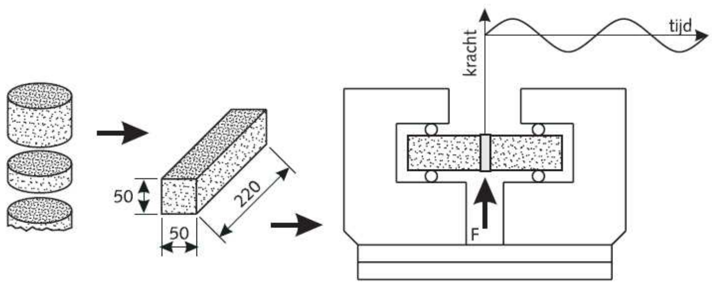


An example of the result of this test is given in the picture below.

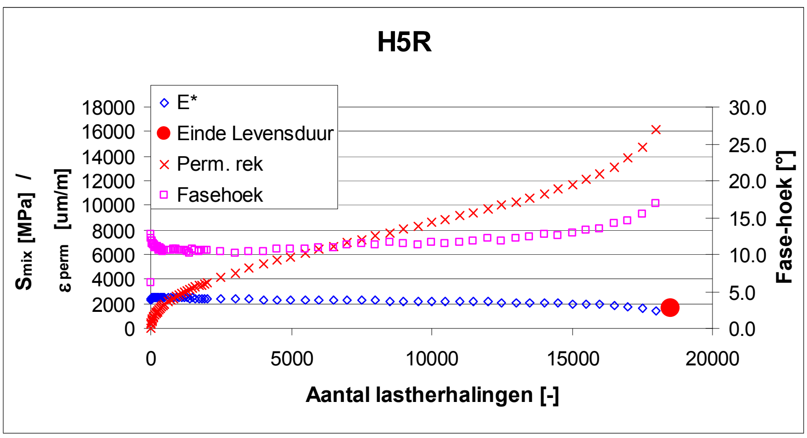


6.7.4 Fatigue test

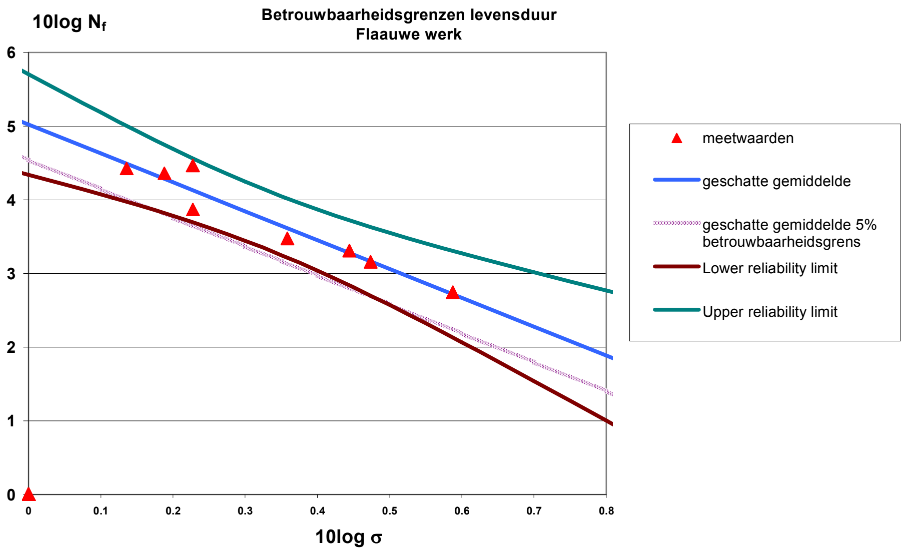
In this test the fatigue of an asphalt layer is simulated. A beam of asphalt with dimensions of 50x50x220 mm (see picture) is supported on two sides. In the middle a sinusoidal force with a certain frequency causes stresses within the material.



By measuring the force and the displacement of the beam the stress in the beam can be calculated. The sinusoidal forcing continues until the beam breaks. By comparing the signal of the forcing with the signal of the displacement the phase difference δ is determined. Also the permanent deformation is measured. An important parameter is the number of times the load is enforced on the beam; this parameter can be compared with the number of times a wave impact can occur on an asphalt revetment.



The fatigue properties are determined by testing several beams under difficult stress conditions. With the results of the tests a graph can be obtained.



The graph shows the stress in relation with the number of load repetitions to failure. For all the weak spots these tests are done and put in the same graph. In this way a linear regression line of the fatigue property is obtained.

$$\log N_f = a \cdot \log(\sigma) + \log(k)$$

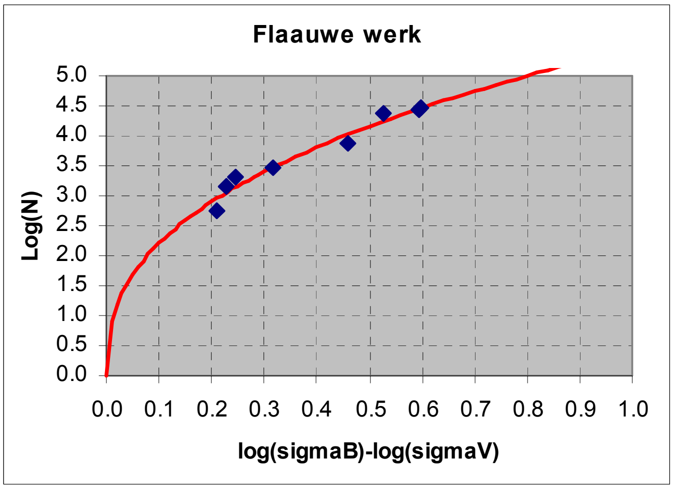
The derivative (a) of this linear regression line is a measure of relation between stress and number of loads to failure. In this way the parameter (a) is a measure for sensitivity to fatigue. The parameters (a) and (k) are used in the program GOLFKLAP for the safety assessment.

Nowadays new developments are under construction. The way the results will be presented will be changed to get more realistic presentation. Also the yielding stress has to be taken into account. The new formulae:

$$\log(N) = C_{\log N} + \beta(\log(\sigma_b)\text{-}\log(\sigma_v) + C_{\log\sigma}{}^{\alpha}$$

Where:

N	number of times loading is applied
α, β	material parameters
C_{logN}, $C_{log\sigma}$	parameters used to get the characteristic line of fatigue
σ_b	yielding strength asphalt
σ_o	the stress in the revetment due to wave loading/ applied stress in fatigue test


Flaauwe werk
Log(N)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0
log(sigmaB)-log(sigmaV)

6.8 References

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Part 3 Wave impact

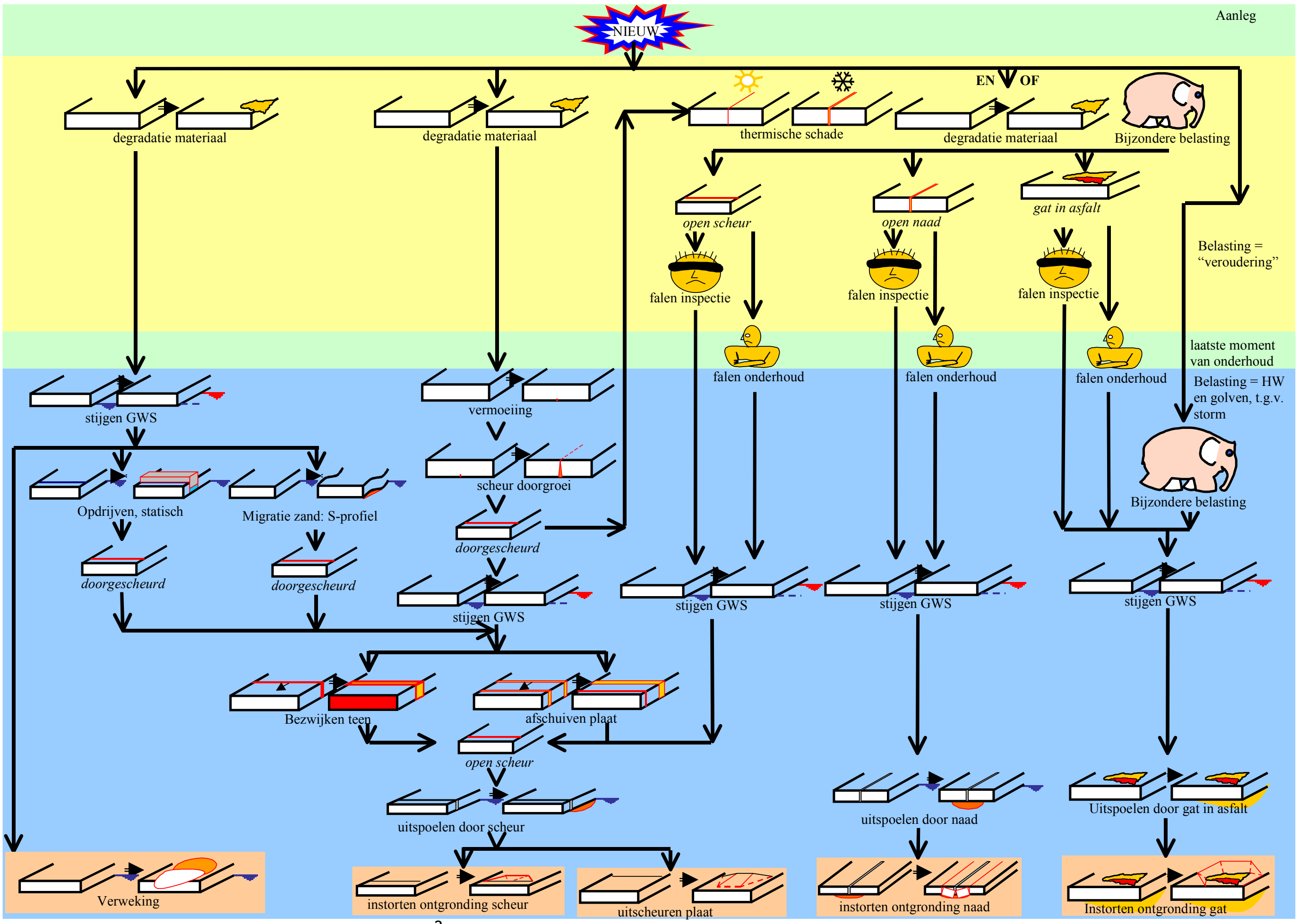
In this part the loads on the asphalt revetment is treated. Chapters two, three and five already mentioned something about loads in the context of failure mechanisms and ageing of the material. Now in more detail the load of a wave impact will be described.

7 Fault tree

The failure mechanisms for which the asphalt revetments are checked in the assessment for safety are:

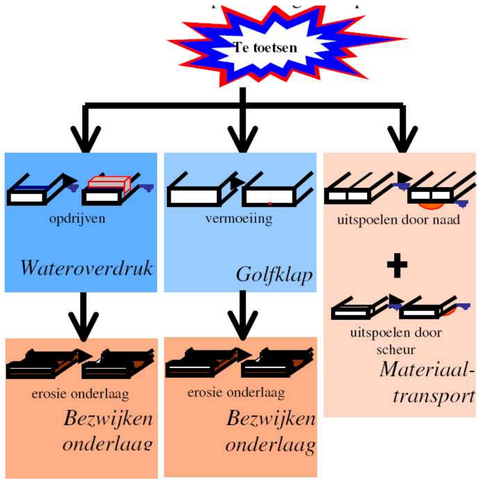
- collapse of the top-layer by wave impact
- uplift of the top-layer by water overpressure
- erosion of the dike body underneath the revetment
- collapse of the sub-layer after the collapse of the top-layer

These failure mechanisms and underlying mechanisms can be placed in a fault tree.
Looking to this fault tree it becomes clear there are many processes involved leading to failure of the asphalt revetment.



The revetment fails when 1 m^2 of the sub layer in the wave attack zone is exposed to erosion; at the bottom of the fault tree this is shown. The consequences of special loads are not taken into account.

This report restricts itself by only looking to that part of the fault tree where wave impact is important. This means only the part of the fault tree where fatigue is mentioned, this part starts with degradation of the material. As explained in chapter five, weathering and other processes lead to degradation of the material. In time asphalt becomes more sensitive for fatigue so in the fault tree this part is tested with GOLFKLAP as shown in the picture below.



After failure for fatigue it is expected that cracks will appear in the layer and the layer will be ripped through because of that. When the layer is ripped through the revetment can slide down and/or sand will be washed out. This leads to exposure of the sub layer to erosion and thus the total collapse of the revetment.

7.1 References

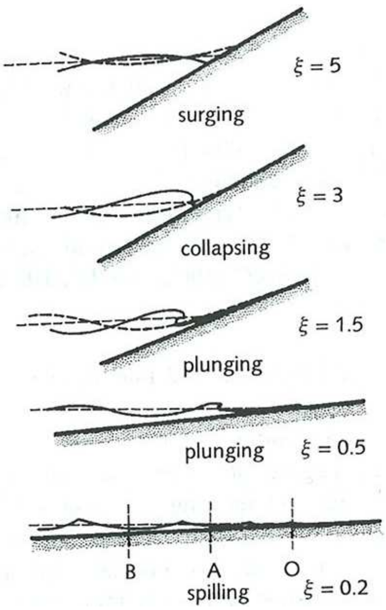
RWS (2007) Voorschrift Toetsen op Veiligheid Primaire Waterkeringen (in Dutch)

8 Development of theories on wave impact till 1990

In this chapter a summary is given of the studies made in the past about wave impacts on dike slopes. Use is made of a literature survey ordered by the Technical Advisory Committee for hydraulic structures in 1988. In the text of this survey references are made to different authors who performed study on wave impact on slopes. In this chapter the most important references are copied and added to the reference list to make clear from which source the information comes initially. For more details is referred to; "VAN VLEDDER G. PH. (1990) Literature survey to wave impacts on dike slope (Report H976), Delft hydraulics, Delft" and its references. The first section gives several definitions which are used in this chapter. The second section gives an overview of the different methods used in the past to model the wave impact. The sections thereafter deal with the experiments made until 1988, and at the end of this chapter the results of the studies are summarized.

8.1 Definitions and context

Now some parameters will be defined which will be used extensively throughout this report. The load on a dike slope is for a large part determined by the way the waves break on the slope, the breaker type. 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)



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

$$L_0 = \frac{gT^2}{2\pi}$$

where:

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

Plunging and collapsing breakers give the highest impact pressures, only the effects of these waves are treated in this report.

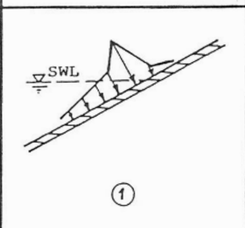
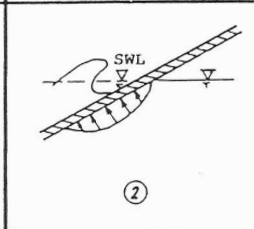
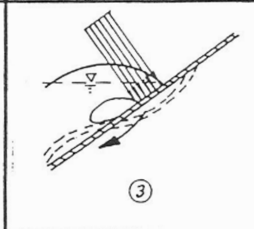
The wave impact on a slope is not only dependent on the breaker type but also on the breaking height. Simgamsetti and Wind summarized the many different definitions of the breaking height in the year 1980. The definition of the breaking point used in the Literature survey is the point where the particle velocity at the crest becomes larger than the wave celerity. The breaking wave height is indicated by H_b and is defined as the difference between the maximum distance in the vertical direction between the crest of the breaking wave and the following through. The breaker depth h_b is defined as the mean depth at the breaking point.

In 1988 the associated forces of breaking waves are divided by Witte into two groups:

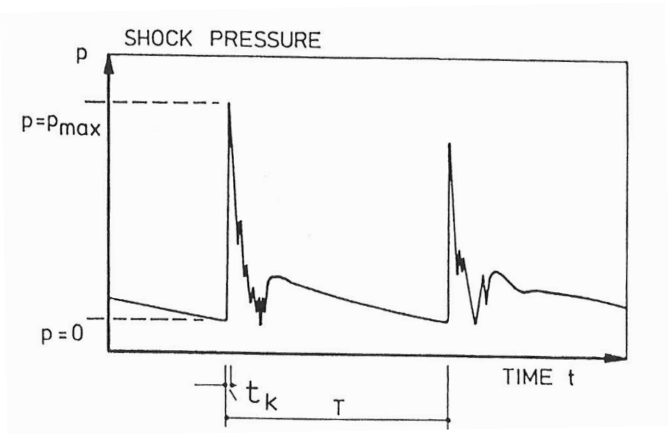
- quasi static forces
- dynamic forces

The dynamic forces are divided in the year 1988 by Oumeraci and Partenscky into three classes:

- quasi-static impact pressure
- uplift pressures
- cyclic shocks and vibrations

Designation	quasi-static impact pressures	uplift pressures	cyclic shocks and vibrations
wave loading cases			
Effect on slope revetments	- Deflections - local failure of the subsoil - collapse of the revetment	- Deflection - lifting - overturning - motion of subsoil particles	- liquefaction or cyclic mobility - sliding of subsoil - collapse

In the Literature survey of *van Vledder* only the last of the third class of forces is addressed. At the moment of impact large pressures occur due to the inertia of the water mass. Such pressures are usually referred to as impact pressures, but also as shock pressures, since the shock propagates through the water mass. A typical example of the time history of the impact pressure is given in the next figure. Based on this figure a number of characteristics of wave impact can be inferred.



- the wave impact generates relatively high pressures, compared to the quasi-stationary pressures
- the short time scale (typically of the order of 10 - 60 ms) of the shock pressures compared to the wave period
- the short durations for the pressure to reach its maximum
- the regular character of the quasi-static pressures

- the ragged character of the impact pressure fluctuations

The following parameters can be defined:

p_{max} the maximum impact pressure

t_k the compression time, the time needed for the pressure to reach its maximum

t_d duration of the impact, equal to the sum of t_k and t_e

t_e the expansion time, the time needed for the pressure to reduce to a hydrostatic value

It is not always clear when a sharp rise in pressure should be referred to as a shock or impact pressure. Grüne (1988) adopts two main boundary conditions to be fulfilled to find the shock pressures:

- the rising time (t_k) up to the maximum pressure must be shorter than the wave period
- the maximum peak pressure (p_{max}) should exceed a certain multiple value of the wave height

By using these boundary conditions some real shock pressures with smaller peak values and a short rising time might be neglected.

The wave impact depends on a number of parameters. These parameters can be divided into two groups; properties of the materials involved in a wave impact and the geometrical properties. A very important property is the air content of the water mass that impacts on the slope. Even when the air content is small the impact pressure reduces considerably. In the year 1966 Führböter showed that the amount of air in the water could be considered as a stochastic variable, giving rise to a stochastic character of the impact pressures, even if the incident waves are regular. Empirical studies in the year 1960 by Prins and Venis also showed that enclosed air reduces impact pressures on structures. Venis also found the stochastic nature of impact pressures and found that impact pressures are distributed to some logarithmic distribution. In breaking waves air concentrations of 20% and 40% may be found as Führböter showed in the year 1970. Also an important parameter is the backwash layer on the slope. In 1966 Führböter also showed that even a thin layer of water on a dike slope considerably reduces the impact pressures on the slope. This is due to the fact that this layer of water has a high compressibility due to enclosed air and due to the spreading effect in the water layer.

Also the geometrical properties are important parameters to determine the wave impact on a slope. The geometry of the dike and the foreshore influence the height and the velocity of the waves. The incident wave field is often described by means of the wave spectrum, which gives the distribution of wave energy as a function of frequency and direction. When waves propagate from deep to shallow water the wave characteristics change due to shoaling, refraction, diffraction and dissipation effects.

To cover the parameters used in this chapter most of them are defined here:

$\chi = \frac{H}{gT^2}$ wave steepness parameter

T_p peak period

L_p peak wave length

H_s significant wave height

$\xi_0 = \frac{\tan(\alpha)}{\sqrt{H_0 / L_0}}$ surf similarity parameter based on deep water wave height (H_0)

$\xi_b = \frac{\tan(\alpha)}{\sqrt{H_b / L_0}}$ surf similarity parameter based on breaking water wave height (H_b)

p_{max} highest theoretical impact pressure measured by one pressure transducer

$P_{max.max}$ highest measured impact pressure during wave impact for all transducers

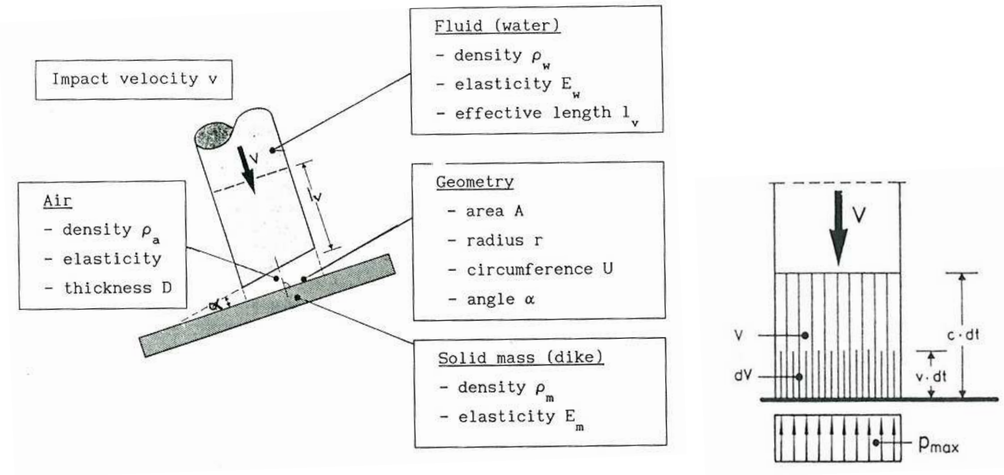
ρ_w density of water

ρ_a density of air

g gravitational acceleration

8.2 Methods of computation

There have been many theoretical investigations to describe the effect of breaking waves on dike slopes. Emphasis is laid on the determination of the maximum pressure during impact and on the time needed to reach this maximum impact pressure. In most investigations a number of simplifications are made. The breaking wave is idealized as a water mass, falling with a certain velocity on the dike slope under a certain angle. To describe the wave impact it is often assumed that a certain amount of air is trapped between the falling water mass and the dike slope, or that the falling mass consists of a homogeneous air-water mixture.



The studies can be divided in two groups. The first group assumes incompressible materials, whereas the second group takes compressibility into account. Within the second group a further division can be made in theories which take the effect of air entrainment into account or not.

8.2.1 Incompressibility of materials

Von Karman in 1929, and Wagner in 1932, determined the maximum pressure p_{max} occurring during impact of an incompressible cone of water on a still water surface, whereas Cumberbatch in 1960 investigated the impact of a water cone on a stiff mass. In these investigations only a value for the maximum pressure p_{max} was determined. The maximum pressure is given by:

$$p_{max} = k\,\rho_w \frac{v^2}{2}$$

In which v is the impact velocity of the water cone, and k the so called dynamic factor. Von Karman used $k = \pi^2 \cot(\alpha)$ and Lewison in 1970 based on Wagner used $k = \pi^2 \cot^2(\frac{\alpha}{4}) + 1$. The result of these studies show that the maximum pressures p_{max} depends on the angle of incidence between the cone and the water surface or stiff mass.

8.2.2 Compressibility of materials

Theories for the computation of the maximum impact pressures are divided into two groups. The first group only takes compressibility of the water or dike slope into account. The second group also takes the presence of air in the form of bubbles in the falling water into account. Von Karman in his investigation used the relation between the velocity of shock waves and the modulus of elasticity. By impact the water mass is compressed and a shock wave propagates through the water mass. The time of compression is shorter then the time the shock wave needs to reach the boundaries of the water mass. With this theory Von Karman came up with a formula for p_{max}:

$$p_{max} = k\,\rho_w \frac{v^2}{2} \text{ with } k = 2\frac{c_w}{v}$$

where c_w is the velocity of the shock wave through the water and v the impact velocity. This theory is extended by various authors with the resulting formula:

$$p_{max} = \rho_w\, v\, c_w \frac{1}{1 + (c_w / c_m)}$$

In this equation is shown that the inclusion of the compressibility of the solid mass reduces the maximum impact pressure.

Later on Lundgren in 1969 and Sellars in 1971 showed that when air is entrained in the water the shock wave in the water air mixture reduces considerably. In 1966 Führböter extended the theory of Von Karman including the effect of air in the water and came up with:

$$p_{max} = \rho_w v c_w \frac{1}{1 + \varepsilon'((E_w / E_a) - 1)}$$

where ε' is the fraction of air volume in the water and E_w/E_a=15500

8.2.3 Vertical walls

In 1939 Bagnold developed a theory to compute the impact pressure on a vertical wall. The basic assumption in his work is that impact forces by breaking waves can only occur by the compression of a certain amount of air enclosed between the water mass of the breaking wave and the vertical wall. In his theory the water and the wall are taken incompressible. The theory of Bagnold is later on extended by Minikin in 1950. In his formula the thickness of the amount of air can be accounted for.

In 1959 Djounkovski and Bojitch published a theory for the computation of the maximum impact pressure that is based on the assumption that the trajectory of the water particles in the breaking wave can be described by a parabola. Also no compressibility is taken into account. The equation written by Djounkoski and Bojitch was rewritten by Krylov in 1966, using dimension analysis. In this equation also a dimensionless parameter α is used for the slope on which the water mass impacts. Another method to compute the maximum impact pressure was given by Nagai in 1960 and 1961, who used the time interval of the compression phase in a parameter. He showed that the compression time is inversely proportional to the maximum pressure.

8.2.4 The theory of Führböter

Führböter did a lot of test on wave impact; his research goes back for many years. In this section is explained how the theory extended in the years. As already mentioned Führböter extended the theory of Wagner by taking air entrainment into account. In his theory only the compressibility of water and air is taken into account and he stressed the need to take the effect of an expanding water mass into account. In 1966 Führböter also used statistical descriptions of impact pressures. Laboratory investigations show that under similar boundary conditions the observed maximum pressures have strong fluctuations between different impacts. It can be shown that these fluctuations are due to random variations in the air content in the tongue of the water mass and to variations in the volume of air enclosed under of the falling water mass. It follows that the natural logarithm of the maximum impact pressure is a stochastic variable with a log-normal distribution. The impact pressures p_{max} can be quantified if the impact velocity and the radius of the falling mass are know. The velocity can be related to the breaking depth and the breaking wave height. In the end the formula has a stochastic part and a deterministic part. In view of the stochastic character of the wave impact and in view of practical design methods of sea dikes, Führböter and Sparboom (1988) prefer to use the equation:

$$p_{max} = \text{const.} \ \rho_w \ g \, H$$

In which the constant depends on the probability of exceedance of a certain maximum pressure and the dike slope.

8.3 Experiments

This section summarizes the result of field and laboratory investigations with respect to the forces and pressures exerted by breaking waves on dike slopes till 1990. In the previous section already some investigations are mentioned which are performed before 1969. In this section only investigation performed after 1966 are mentioned. Wave impact pressures are usually measured with one or more pressure transducers embedded in the slope of the dike. Most wave impacts occur in the region just below still mean water so the transducers are normally located in that region. To detect the quick changes in pressure the transducers must have a high sampling rate and the transducers also have to withstand very high pressures. Also the size of the transducers is important. When the transducer is too small air bubbles may cover it and when too large high impact pressures are underestimated due to smoothing effects. The area where impact occurs is small so the transducers should be placed close together.

8.3.1 Laboratory experiments

To simulate the tongue of a breaking wave Führböter in 1966 and 1969 carried out laboratory experiment with a water-jet. Führböter found impact velocities varying between 4 m/s and 8.3 m/s. These correspond with plunging breakers with heights of 1m - 2m.

8.3.2 Scale models

In 1985 Zhong performed experiments in the 2 m wave flume of the Leichtweiss Institute of the Technical University of Braunschweig. Tests were performed on a 1:10 scale. A slope of 1:4 was used. The incident waves were regular, the length of the channel was about 100 m, the width 2 m and the depth 1.2 m. Waves could be generated with periods between 1.0 s and 2.8 s and heights up to about 25 cm. Zhong found that the nature of impact pressures is indeed stochastic. His results indicate that p_{max} is log-normally distributed.

In 1986 Delft Hydraulics performed investigations to the toe protection of sand dunes. Three transducers were placed with a spacing of one meter. The experiments were performed with series of random waves. In each series a fixed significant wave height (H_s), peak period (T_p) and water depth (h) was used. A mean JONSWAP spectrum was used to distribute the individual wave components. The significant wave height varied from 1.0 to 1.83 m, the peak period was almost constant and equal to 4.9 s. The series of waves consisted of 219 waves, of which 55 were identified as waves generating an impact. Visual inspection of the cumulative distribution of these 55 impacts suggests that these impacts are log-normally distributed. However, visual inspection of the statistical distribution of all 219 maximum impact pressures showed that they were Rayleigh distributed.

Also Führböter performed scale model tests. In 1986 a test was performed in a wave flume in a 1:10 scale model. The summarized results:

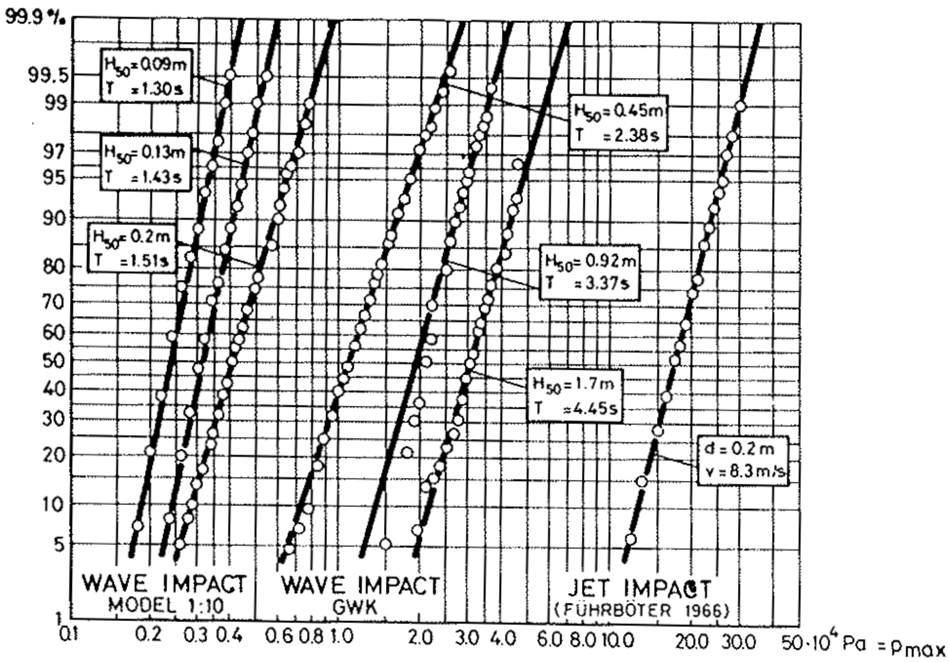
H/gT^2=0.006			H/gT^2=0.009		
N	H (m)	T (s)	N	H (m)	T (s)
200	0.085	1.13	200	0.105	1.06
200	0.125	1.43	200	0.160	1.30
200	0.200	1.84	200	0.175	1.41
			200	0.200	1.51

In this test the maximum impact pressures where also log-normally distributed. In addition Führböter found the following characteristics for a dike slope of 1:4:

$$P_{max.max}(P_u) = k(P_u)\,\rho_w\, g\, H$$

Where $P_{max.max}$= max(p_{max}), highest maximum impact pressure, P_u the probability of non-exceedance and k the dynamic factor of impact.
With:

$$\begin{aligned} k(P_u = 50\%) &= 2.2 \pm 0.4 \\ k(P_u = 90\%) &= 3.0 \pm 0.5 \\ k(P_u = 99\%) &= 3.9 \pm 0.6 \\ k(P_u = 99.9\%) &= 4.8 \pm 0.7 \end{aligned}$$



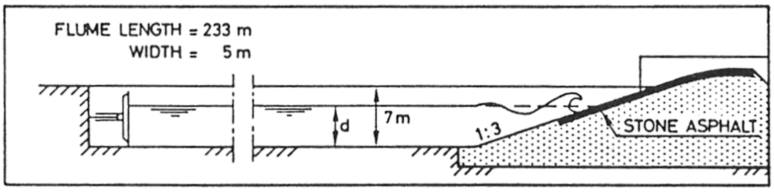
Comparison of results from water-jets, 1:10 scale test and full-scale tests

8.3.3 Full scale laboratory experiments

The laboratory experiments described in this section are performed, either in the large wave channel in Hannover or in the Delta Flume in Delft. The wave channel in Hannover has a

length of 324 m, a width of 5 m and a depth of 7 m. In this flume regular and random waves up to 2 m can be generated.
The Delta Flume has a length of 233 m, a depth of 7 m and a width of 5 m.

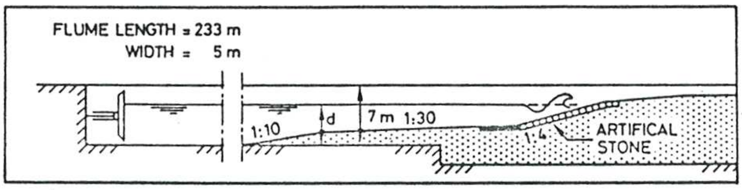
In 1983 Stive analyzed two series of full-scale measurements of regular waves breaking on a dike slope in the Delta Flume. The first series concerns a 1:3 slope of open stone asphalt revetment and the second concerns a 1:4 slope of a regularly placed artificial stone revetment.



DELTA FLUME, SLOPE 1:3

$\xi = \frac{\tan\alpha}{\sqrt{H\cdot(1.56\cdot T^2)^{-1}}}$	WATER DEPTH d (m)	WAVEHEIGHT H (m)	WAVEPERIOD T (s)	WAVE BOUNDARY PARAMETER $\chi = H\cdot(g\cdot T^2)^{-1}$
1.33	4.65	0.90	3.0	0.010
1.40	5.00	0.80	3.0	0.009
1.49	4.86	1.25	4.1	0.008
1.56	4.66	0.65	3.0	0.007
1.75	4.75	0.91	4.1	0.008
1.78	5.00	0.87	4.0	0.006
1.82	4.68	0.48	3.0	0.005
1.84	5.00	1.28	5.0	0.005
1.95	5.00	0.41	3.0	0.005
2.01	5.00	1.54	6.0	0.004
2.15	5.00	0.60	4.0	0.004
2.30	4.68	0.53	4.0	0.003
2.49	5.00	0.70	5.0	0.003

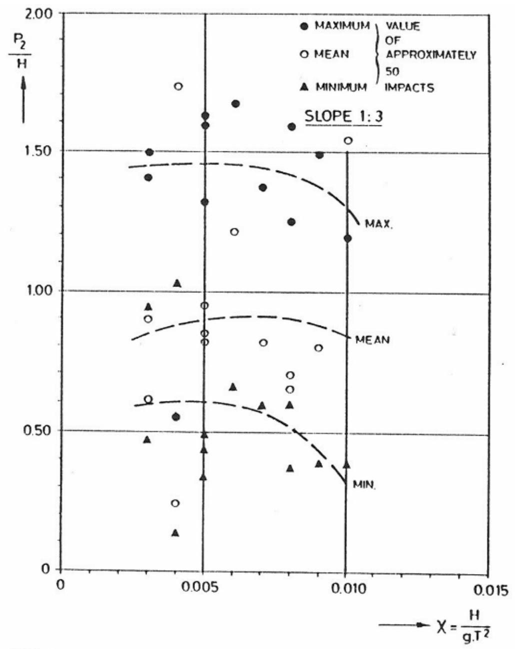
DELTA FLUME, SLOPE 1:3



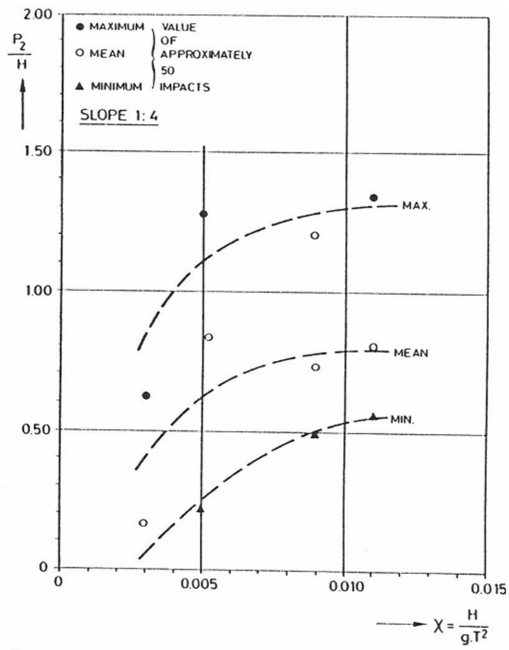
DELTA FLUME, SLOPE 1:4

$\xi = \frac{\tan\alpha}{\sqrt{H \cdot (1.56 \cdot T^2)^{-1}}}$	WATER DEPTH d (m)	WAVEHEIGHT H (m)	WAVEPERIOD T (s)	WAVE BOUNDARY PARAMETER $\chi = H \cdot (g \cdot T^2)^{-1}$
0.94	4.25	0.50	2.1	0.011
0.99	4.25	0.45	2.1	0.010
1.04	4.25	0.40	2.1	0.009
1.36	4.50	0.95	4.2	0.005
1.87	4.50	0.50	4.2	0.003

DELTA FLUME, SLOPE 1:4



THE NORMALIZED "IMPACT" PRESSURE AT SLOPE 1:3, AS FUNCTION OF χ



THE NORMALIZED "IMPACT" PRESSURE AT SLOPE 1:4, AS FUNCTION OF χ

Stive shows that with increasing wave steepness the maximum impact pressure decreases. Based on these experiments a relation was found between the maximum of the observed maximum impact pressures and the incident wave height. The result by Stive:

$$P_{max.max} = 2.7\rho_w gH \quad \text{(slope 1:3)}$$

$$P_{max.max} = 2.3\rho_w gH \quad \text{(slope 1:4)}$$

In 1984 Delft Hydraulics performed full scale experiments of breaking wave on a 1:6 slope in the Delta Flume. Based on the previous experiment of Stive the set-up of this experiment was carefully designed, this to obtain high quality measurements of impact pressures. In this experiment both regular and random waves were generated. 35 pressure transducers are used with a closer spacing near the plunging point. In addition a set of 10 pressure transducers were placed in horizontal direction. The incident wave parameters are given in the next table where

$$\xi_{sp} = \frac{\tan(\alpha)}{\sqrt{H_s / L_p}} \;,\; \chi = \frac{H}{gT^2} :$$

Regular waves, slope 1:6					
No.	H (m)	T (s)	h (m)	1000 χ	ξ_o
1	0.53	4.50	3.86	2.7	1.29
2	1.01	4.50	4.14	5.1	0.93
3	1.49	4.50	4.36	7.5	0.77
4	1.88	4.50	4.38	9.5	0.68
5	0.54	5.80	4.02	1.6	1.64
6	1.12	5.80	4.18	3.4	1.14
7	1.46	5.80	4.40	4.4	1.00
8	2.05	5.80	4.45	6.2	0.84
9	0.56	7.09	4.19	1.1	1.97
10	1.06	7.09	4.36	2.2	1.43
11	1.59	7.09	4.45	3.2	1.17
12	2.11	7.09	4.45	4.3	1.02

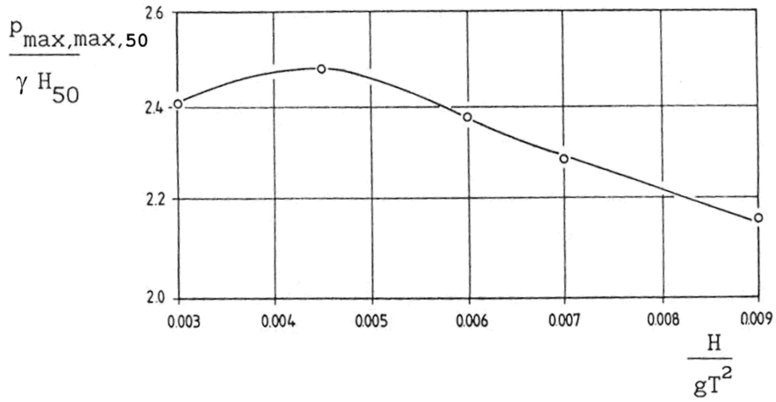
Random waves (JONSWAP spectrum), slope 1:6					
No.	H_s (m)	T_p (s)	h (m)	1000 χ	ξ_{sp}
20	1.36	8.53	4.74	1.9	1.52
21	1.68	6.40	4.62	4.2	1.03
22	1.51	4.80	4.61	6.7	0.81
23	0.62	6.40	4.35	1.5	1.69
24	0.82	7.68	4.32	1.4	1.77
25	0.95	3.07	4.19	10.3	0.66

Unfortunately no analysis of the results are made.

In 1985 full scale experiments described by Zhong were performed in the wave channel in Hannover. Regular waves are used in this experiment. The result of Zhong indicates that the distribution of p_{max} is log normally distributed. For each set of 200 waves Zhong computes the value of $P_{max.max,50}$ that is exceeded by 50% of the impacts. Although regular waves were used, the point of breaking varied. In his results Zhong used the notation H_{50} to indicate the mean incident wave height. Mean and maximum values for $P_{max.max,50}$ are:

$$\frac{P_{max.max,50}}{\rho_w g H_{50}} = 2.3 \qquad \text{(mean value)}$$

$$\frac{P_{max.max,50}}{\rho_w g H_{50}} = 3.2 \qquad \text{(maximum value)}$$



With $\gamma=\rho_w g$ the specific weight of water. These results show that with increasing wave steepness the normalized impact pressure decreases.

One year later in 1986 Führböter also performed experiments in the wave channel of Hannover. An impermeable 1:4 slope was used in the large wave channel and also experiments in a smaller wave flume are done with a scale of 1:10. The main conclusion from this study is that it confirmed his theory about the log-normal distribution of maximum impact pressures. This holds for jet-, model- and full-scale experiments, but also for experiments with different wave steepness values and dike slopes.

P_1/P_{50}	water-jet	scale 1:10 full-scale
P_{90}/P_{50}	1.5	1.4 ± 0.1
P_{99}/P_{50}	2.1	1.8 ± 0.2
$P_{99.9}/P_{50}$	2.7	2.3 ± 0.3

In the wave channel of Hannover Führböter and Sparboom (1988) analyzed wave impacts for two dike slopes (1:4 and 1:6). There results are summarized as:

$$P_{max.max} \cong 6\rho_w gH \qquad \text{(slope 1:4)}$$

$$P_{max.max} \cong 4\rho_w gH \qquad \text{(slope 1:6)}$$

These results differ from the results of Stive. This could be because different slope material is used, in the experiment of Führböter and Sparboom concrete is used while Stive used open stone asphalt. In the experiment of Stive also a berm is present what could have influence on the maximum wave impact.

In 1988 Witte presents results of experiments with regular waves breaking on a dike with slope 1:4, a study performed in the wave channel of Hannover. The incident waves have the following characteristics, wave height H=1.25 m, wave period T=4,6 s, water depth h=4.8 m, surf similarity parameter ξ_0=1.29. Witte found that the distribution of maximum impact pressures could not always be described by the standard two-parameter log-normal distribution. Witte transformed all impact pressures by means of a linear transformation into a three-parameter distribution.

8.3.4 Field experiment

Grüne (1988) presents the result of field experiments done in 1969 and 1973. This experiment was executed at the Eiderdam and the island Wangerooge. The slope of the revetment at the Eiderdam was 1:4 in the lower and 1:6 in the upper part, and consisted of asphalt stone revetment. At the island Wangerooge the asphalt stone had a nearly constant slope of 1:4. Grüne analyzed the wave data and came up with a relation between the incident significant wave height (H_s), measured just in front of the toe of the dike, and the maximum impact pressure (P_{max}). For the Eiderdam location Grüne found:

$$P_{max.max} \approx 7\rho_w g H_s$$

And for the island of Wangerooge:

$$P_{max.max} \approx 5\rho_w g H_s$$

These results differ from the result of Stive in the year 1984, according to Grüne this due to different bottom topography.

8.3.5 Summary of experiments

Year	Author	Experiment type	Where	Slope	Wave distribution	Number of waves	Result
1966, 1969	Führböter	Water-jet		30° - 90°		600	Impact velocities Log-normal distribution p_{max}
1984	Stive	Full scale	Delta Flume	1:3 1:4	regular		Relation incident wave height and $P_{max,max}$
1984	Delft Hydraulics	Full scale	Delta Flume	1:6	regular JONSWAP		
1985	Zhong	Scale model 1:10	Braunsweig	1:4*	regular	1000	Log-normal distribution p_{max}
1985	Zhong	Full scale	Hannover				Log-normal distribution p_{max} $P_{max,max}/(\rho_w gH)$ decreases with increasing wave steepness
1986	Delft Hydraulics	Scale model 1:3	Delta Flume	1:3.6	JONSWAP		Log-normal distribution p_{max} Rayleigh distribution all waves
1986	Führböter	Scale model 1:10	Braunsweig	1:4*	regular	14000	Log-normal distribution p_{max} for different wave steepness and different dike slopes
1986	Führböter	Full scale	Hannover	1:4**	regular	2372	
1988	Führböter and Sparboom	Full scale	Hannover	1:4** 1:6	regular JONSWAP	2372	$P_{max,max}/(\rho_w gH)$ decreases with increasing wave steepness Relation incident wave height and P_{max}
1988	Witte	Full scale	Hannover	1:4**	regular	175	Three parameter log-normal distribution p_{max}
1988	Grüne	Field experiment	Wangerooge Eiderdam	1:4			Relation incident wave height and $P_{max,max}$
				*,** refer to the same experiment			

8.4 Comparison of the results

8.4.1 Wave height

As already mentioned there is a difference in results in the relation between incident wave height and the maximum impact pressure found by different investigators, this mainly because the circumstances in the experiments are not the same. Some general relations however are confirmed by most of the investigators. The relation between the incident wave height and the maximum impact pressure is described with:

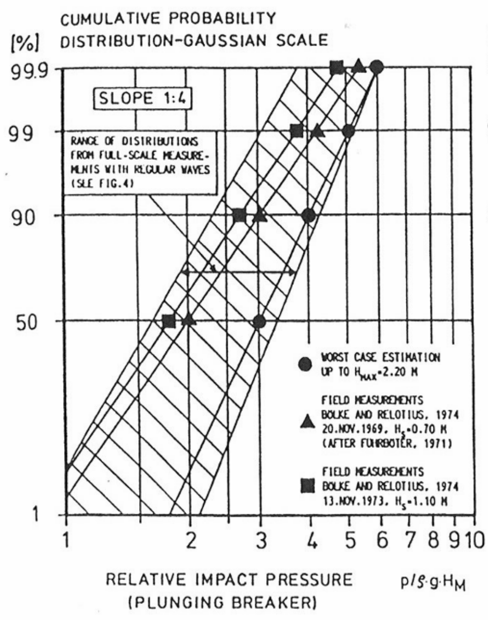
$$P_{max.max} = \text{const.}\ \rho_w\ g\ H$$

Where the constant is given in the next table:

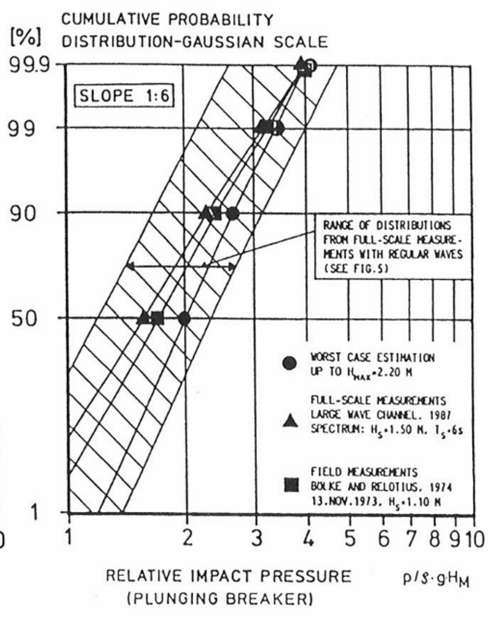
Year	Investigator(s)	Slope	constant	Wave steepness (χ)
1969	Skladnev and Popov	1:4	2	0.03
1984	Stive	1:3 1:4	2.7 2.3	0.003 - 0.01 0.003 - 0.011
1984	Delft Hydraulics	1:6		0.0027 - 0.0103
1985	Zhong			0.003 - 0.009
1986	Führböter	1:4*		0.006 0.009
1988	Führböter and Sparboom	1:4* 1:6	6 4	0.0015 - 0.038
1988	Witte	1:4*		0.006
1988	Grüne	Eiderdam 1:4 Wangerooge 1:4	7 5	
		* refer to the same experiment		

8.4.2 Log-normal distribution

Also the log-normal distribution of the maximum impact pressure is confirmed by most of the investigators. Because the maximum impact pressure is considered to be a stochastic parameter Führböter in 1986 introduced a new notation. Where the maximum pressure per transducer is denoted by p_{max} and the maximum pressure that is not exceeded in n% of the cases is denoted as $p_{max,n}$. $p_{max,99.9}$ is considered as the highest measured maximum impact pressure. When the highest impact pressure for all transducers is mend the notation $P_{max.i}$ ($P_{max,max.i}$ in this review) is used with i running over all sensors. The results of the tests done by Führböter are shown in the next picture.



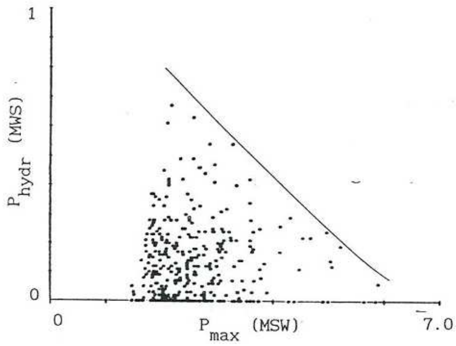
Shock pressure distribution for the worst case, slope 1:4



Shock pressure distribution for the worst case, slope 1:6

8.4.3 Backwash water

The effect of backwash water is also described by different authors. In 1962 Greslou and Montaz noted that the highest impact pressures occur on dry slopes. Führböter studied the effect of the backwash water on maximum impact pressures in 1966. In the test with the water jet the thickness of the layer was varied and the impact pressures were measured. In 1986 Führböter with the full scale experiment found that the layer thickness also can be described with a log-normal distribution. Oumeraci in the year 1984 states that even a thin layer of water of 2 - 3 cm significantly reduces impact pressures. Grüne (1988) also examined the effect of the backwash layer. In his study he related the thickness of the backwash layer to the quasi-static pressure just before wave impact. The quasi-static pressure is in his analysis related to the thickness of the backwash layer (p_{hydr}).



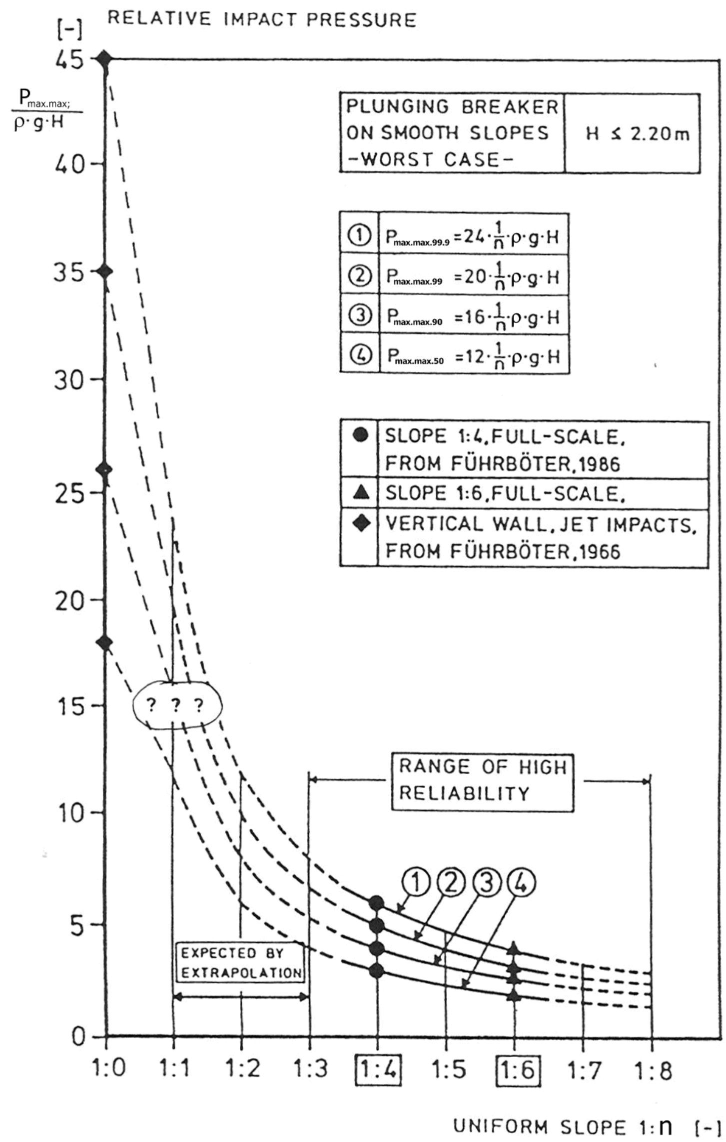
In the picture can be seen that the thicker the backwash layer the lower the maximum impact pressure.

8.4.4 Dike slope

Führböter and Sparboom (1988) investigated the relation between the impact pressure and the dike slope. They found that P_{max} is inversely proportional with the slope n:

$$P_{max.max} = c\frac{1}{n}\rho_w gH \text{ with c as a constant}$$

Führböter and Sparboom used the data acquired by Führböter in 1966 in the experiment with the water jet on a vertical wall. They summarized their findings (picture notations are changed with respect to original picture):



8.4.5 Time aspects

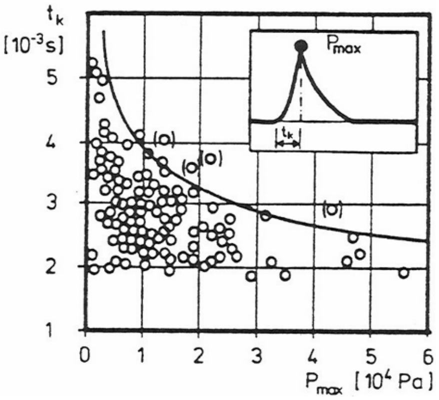
In the beginning of this chapter some parameters concerning the duration time of an impact are defined.

t_k the compression time, the time needed for the pressure to reach its maximum

t_d duration of the impact, equal to the sum of t_k and t_e

t_e the expansion time, the time needed for the pressure to reduce to a hydro-static value

In 1988 Witte and Grüne (1988) studied the relation between the rising time t_k and the maximum impact pressure. The result of Witte is shown in the picture below.

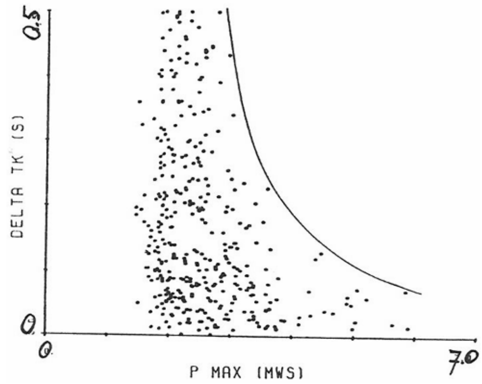


Witte suggested the dimensionless relation:

$$p'_{max} = 14 t'^{\,-0.28}_{k} \quad \text{with} \quad t'_k = \frac{c_w}{H/2} t_k$$

He also found that the rising time t_k of the impact pressure is proportional to $1/p_{max}$.

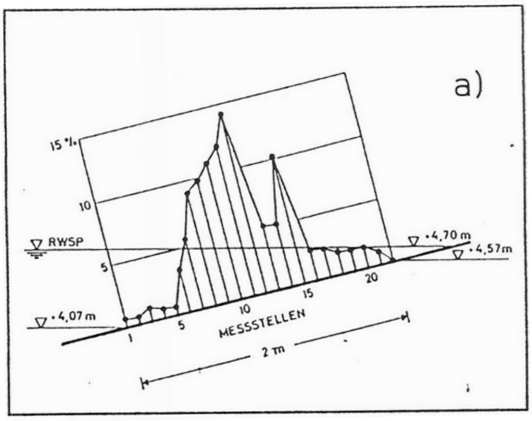
Grüne (1988) also analyzed the relation between the maximum pressure and the rising time. Based on field experiments on the island of Wangerooge he found that the higher the maximum pressure the lower the rising time.



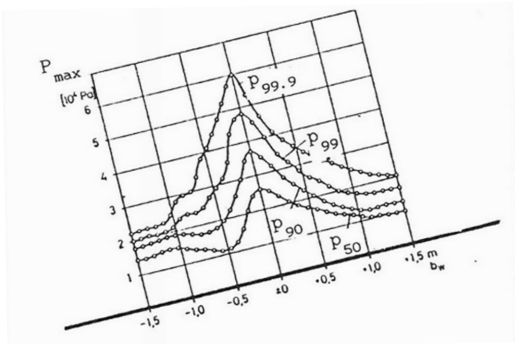
About the duration of the impact pressure is not much known. Only Stive and Grüne (1988) report on the duration of wave impacts. Stive indicates the impact duration by the parameter t_2 and Grüne by Δt_{dk}. The typical durations Stive found are $t_2/T=0.01$ for $\chi=0.005$ and $t_2/T=0.1$ for $\chi=0.01$. These results indicate increasing impact duration with increasing wave steepness.

8.4.6 Spatial variation

The impact point of breaking waves on a slope varies per event. Even if the incident wave field consists of regular waves with constant height, the position of the impact varies. These variations are due to stochastic effects related to the amount of air trapped into the water, affecting the breaking process of the wave.



It is found by Witte that 60% of maximum impact pressures for regular waves occur in the region 0.4H - 0.6H below still water level. For random waves however this position varies considerably. In a storm also the still water level will vary in time.
There is also a variation in space in the horizontal direction per impact. Of the investigations mentioned in this section only the Delta Flume experiment of breaking waves on a slope (Delft Hydraulics in 1984) treats this variation in horizontal direction. The variation in the vertical direction however is investigated by more authors.
Stive based on his full-scale experiments in 1984 in the Delta Flume proposes a width of the impact area in the order of 10% of the incident wave height. In this area impact pressures are expected with almost the same magnitude of the highest impact pressure. The area where the impact pressure will be at least 80% and 50% of the maximum impact pressure will have a width of about 40% and 80% of the incident wave height respectively. Also Witte investigated the width of the impact pressures; he analyzed the data of 175 waves.



Führböter and Sparboom (1988) published also results about the impact area and in the same year also Grüne (1988) presented his results.

8.5 References

FÜHRBÖTER, A. AND SPARBOOM, U. (1988) Full-Scale Wave Attack of Uniformly Sloping Sea Dykes, Proceedings ICCE 1988, *Malaga, Spain*

GRÜNE, J. (1988) Wave-induced Shock Pressures under Real Sea State Conditions, *Proceedings ICCE 1988, Malaga, Spain*

SCHIERECK, G.J. (2004) Introduction to bed, bank and shore protection, *Delft University Press, Delft*

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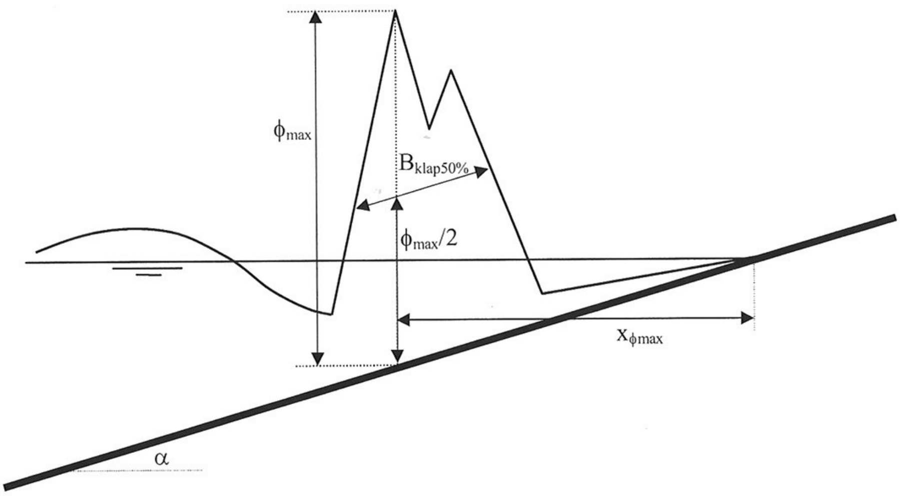
9 Summary of experiments done by WL | Delft Hydraulics from 1990 to 2004

9.1 Large scale experiments

To verify the program several experiments are used which are executed in the Delta flume and the Schelde flume. The mean goal in these experiments is to come up with some relations between the wave conditions H_s and T_p, the slope, and the properties of the wave impact. The following properties of wave impact were looked for:

- maximum pressure on the slope during wave impact, represented as a pressure height with respect to the slope: ϕ_{max}
- width of the wave impacts: $B_{klap50\%}$ (at $\phi_{max}/2$)
- the location of wave impacts: $x_{\phi max}$



In this section the findings of M. Klein Breteler (2007) are summarized. Klein Breteler used the results of several experiments done to get more knowledge about the behaviour of stone revetments under wave attack.

To get to a relation between the properties of the wave impact and the wave conditions, many experiments are analyzed. In these experiments the pressure height on the slope is measured. In the description of the experiments the following properties are described:

- the construction (the slope and type of revetment)
- the sample frequency
- distance between instruments

The measured wave conditions are given with the following variables:

- the significant wave height H_s (m) at the toe of the slope, based on the energy (H_{m0})
- the wave period T_p (s) at the peak of the wave spectrum
- the spectral measure for the wave period $T_{m-1.0}$ (s)

- the wave steepness $s_{op}=H_s/L_{op}$ (-), where $L_{op}=gT_p^2/2\pi$ (m) is the wavelength based on the peak-wave period, measured back to deep water, and g (m/s^2) gravitational acceleration
- breaker parameter $\xi_{op} = \frac{\tan\alpha}{\sqrt{s_{op}}}$ (-), where $\tan\alpha$ (-) is the slope.

Klein Breteler selected out of the total experiments some test based on requirements like; the distance between the transducers, the duration of the test, the quality of additional information.

9.1.1 Delta Flume 1991

In 1991 Wouters did experiments with waves breaking on placed stone revetments, placed on a slope of 1:3. The test duration was 4000 s, about 1000 waves. A JONSWAP spectrum is used. The top layer contained 9 pressure transducers below still water. The relative distance between these transducers was $d_{dro}/H_s \approx 0.5$ or 0.7 where d_{dro} is the distance between the transducers. The sampling rate of the measured pressures was 50 Hz.

9.1.2 Delta Flume 1992

This experiment is executed by Derks and Klein Breteler. The revetment consisted of asphalt with a slope of 1:4. In the selected tests a Pierson-Moskowitz spectrum is used and 28 transducers where placed in the slope of which 3 were broken. The duration of the test varied between 3600 s and 5400 s (700 or 1300 waves). The mean distance between the transducers, in the relevant part, measured along the slope, was 21 or 26 cm. The relative distance was $d_{dro}/H_s \approx 0.17$ or 0.28. The sampling rate was 100 Hz.

9.1.3 Delta Flume 1997/1998

From this experiment 25 tests are selected to use in the analysis. The revetment consisted of placed stones on a slope of 1:3.5. In the slope a berm is constructed with a width of 5 m and a height of 5 m above the bottom of the flume. Which tests are used are stated in the table below, in the tests different blocks are used. The table shows the difference between the tests. The duration of the test was about 1000 waves and most of the tests are executed with a JONSWAP spectrum. The sampling rate of this test was 40 Hz.

In the test with the numbers 5o** 11 pressure transducers are placed under and 12 transducers on the revetment. The revetment consisted of flat concrete blocks with a diameter of D=0.21 m. The distance between the transducers on the revetment varied from 0.48 till 0.98 m, with in the most relevant part of the slope a mean value of $d_{dro} \approx 0.60$ m and $d_{dro}/H_s=0.37$ m.

The tests with the number 12ao** where done on a revetment with at the side placed concrete blocks (D=0.20 m). As in the test with the numbers 5o** also 11 transducers were placed under the revetment but now 13 transducers on the revetment. The distance between the transducers on the revetment varied from 0.10 till 0.50 m, with in the most relevant part of the slope a mean value of $d_{dro} \approx 0.30$ m and $d_{dro}/H_s=0.24$ or 0.36 m.

Also Haringman blocks (D=0.20 m) were used in the test with number 21o**. These blocks were also placed on their side. In test number 21o** and 23o** the transducers were almost on the same place. The distance between the transducers measured in the direction of the wave flume varied between 0.09 and 0.40 m. In the relevant part of the slope the mean distance was: d_{dro}≈0.19 m and d_{dro}/H_s≈0.11 or 0.38.

Proef	h (m)	H_s (m)	T_p (s)	s_{op} (-)	ξ_{op} (-)	cotα (-)	Spectrum
5o05	4,02	1,640	5,60	0,033	1,56	3,5	J
5o06	4,00	1,640	6,90	0,022	1,92	3,5	J
12ao7	4,63	0,840	4,40	0,028	1,71	3,5	J
12ao9	4,91	1,270	5,50	0,027	1,74	3,5	J
21o02	4,55	0,728	3,26	0,044	1,36	3,5	J
21o03	4,80	0,761	4,29	0,026	1,75	3,5	J
21o06	4,70	0,737	4,28	0,026	1,78	3,5	P-M
21o08	4,80	0,754	5,08	0,019	2,09	3,5	J
21o11	4,85	0,939	4,83	0,026	1,78	3,5	J
21o14	4,98	1,563	6,36	0,025	1,82	3,5	J
21o15	5,01	1,699	6,43	0,026	1,76	3,5	J
21o16	5,00	1,550	7,64	0,017	2,19	3,5	J
23o01	4,55	0,430	3,16	0,028	1,72	3,5	J
23o02	4,55	0,524	3,55	0,027	1,75	3,5	J
23o04	4,55	0,530	3,56	0,027	1,75	3,5	J
23o08	4,95	0,940	4,82	0,026	1,77	3,5	J
23o10	4,97	1,334	5,53	0,028	1,71	3,5	J
23o11	4,94	1,543	6,35	0,025	1,83	3,5	J
AS202	5,01	1,430	7,14	0,018	1,87	4	P-M
AS401	5,10	1,510	8,65	0,013	2,20	4	P-M
AS601	4,65	0,760	2,98	0,055	1,07	4	P-M
sz626	4,96	0,724	4,10	0,028	2,01	3	J
sz627	4,94	1,056	3,90	0,044	1,58	3	J

Tabel 6 Deltagootproeven: gerealiseerde condities in proeven uitgevoerd in 1997/1998 (5o05 t/m 23o11), in 1992 (AS202 t/m AS601) en in 1991 (sz626 en sz627)

9.1.4 Delta Flume 2004

In 2004 Klein Breteler did experiences with Hydroblocks and relatively long waves. The blocks are placed on a slope of 1:3.5 and 12 tests were executed. 11 of these tests are considered in the analysis. Again the duration of the test was about 1000 waves except for test P26; this test consisted out of 438 waves. The pressure is measured with 21 transducers on the revetment and 13 transducers under the revetment. In the area where the highest impacts were expected the transducers were installed with a spacing of 0.23 m. At the sides of this area the spacing was larger, till 0.97 m. The mean distance at the relevant part of the slope was: d_{dr0}≈0.35 m and d_{dro}/H_s≈0.4 or 0.7. The measured pressures are saved in files with a frequency of 200 Hz.

Proef	h (m)	H_s (m)	T_p (s)	$T_{m-1,0}$ (s)	s_{op} (-)	ξ_{op} (-)	cotα	Spectrum *
P10	4,20	0,27	4,47	4,14	0,009	3,07	3,5	P-M
P11	4,20	0,48	8,99	7,90	0,004	4,63	3,5	P-M
P12	4,51	0,56	9,95	8,87	0,004	4,75	3,5	P-M
P13	4,80	0,65	11,31	9,34	0,003	5,01	3,5	P-M
P14	5,02	0,76	12,16	9,97	0,003	4,98	3,5	P-M
P15	5,17	0,84	12,82	10,44	0,003	4,99	3,5	P-M
P16	5,18	0,96	11,74	9,50	0,004	4,28	3,5	P-M
P22	4,40	0,51	6,86	6,17	0,007	3,43	3,5	P-M
P23	4,41	0,63	7,64	6,89	0,007	3,44	3,5	P-M
P24	4,75	0,74	8,37	7,38	0,007	3,47	3,5	P-M
P25	4,80	0,89	9,21	7,90	0,007	3,49	3,5	P-M
P26	4,90	1,05	9,46	8,19	0,008	3,30	3,5	P-M

Tabel 7 Deltagootproeven: gerealiseerde condities in proeven uitgevoerd in 2004

9.2 Small scale experiments

9.2.1 Schelde Flume 1993

In the investigation done in the Schelde Flume was aimed to get answers about loads on slopes, wave run up and wave overtopping. The experiments are performed on two different slopes, namely 1:3 and 1:4 and with a normal wave steepness ($0.01<s_{op}<0.04$, with $s_{op}=H_s/1,56T_p^2$).

Proef	H_s (m)	T_p (s)	s_{op} (-)	ξ_{op} (-)	cotα (-)	Spectrum	bijzonderheid
P3315	0.19	1.66	0.045	1.57	3	Jonswap	voorland, h_{teen} = 0,6 m
P3006	0.17	1.69	0.038	1.70	3	Jonswap	h = 0,6 m, SG
P3003	0.17	1.95	0.029	1.95	3	Jonswap	h = 0,6 m, SG
P4315	0.19	1.60	0.049	1.13	4	Jonswap	voorland, h_{teen} = 0,6 m
P4003	0.16	1.95	0.028	1.50	4	Jonswap	h = 0,6 m, SG
P4002	0.14	2.21	0.019	1.82	4	Jonswap	h = 0,6 m, SG
P4001	0.10	2.48	0.010	2.44	4	Jonswap	h = 0,6 m, SG

These tests were selected to make the comparison with the full scale test of 1997/1998 in the Delta Flume. The duration of the test was 1000 waves. The pressure is measured with 29 or 30 transducers under the still water line. In the area of highest impact the distance between the transducers was about 0.039 m. At the borders of this area the distance is 0.24 m and 0.39 m. The mean distance between the transducers in the relevant part is: $d_{dro}≈0.041$ or 0.047 m and $d_{dro}/H_s≈0.21$ or 0.40. The sample frequency of the measured pressures was 50 Hz.

9.2.2 Schelde Flume 2003

To investigate the influence of wave steepness on the stability of stone revetments, this experiment is executed in 2003. The top layer was constructed of thick wood to get the construction as stiff as possible. An aluminium plate with pressure transducers was constructed in the middle of the slope. There are 33 test executed on slopes of 1:3 and 1:4.

Proef	H_s (m)	T_p (s)	$T_{m-1,0}$ (s)	s_{op} (-)	ξ_{op} (-)	cotα (-)	Spectrum
T301	0.216	2.82	2.579	0.017	2.52	3	P-M
T302	0.220	3.08	2.832	0.015	2.74	3	P-M
T303	0.221	3.42	3.029	0.012	3.03	3	P-M
T304	0.220	3.54	3.202	0.011	3.15	3	P-M
T305	0.197	3.57	3.240	0.010	3.35	3	P-M
T306	0.171	3.57	3.224	0.009	3.60	3	P-M
T307	0.173	3.81	3.395	0.008	3.81	3	P-M
T308	0.173	4.00	3.569	0.007	4.00	3	P-M
T309	0.147	4.00	3.500	0.006	4.34	3	P-M
T310	0.148	4.05	3.615	0.006	4.39	3	P-M
T311	0.120	4.00	3.525	0.005	4.81	3	P-M
T312	0.121	4.30	3.767	0.004	5.15	3	P-M
T313	0.095	4.08	3.741	0.004	5.51	3	P-M
T314	0.096	4.60	3.921	0.003	6.19	3	P-M
T315	0.098	5.29	4.139	0.002	7.04	3	P-M

Proef	H_s (m)	T_p (s)	$T_{m-1,0}$ (s)	s_{op} (-)	ξ_{op} (-)	cotα (-)	Spectrum
T401	0.197	3.43	3.105	0.011	2.42	4	P-M
T402	0.197	3.78	3.373	0.009	2.66	4	P-M
T403	0.172	3.81	3.395	0.008	2.87	4	P-M
T404	0.146	3.81	3.374	0.006	3.11	4	P-M
T405	0.146	4.04	3.587	0.006	3.30	4	P-M
T406	0.120	4.04	3.549	0.005	3.64	4	P-M
T407	0.120	4.10	3.720	0.005	3.70	4	P-M
T408	0.122	4.52	3.908	0.004	4.04	4	P-M
T409	0.094	4.10	3.773	0.004	4.17	4	P-M
T410	0.095	4.51	3.932	0.003	4.57	4	P-M
T411	0.095	4.53	4.096	0.003	4.59	4	P-M
T412	0.097	5.78	4.338	0.002	5.80	4	P-M
T413	0.081	5.57	4.415	0.002	6.12	4	P-M
T414	0.076	5.79	4.533	0.001	6.56	4	P-M
T415	0.074	6.34	4.750	0.001	7.28	4	P-M

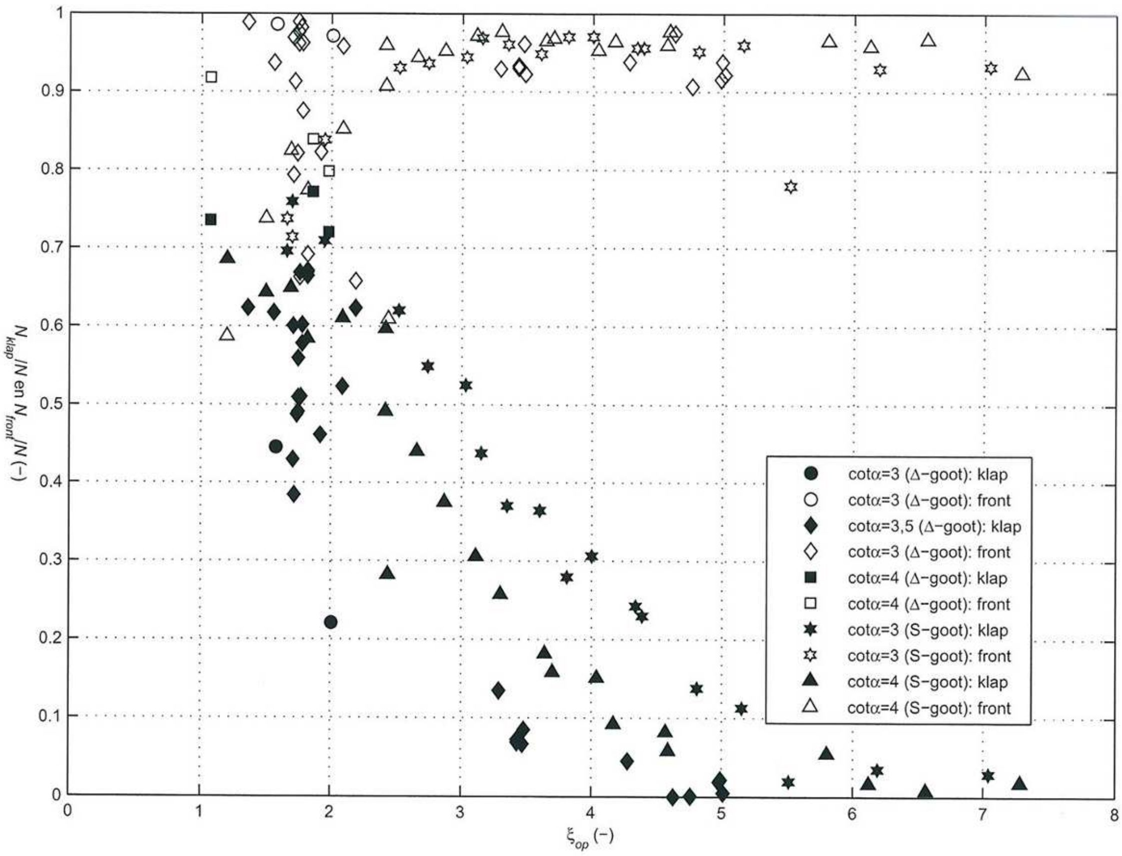
Proef	H_s (m)	$*T_p$ (s)	$T_{m-1,0}$ (s)	s_{op} (-)	ξ_{op} (-)	cotα (-)	Spectrum
T501	0.142	3.03	1.856	0.010	2.51	4	dubb. topp
T502	0.147	4.19	2.333	0.005	3.42	4	dubb. topp
T503	0.157	3.05	2.790	0.011	2.40	4	dubb. topp

The duration of the test was about 1000 waves where in the tests with number T5** a spectrum with two tops is used. There where 42 transducers installed and in the area where the most impacts were expected the distance between them was 0.021 m, at the sides of this area 0.20 m. The mean distance d_{dro}≈0.034 m and d_{dro}/H_s≈0.16 or 0.34 (except for test T413 till T415: d_{dro}/H_s≈0.45). The sampling rate of the transducers was 2000 Hz and in the analysis they are filtered to get a rate of 100 Hz.

9.3 Analysis of the experiments

9.3.1 Number of waves

The relation between the number of wave impacts and the number of waves in the experiment is called the relative number of waves. They are given in the next picture:



Where: $\xi_{op} = \tan\alpha / \sqrt{s_{op}}$, Δ-goot= Delta Flume, S-goot= Schelde Flume. The black dots represent a wave impact and the white dots a wave front.

The next experiments are taken into account is the picture:

- cotα=3 (Δ-goot): Wouters in 1991 (with 1.6<ξ_{op}<2.0)
- cotα=3.5 (Δ-goot): Klein Breteler in 2000 (with 1.4<ξ_{op}<2.2) and Klein Breteler (et.al.) in 2006 (with 3.3<ξ_{op}<5.0)
- cotα=4 (Δ-goot): Derks and Klein Breteler in1992 (with 1.1<ξ_{op}<2.2)
- cotα=3 (S-goot): Van der Meer and De Waal in 1993 (with 1.6<ξ_{op}<2.0) and Kuiper and Van Vossen in 2003 (with 2.5<ξ_{op}<7.0)
- cotα=4 (S-goot): Van der Meer and De Waal in 1993 (with 1.1 <ξ_{op}<2.0) and Kuiper and Van Vossen in 2003 (with 2.4 <ξ_{op}<7.0)

In the picture can be seen that for most of the waves a wave front develops. In some of the experiments less wave impacts are observed. This can be explained by the fact that not all the waves are recorded by the instruments, this because not enough instruments are placed or the impact is too weak to notice. Because of scale effects, there are more weak impacts when a large scale experiments are executed and these are more difficult to record.
The trend for experiments in de S-flume can be described with the following equation:

$$\frac{N_{impact}}{N} = 1 - \frac{0.057}{\sqrt{s_{op}}}; \quad \text{for } 0.004 \leq s_{op} < 0.05$$

with:

N_{klap} = number of detected impacts

N = number of waves

9.3.2 Maximum pressure height on the slope

The maximum pressure height on the slope is defined as the maximum pressure at impact divided by ρg.

$$\phi_{max} = \frac{p_{max}}{\rho g}$$

with:

ϕ_{max} = maximum pressure height on the slope at wave impact (m)

p_{max} = maximum pressure on the slope at wave impact (Pa)

ρ = specific weight of water (kg/m^3)

g = gravitational acceleration (m/s^2)

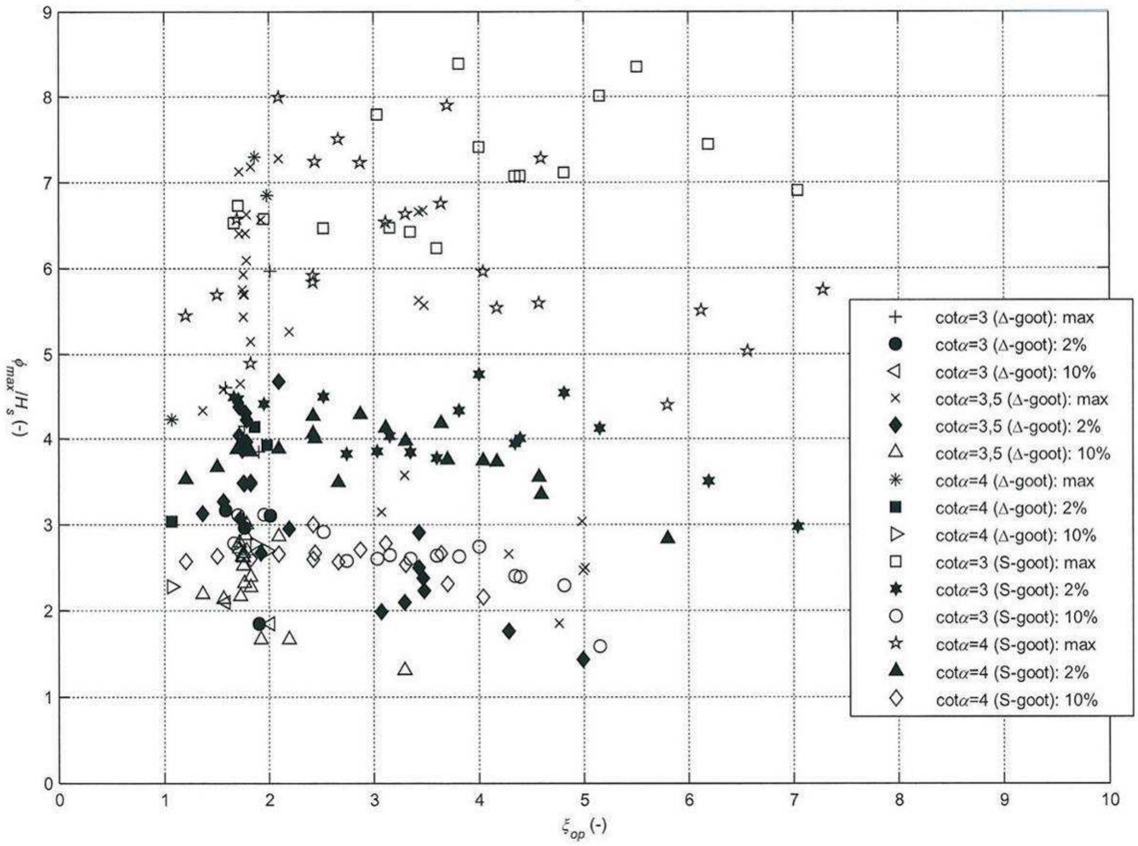
Because the maximum pressure height is different for every wave some statistical distinctions are made:

$\phi_{max,max}$ = the highest value of ϕ_{max} in the test (m).

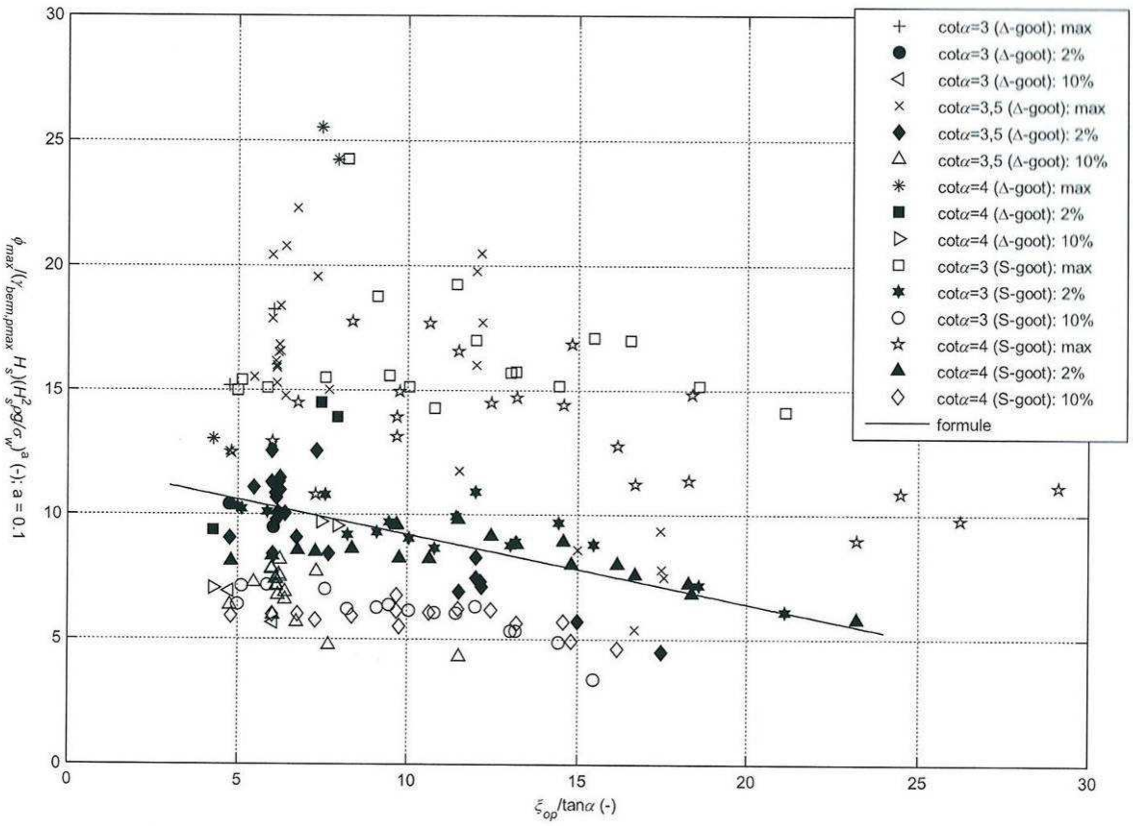
$\phi_{max,\ 2\%}$ = value of ϕ_{max} with a probability of exceedance frequency of 2% in the test, related to the amount of waves (m).

$\phi_{max,\ 10\%}$ = value of ϕ_{max} with a probability of exceedance frequency of 10% in the test, related to the amount of waves (m).

The results are stated in the next picture. It can be noticed that the maximum values of ϕ_{max} have a lot of scatter but the values of $\phi_{max,\ 2\%}$ and $\phi_{max,\ 10\%}$ have less scatter. In the analysis the values of $\phi_{max,\ 2\%}$ are used. In 2006 Klein Breteler studied the measuring points and concluded that the scatter was caused by scale effect and the presence of the berm in the flume.



Klein Breteler in his analysis uses a Froude-Weber scaling to reduce the scatter which is causes by scale effects. Also the influence of the berm is taken into account which resulted in the following picture.



9.3.3 Width of the wave impact

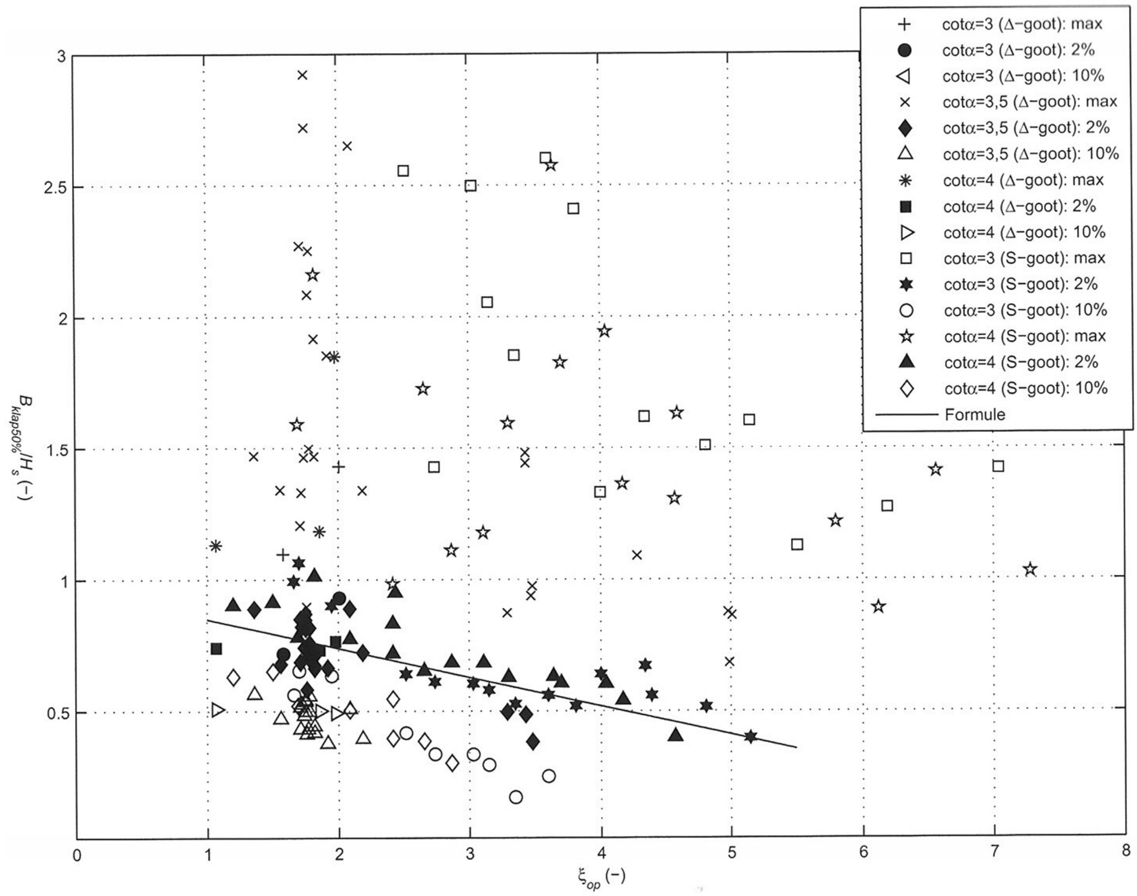
In the introduction of this chapter the width of the impact is already defined as: $B_{klap50\%}$ (at $\phi_{max}/2$). The 33% highest waves were analyzed, with as a consequence that for some test only a few waves could be used. The relation between the measurements and the breaker parameter can be stated in the formulae:

$$\frac{B_{klap50\%2\%}}{H_s} = 0.96 - 0.11\xi_{op} \quad \text{for } 1 \leq \xi_{op} < 5.5$$

With:

$B_{klap50\%2\%}$ = width of the wave impact half the wave impact height with 2% probability of exceedance (m).

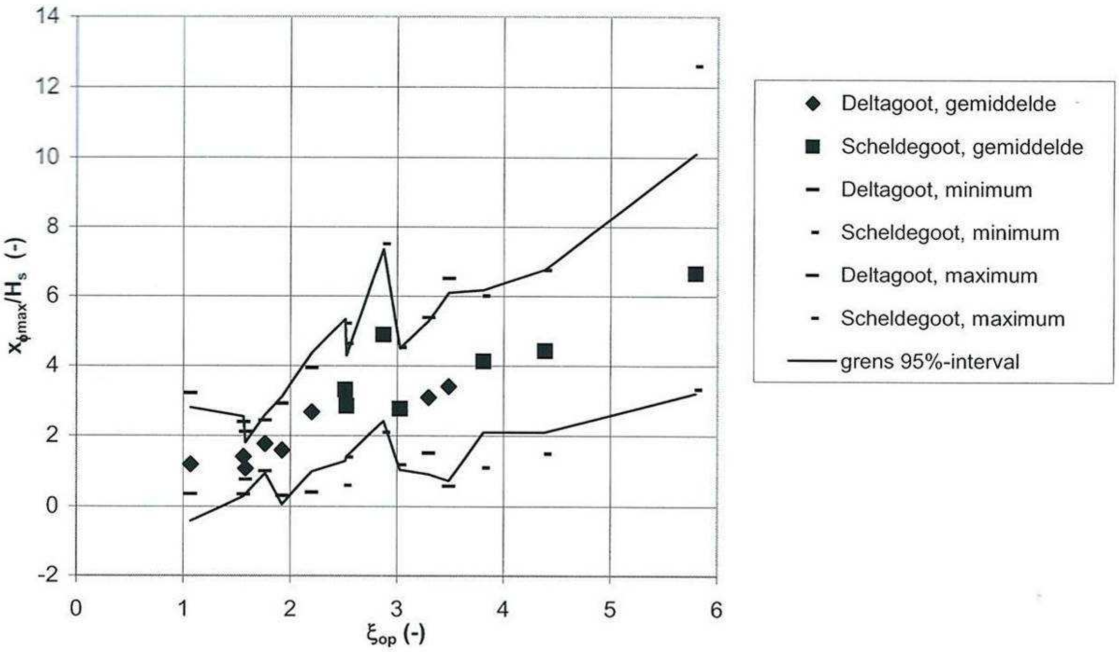
Also is the relation between ϕ_{max} and $B_{klap50\%}$ is studied and no relation is found.

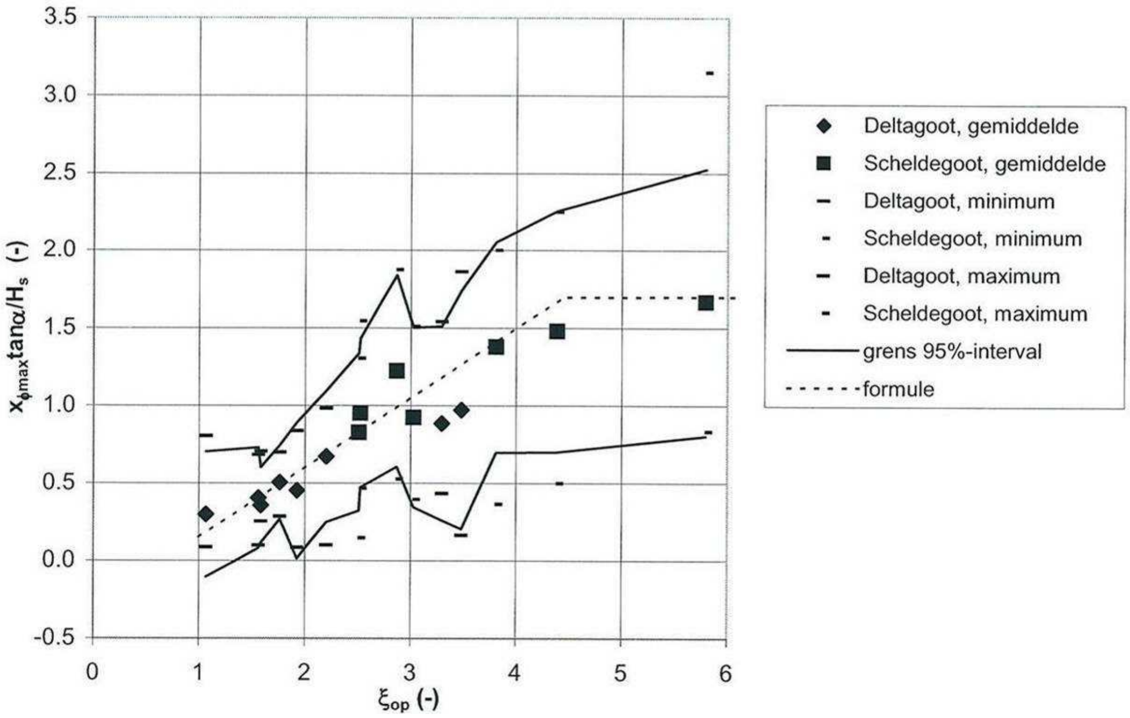


9.3.4 Location of the wave impacts

To determine the normative location of the wave impacts as a function of wave conditions and slope, the 50 biggest waves are selected from several tests. A statistical analysis of the location of impact is done with as result the next picture.

proef	H_s (m)	T_p (s)	$\tan\alpha$ (-)	s_{op} (-)	ξ_{op} (-)
as401	1,51	8,65	0,25	0,013	2,20
as601	0,76	2,98	0,25	0,055	1,07
sz627	1,06	3,90	0,33	0,045	1,58
5o05	1,64	5,60	0,29	0,034	1,56
5o06	1,64	6,90	0,29	0,022	1,92
21o15	1,70	6,43	0,29	0,026	1,76
P26	1,05	9,46	0,29	0,008	3,29
P25	0,89	9,21	0,29	0,007	3,48
t301	0,22	2,82	0,33	0,017	2,52
t303	0,22	3,42	0,33	0,012	3,03
t307	0,17	3,81	0,33	0,008	3,81
t310	0,15	4,05	0,33	0,006	4,39
t403	0,17	3,81	0,25	0,008	2,87
t412	0,10	5,78	0,25	0,002	5,79
t501	0,14	3,03	0,25	0,010	2,51





The trend line of the mean values can be expressed as:

$$\frac{x_{\phi \max} \tan \alpha}{H_s} = \min\{0.45\xi_{op} - 0.3; 1.7\} \qquad \text{for } 1 \leq \xi_{op} < 6$$

Where $x_{\phi max}$ = the horizontal distance between the point where still water and slope cross, till the location of highest pressure height of wave impact, in meters.

9.4 References

KLEIN BRETELER, M. (2007) Validatie van GOLFKLAP (Report H4134) (in Dutch), *WL│Delft Hydraulics*

10 ICCE publications about wave impact

In chapter eight international research done till 1990 is already treated. Führböter, Sparboom, Grüne and Witte did most of this research in Germany. In chapter nine the findings of Klein Breteler are summarized, he investigated the experiments done by WL | Delft Hydraulics from 1990 till 2004. In this chapter the main findings from 1990 until now presented on the International Conference on Coastal Engineering (ICCE) will be summarized. First, the coastal engineering manual is consulted about wave impact on asphalt revetments.

10.1 ICCE 1990, Delft, The Netherlands

In the proceedings of the year 1990 a paper was found with the title; "a model for breaking wave impact pressures". This paper is published by Cooker and Peregrine and is about wave impacts on vertical walls. From the paper it becomes clear that more knowledge needed to be obtained to prevent further damage on coastal structures. The paper discusses the way the impact force should be schematized; it is clearly the almost first step in research on wave impacts on vertical walls. Because Peregrine published much more papers about this subject, no details of this paper will be displayed here.

10.2 ICCE 1992, Venice, Italy

In the proceedings, much subjects where treated. This section summarizes the paper published by Grüne with the title; "Loads on sloping seadykes and revetments from wave-induced shock pressures". In this paper Grüne first analyzes the shock pressure, some definitions are given. Then some general conclusions are made about the characteristics of the shock pressure occurrence. Grüne tries to give some relations between the maximum pressure and the compression and decompression time. It becomes clear that higher peak values of the shock pressure only occur with decreasing water sheet thickness. Shown is that the maximum shock pressure have relation with the compression and decompression time. Also Grüne compares the experiment done in Wangerooge and at the Eiderdam with experiments done in a wave flume. He states that when a comparison is made between field and laboratory date from regular waves, test both should be related to mean wave height H_m. Last but not least Grüne presents; A "dynamic" loading model. The results of his comparison between the different parameters leaded to the table presented with the picture. In this table the margins of the parameters are summarized.

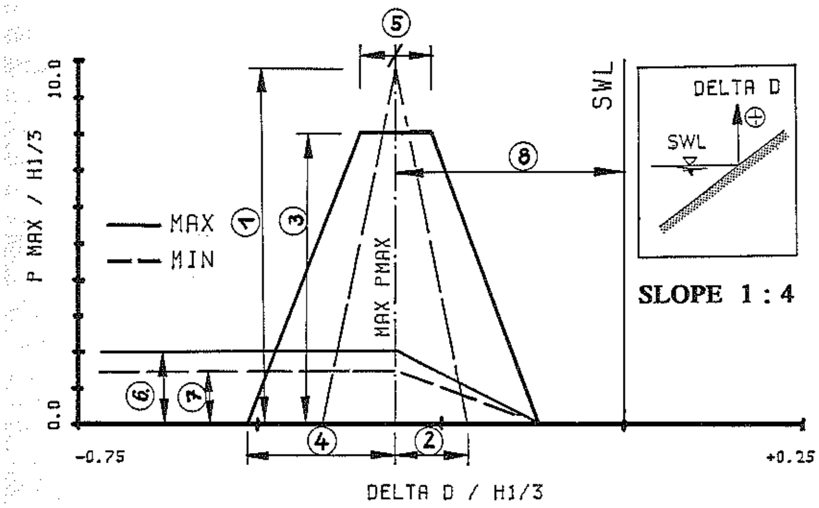


Fig. 17 "Dynamic" loading model for slope 1 : 4

			RANGE	
S	1	peakpressure	3.0 <= Pmax / H1/3 <= 10.0	MIN
S	2	acting width	0.10 <= Delta D / H1/3 <= 0.25	MIN
S	3	peakpressure	3.0 <= Pmax / H1/3 <= 8.0	MAX
S	4	acting width	0.15 <= Delta D / H1/3 <= 0.30	MAX
S	5	acting width	0.05 <= Delta D / H1/3 <= 0.10	MAX
W	6	wavepressure	Pmax / H 1/3 <= 2.0	MAX
W	7	wavepressure	Pmax / H 1/3 <= 1.5	MIN
S	8	acting point	-1.0 <= Delta D / H 1/3 <= +0.5	

Table 1 Ranges for boundary conditions of the loading model in fig. 17

10.3 ICCE 1994, Kobe, Japan

The paper of interest presented at this conference is again published by Grüne. The title of this paper is; "Wave Loads on Seadykes With Composite Slopes and Berms". The investigations where done in de Large Wave Channel at Hannover in 1976. Different types of dyke cross-sections were installed.

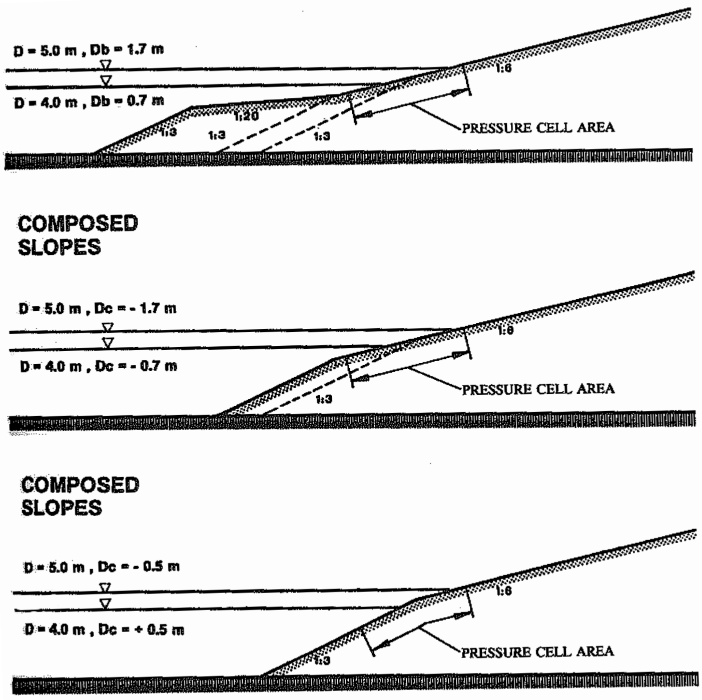


Fig. 1 Dyke cross-sections with berm and composed slopes, investigated in the LARGE WAVE CHANNEL (GWK)

The result of the analysis is shown in some pictures. First some definitions and then the influence of the berm is shown.

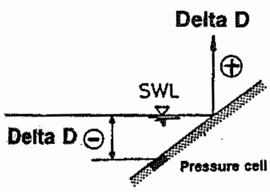


Fig. 2 Definition of *Delta D*

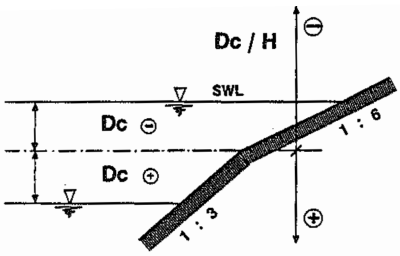


Fig. 3 Definition of *Dc*

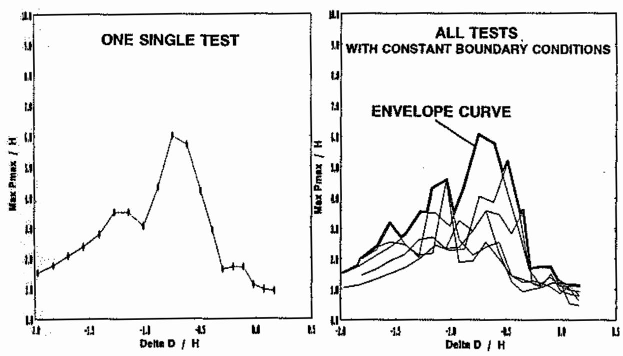


Fig. 4 Example for a local distribution of pressure data *Max Pmax/H*

In the study the relation between different wave heights, wave spectra, and berm heights is shown. Grüne concludes his paper with some statements:

Where wave trains are broken at least partly, before they reach the slope, the maximum pressure values max Pmax/H increases with decreasing absolute wave height.

Where wave trains are broken completely by the slope, the pressure values max Pmax/H may increase with increasing absolute wave height.

Regular waves always lead to smaller related pressure values max Pmax/H, if regular wave heights H are compared with irregular wave heights $H_{1/3}$.

For a slope with a berm in front, the peak pressure max Pmax/H may decrease due to breaking effect or increase due to shoaling effect depending on water depth on the berm.

For composed slopes there is a transition range, which is asymmetric with respect to the distance Dc between slope junction level and still water level.

10.4 ICCE 1996, Orlando, Florida

Most of the papers about wave impact presented at this conference are related to impacts on vertical structures. One of these papers is about the nature of high peak wave pressures. The paper is presented by Jordan Marinski and the title is; "Physical study of the Nature of High Peak Wave Pressures". In his study, the effect of air entrainment is investigated. The set-up of the experiment is shown below, where the thicknesses of the air cushion is adjustable.

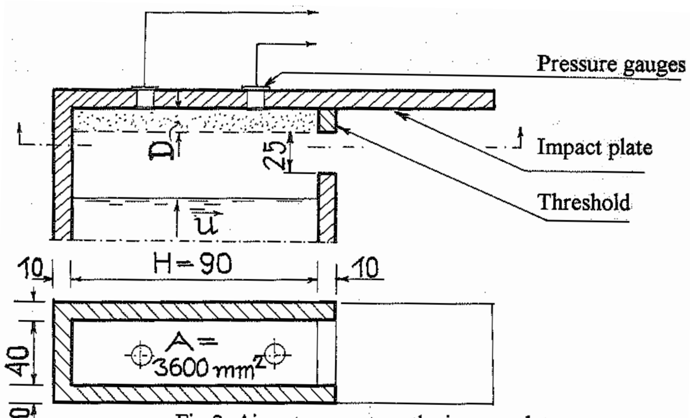


Fig.2. Air entrapment on the impact plate

Marinski divided the results in three different groups:

1. The effect of an air volume at D/H≤0.06
2. The effect of an air volume at 0.06≤D/H≤0.22
3. The effect of an air volume at 0.22≤D/H

The result is shown in the picture below. It can be seen that there is a considerable difference between the three parts.

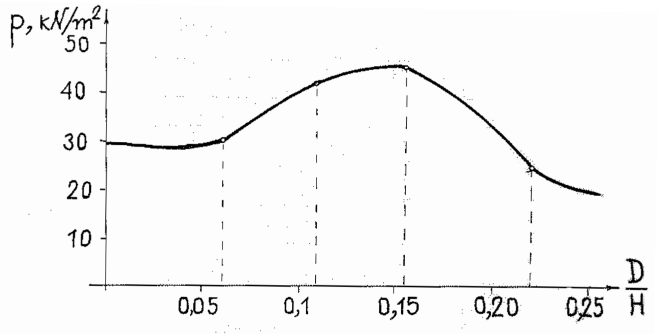


Fig. 4. Effect of dimensionless thickness (D/H) on magnitude of impact pressure

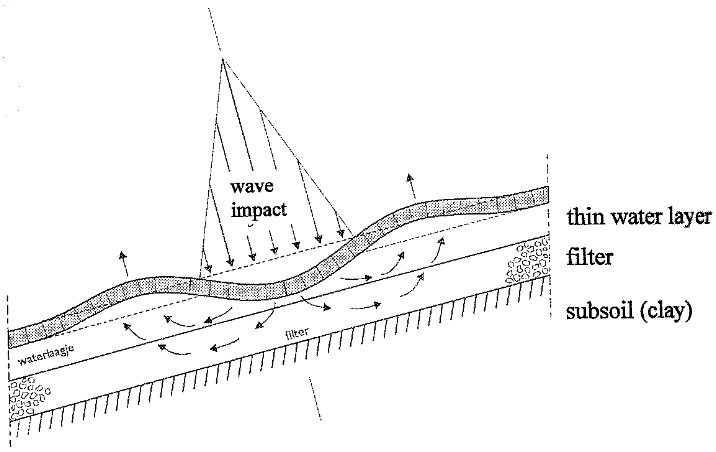
Marinski concluded his paper with the statement that the closed air volume as a physical medium under compression could not be a cause for the high peak short-period pressure.

10.5 ICCE 1998, Copenhagen, Denmark

In the papers of this conference nothing about wave impact on dike slopes is found. Again about wave impact on a vertical slope a paper is found about modelling the wave impact. The paper is presented by Deborah J. Wood and D. Howell Peregrine with the title; "Two and three-dimensional pressure-impulse models of wave impacts on structures". This paper will not be summarized here.

10.6 ICCE 2002, Cardiff, United Kingdom

At this conference an interesting paper about grouted basalt revetments is presented. It was published by Klein Breteler (et. al.) with the title; "Stability of Placed Basalt Revetments with Asphalt Grouting". In the paper an experiment done on grouted basalt revetment is described. In a field test an existing revetment in the Netherlands is tested for uplift. The revetment is loaded with an uplift pressure higher then the own weight. This resulted in the lift of the revetment what leaded to the release of the pressure. A good toe structure should prevent the filler for washing away. Also a pulling test is done, during this pulling test the pulling force was increased up to a maximum of 30 to 40 kN in a period of approximately one hour. Some theory about wave impact is also presented.



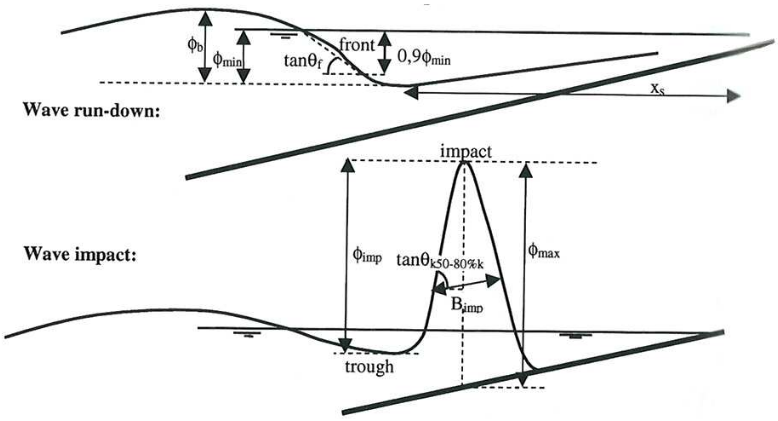
This theory is not used nowadays so only the conclusions of this paper will be displayed here;

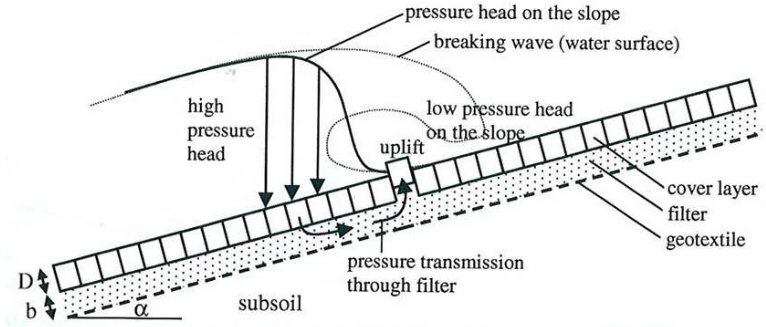
- Superficially grouted revetments are able to withstand large wave attack during a short period (less then a few hours)
- In case of a permeable toe, a static uplift pressure will only lift the revetment a few cm, without leading to serious damage. The lifted revetment has a thin water layer between the cover layer and the filter layer, which is able to discharge the water to the permeable toe. This will lead to a decrease of the freatic level and a decrease of the uplift pressure
- A revetment of deeply grouted basalt blocks shows a large upward motion during wave rundown, and only an oscillating motion during wave impact
- Deeply grouted revetments seem to be able to withstand large wave attack, although further investigations to other damage mechanism is still necessary

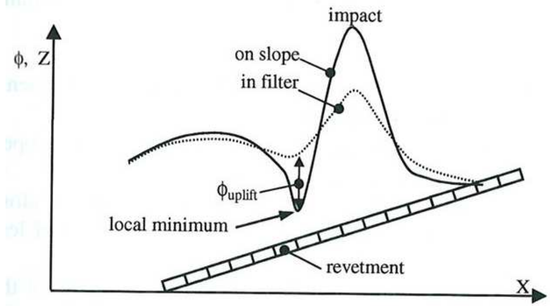
10.7 ICCE 2006, San Diego, California

On this conference held in San Diego the basics of the program GOLFKLAP are explained. In the previous chapter these basics are already explained so it will not be repeated here. Also published at this conference are the results of a study at the influence of wave steepness on stability of placed block revetments. This study is performed by Klein Breteler (et. al.) with the used of small scale and large scale tests. Also numerical calculations are made. Klein Breteler (et. al.) distinguishes two types of wave loading:

- the pressure front during wave rundown (just before wave impact occurs
- the pressure distribution during wave impact







Klein Breteler (et. al.) concluded with regard to the relation between wave load and wave steepness that for relatively long waves (ξ_{op}>2.5) the wave load:

- increases with increasing breaker parameter for low permeability revetments
- decreases with increasing breaker parameter for low permeability revetments

This trend is confirmed by the numerical calculations with Zsteen and the large-scale tests in the Delta Flume of WL│Delft Hydraulics.

10.8 References

COOKER, M.J. AND PEREGRINE, D.H. (1990) A Model for Breaking Wave Impact Pressures, Proceedings ICCE 1990, *Delft*

GRÜNE, J. (1992) Loads on Sloping Seadykes and Revetments from Wave-Induced Shock Pressures, *Proceedings ICCE 1992, Venice*

GRÜNE, J. (1994) Wave Loads on Seadykes with Composite Slopes and Berms, *Proceedings ICCE 1994, Kobe*

KLEIN BRETELER, M. ET AL (2002) Stability of Placed Basalt Revetments with Asphalt Grouting, *Proceedings ICCE 2002, Cardiff*

KLEIN BRETELER, M. ET AL (2006) Influence of Wave Steepness on Stability of Placed Block Revetments, *Proceedings ICCE 2006, San Diego*

MARINSKI, J. (1996) Physical Study of the Nature of High Peak Wave Pressures, *Proceedings ICCE 1996, Orlando*

11 The program GOLFKLAP

11.1 Introduction

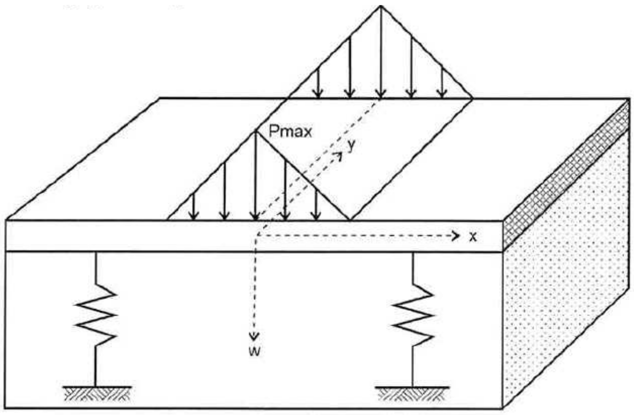
In this chapter the basics of the program GOLFKLAP itself will be discussed; a summary of the manual of GOLFKLAP is given.

11.2 History

After the publication of the guideline in 1984 some questions arose about the schematization of the load on the asphaltic revetment. The guideline of 1984 prescribed to use a line load and the result of a calculation could be; the thinner the layer the lower the stress in the layer. This result needed to be adjusted and in the years this resulted in the development of the program GOLFKLAP.

11.3 Basic theory of GOLFKLAP

The system of asphalt layer and sub soil is schematized as a plate which is supported by springs. The load is schematized as a triangle load.



The maximum stress which occurs under the load can be calculated with:

$$\sigma = \frac{p_{max}}{4\beta^2 \beta z}(1 - e^{(-\beta z)}(\cos(\beta z) + \sin(\beta z)))\frac{6}{h^2}$$

Where:

$$\beta = 4\sqrt{\frac{3c(1-\nu^2)}{Sh^3}}$$

σ	the strain at underside of the revetment (MPa)
p_{max}	maximum pressure impact (Pa)
h	layer thickness (m)
z	half width triangle load (=0,5H)
c	bed constant (MPa/m)
S	modulus stiffness (MPa)
ν	Poisson's constant for asphalt (-)

11.3.1 Wave load

The maximum pressure impact is defined as:

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

Where:

ρ_w density water (kg/m^3)

g acceleration of gravity (=9,81 m/s^2)

q impact factor, dependent on slope (-)

H_s wave height (m)

The maximum pressure impact is dependent on the wave height and the impact factor. The impact factor is used to account for the variation of wave height. The analysis of Führböter (1988) is used to compose a probability density function of the impact factor. Führböter composed the probability function of the number of waves which create an impact together with the probability function of the intensity of the wave impact. The result is:

$$p(q) = \frac{1}{\sigma_q \sqrt{2\pi}} e^{-(\frac{(q-q_{gem})^2}{2\sigma_q^2})}$$

Where:

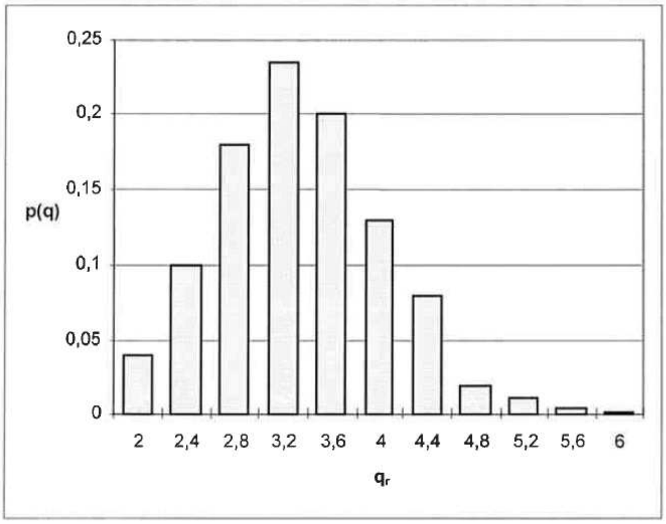
q impact factor

p(q) probability of occurrence of impact factor q

σ_q deviation of probability function

q_{gem} the mean impact factor

The probability density function for a slope of 1:4 can be displayed in a graph.



The impact factor of slope other then 1:4 can be calculated with:

$$q_\alpha = \frac{tan(\alpha)}{0.25} q_r$$

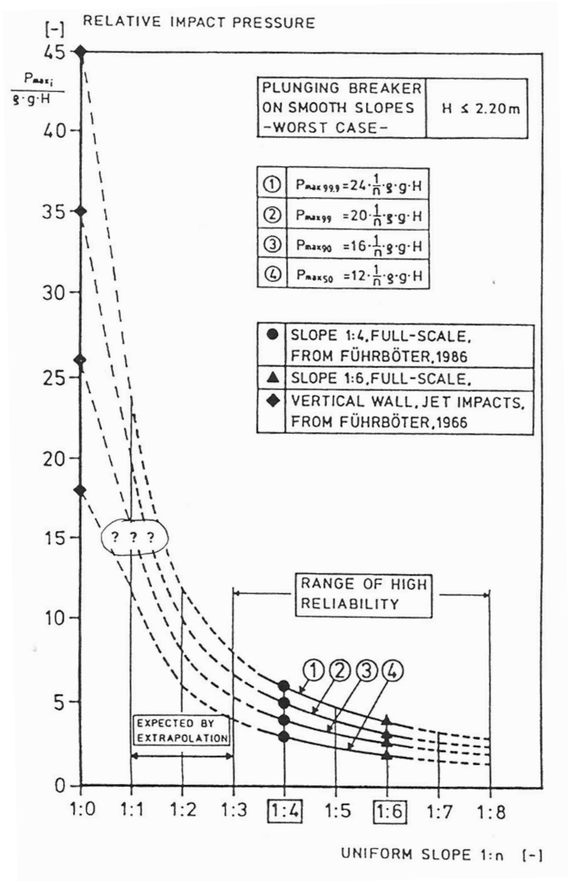
Where:

q_α impact factor for slope α (-)

tan(α) tangent of the slope α

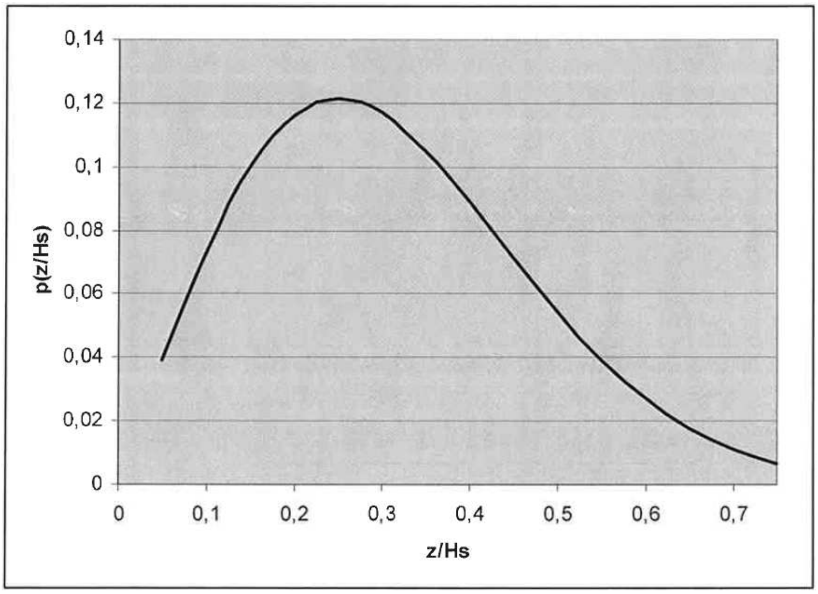
q_r impact factor for slope 1:4

This formula is based on research done by Führböter (1988).



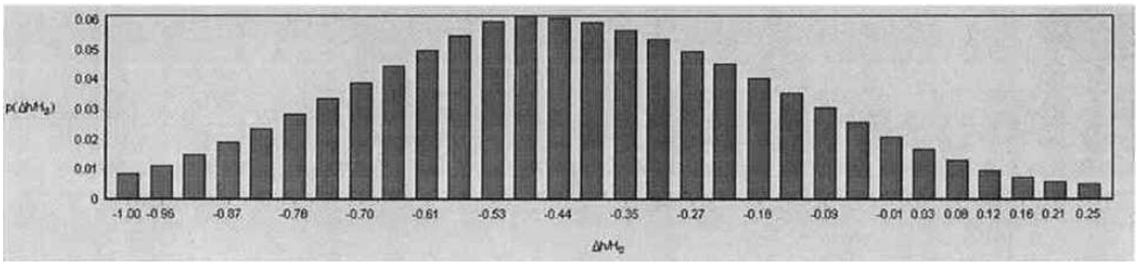
11.3.2 Width of the wave load

The width of the wave load determines the tension of the load. The width is related to the wave height and varies between 0.5 and 1.5H. The relation between the width and the height of the wave load is given in the next picture:



11.3.3 Distribution of the impact point

The distribution of the impact point is based on Grüne (1988). Grüne found that most of the wave impacts occur at $0.5H_s$ below SWL. The probability function Grüne made is based on experiments done in 1982 with real sea waves at the Eiderdam and the island Wangerooge.



11.3.4 Fatigue of the asphalt revetment

In section 6.7 is explained how the parameters (a) and (k) are obtained from a fatigue tests. These parameter are used to determine when the asphalt layer will fail due to fatigue.

$$\log N_f = a \cdot \log(\sigma) + \log(k)$$

With the rule of Miner is defined when the construction fails. The construction is safe when:

$$\sum \frac{n_i}{N_{f,i}} \leq 1$$

With: n_i = the number of times loading of load i occurs

$N_{f,i}$ = the number of loading times where the revetment fails

11.4 References

DE LOOFF, A.K. ET AL (2004) Handleiding GOLFKLAP 1.2 (Report e038390) (in Dutch), *KOAC•NPC, Nieuwegein*

FÜHRBÖTER, A. AND SPARBOOM, U. (1988) Full-Scale Wave Attack of Uniformly Sloping Sea Dykes, *Proceedings ICCE 1988, Malaga, Spain*

GRÜNE, J. (1988) Wave-induced Shock Pressures under Real Sea State Conditions, *Proceedings ICCE 1988, Malaga, Spain*

12 Conclusions

12.1 Introduction

When reading this literature review it is clear; there has been done a lot of research on the subject of wave impacts. To guarantee a certain safety level knowledge about the failure mechanisms is very important and history shows that one has to keep searching for new and safe solutions to withstand the forces of the sea.

Most of the research done before the year 1990 is done in Germany. After 1990 the attention of the German researchers (Führböter, Sparboom, Grüne, Oumeraci) was shifted to vertical walls and slender piles. Also in Britain most of the research is concentrated on vertical walls. Britain researchers one can find are; Peregrine, Bullock and Obhrai.

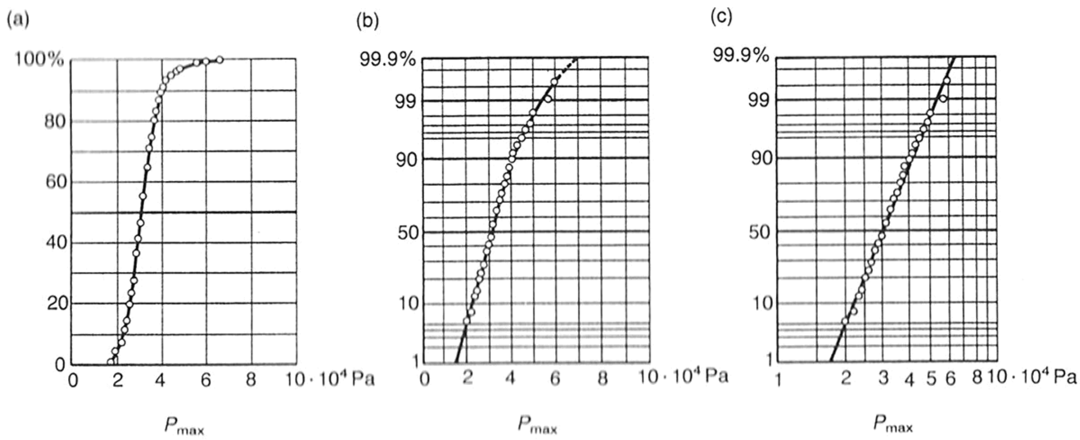
In the Netherlands, as mentioned in chapter eight, also research has been done on asphalt revetments. After 1990 a lot of the research is concentrated on stone revetments, this is summarized in chapter nine; a lot of this research is done by, or under the supervision of, Klein Breteler.

The goal of this literature review was to obtain insight into what knowledge is available at the moment and it is tried to cover most of the research by looking into guidelines, reports and papers presented on the ICCE. Also extensively use is made of knowledge obtained by the developers of GOLFKLAP namely the company KOAC•NPC, Nieuwegein, the Netherlands.

In the following paragraphs the conclusions are arranged by subject.

12.2 Log-normal distribution

All the researchers confirm the log-normal distribution of the maximum wave impact pressure. Führböter explains why the maximum wave impact is log-normally distributed. The maximum impact pressure is dependent on the elasticity of air and water, hydraulic radius of the impact area and the included air content. The included air content has a big influence on the relation between the elasticity of water and air. Also the hydraulic radius of the impact area is influenced by the included air content. This amount of air content depends on the geometry of the dike and on the incident wave conditions. Because the included air content is Gaussian distributed the other parameters are also Gaussian distributed. With his data Führböter found that the maximum impact pressure follows a log-normal distribution. Führböter (1988) based his research on twelve tests on a 1:4 slope and 13 tests on a 1:6 slope, done with 200 regular waves and also on experiments on 1:10 scale and jet-impacts.



Probability distributions of impact pressures on smooth 1:4 slope (full-scale tests with regular waves $H = 1.3$ m and $T = 4.6$ s). (a) linear; (b) Gauss; (c) normal-log.

Grüne (1988) made a comparison between the maximum impact pressure of the wave channel experiments (based on H_m) and the measured real sea state impact pressures based on H_s.

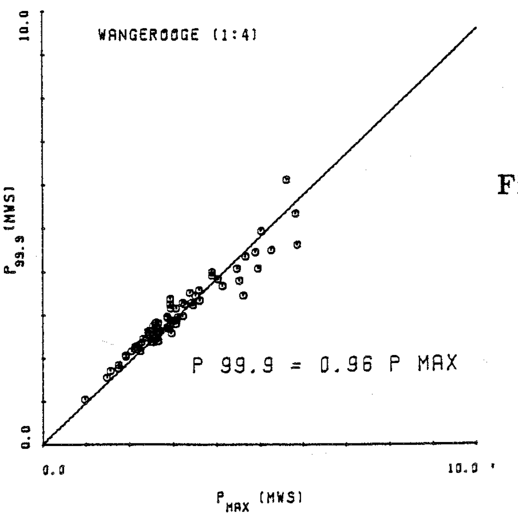


Fig. 11 Relation between statistical parameters $p_{99.9}$ and measured maximum pressure p_{max} for different time intervals, measured at WANGEROOGE location

This shows that also under real sea state conditions the maximum impact distribution is log-normally distributed. However, van Vledder concluded that the log-normal distribution of the maximum wave impacts only is checked visually and recommends to make an statistical check too.

12.3 Maximum impact pressure

A lot of researchers give some information about the maximum impact pressure in the experiment. In chapter eight and nine the findings of the researchers are displayed. The difference however is the height of maximum impact pressure that is exerted by the waves. In this section the differences will be compared with each other. It should be noted that different definitions of the impact pressure exist. There is the maximum impact pressure per transducer per test series, the maximum pressure over all transducers per test series and the maximum pressure over all transducers and over all series, so per experiment. In this section the $P_{max.max}$ is used for the highest maximum impact pressure value per experiment. Führböter and Sparboom (1988) used $P_{max.max.n}$ where the n means the probability of non-exceedance.

Van Vledder gives the following conclusions in his literature survey about the maximum wave impact:

- with even a little amount of air in the air-water mixture of the water mass the maximum impact pressure degreases significantly.
- the experiments show a proportionality of the maximum impact pressure with the tangent of the slope angle.
- the rising time t_k is inversely proportional with maximum impact pressure p_{max}.

In chapter eight is shown what most researchers found for the maximum wave impact, related to the incident wave height:

$$P_{max.max} = \text{const} \ \rho_w g H$$

Year	Investigator(s)	Slope	constant	Wave steepness (χ)
1969	Skladnev and Popov	1:4	2	0.03
1984	Stive	1:3 1:4	2.7 2.3	0.003 - 0.01 0.003 - 0.011
1984	Delft Hydraulics	1:6		0.0027 - 0.0103
1985	Zhong			0.003 - 0.009
1986	Führböter	1:4*		0.006 0.009
1988	Führböter and Sparboom	1:4* 1:6	6 4	0.0015 - 0.038
1988	Witte	1:4*		0.006
1988	Grüne	Eiderdam 1:4 Wangerooge 1:4	7 5	
		* refer to the same experiment		

This table shows the big difference in findings of the researchers. Also Klein Breteler found a relation between the maximum impact pressure, the wave height and the wave steepness (Klein Breteler 2007)

- maximum pressure height of wave impact with 2% exceedance frequency:

$$\left(\frac{\phi_{max\,2\%}}{\gamma_{berm,\phi max} H_s}\right)\left(\frac{\rho g H_s^2}{\sigma_w}\right)^{0.1} = 16 - \frac{0.36}{\sqrt{s_{op}}} \qquad \text{where } s_{op} > 0.002$$

Where:

ϕ_{max} = the maximum pressure height on the slope at impact (m).

σ_w = the surface tension of water (0.073 N/m).

$\gamma_{berm,\phi max}$ = a factor which accounts for the influence of a berm in front of the slope. It is a dimensionless parameter with the following properties (Klein Breteler, 2007):

$$\gamma_{berm,\phi max} = 0.17\left(\frac{h_b}{H_s} - 1.2\right)^2 + 1 \qquad \text{where } -0.2 \le \frac{h_b}{H_s} \le 1.2$$

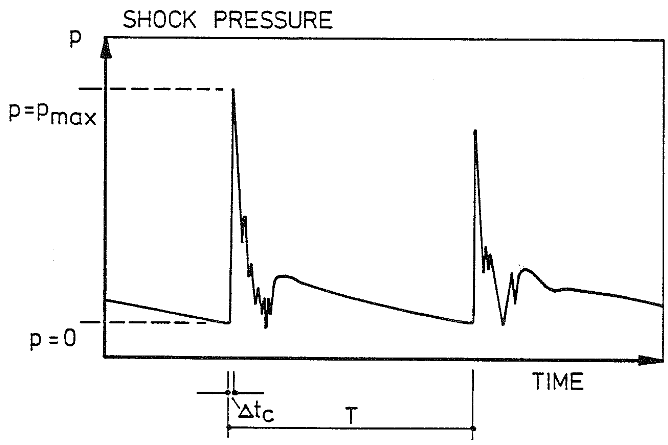
$$\gamma_{berm,\phi max} = 1 \qquad \text{where } \frac{h_b}{H_s} > 1.2$$

Because of scale effects Klein Breteler (2007) added the term $\left(\frac{\rho g H_s^2}{\sigma_w}\right)^{0.1}$.

When measuring the maximum impact pressure the sampling rate is of big importance. In section 12.4 more about the influence of the sampling rate. The impact pressure can be divided into different stages and when analysing a wave record an impact pressure has to be defined. Grüne (1988) gave the following definition:

- the rising time up to the maximum peak pressure must be much shorter than the wave period (roughly less than 1/10 of the wave period)
- the maximum peak value should exceed a certain multiple value of the wave height, that means a setting of a minimum value

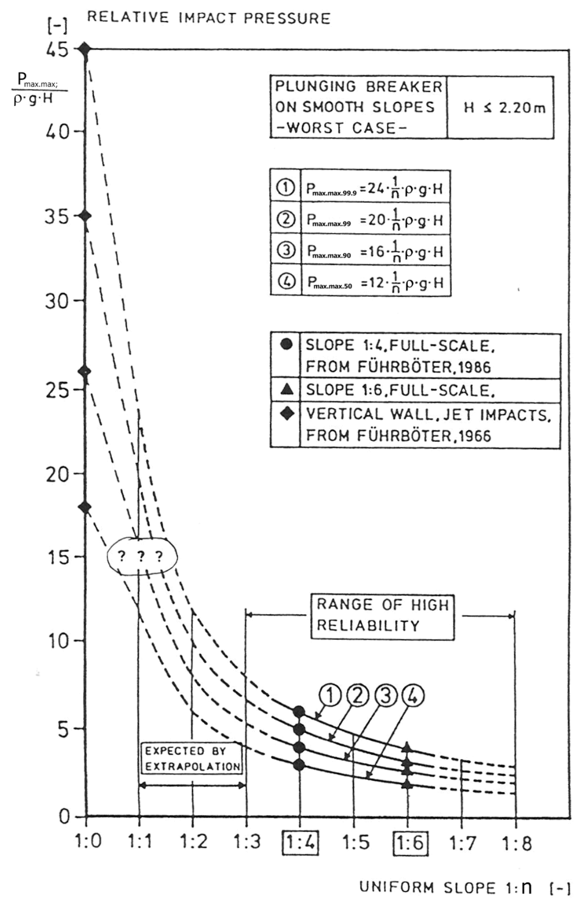
One can imagine that with a low sampling rate this maximum peak value is not always measured. Führböter and Sparboom (1988) state that peak pressure are highly effective during a relative short time (10 <Δt_c< 60 milliseconds).



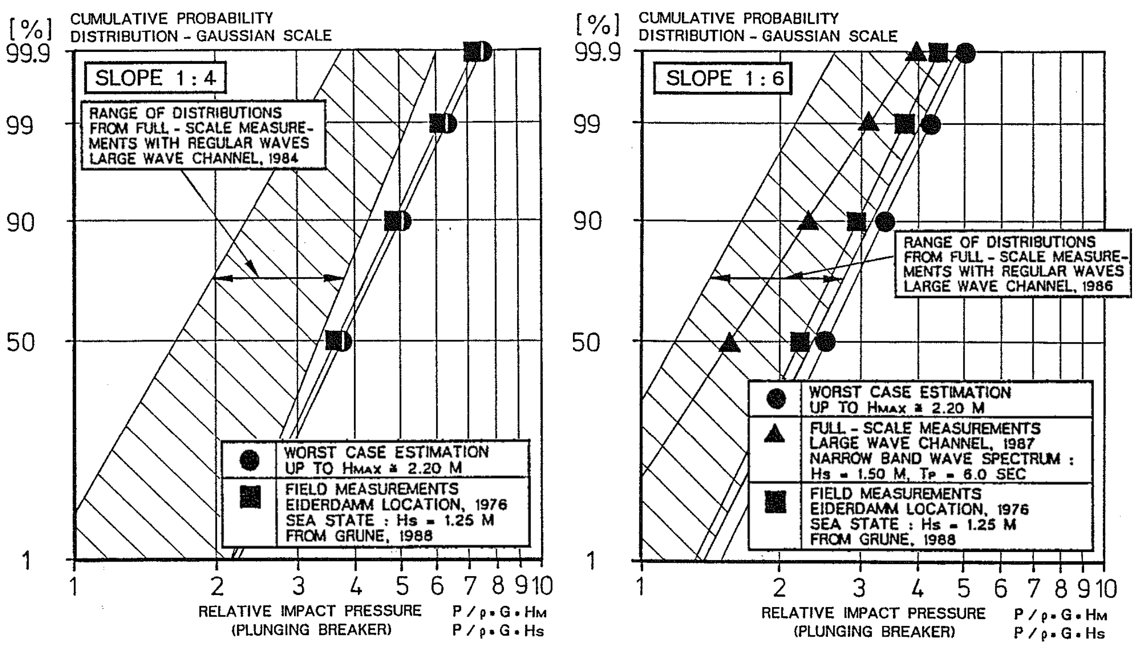
The maximum impact pressure is also dependent on the slope. This is explained by the influence of the backwash layer. Van Vledder concluded:

- the thickness of the backwash layer is important. The backwash layer reduces the impact pressure. Mild slope have thicker backwash layers then steep slopes.

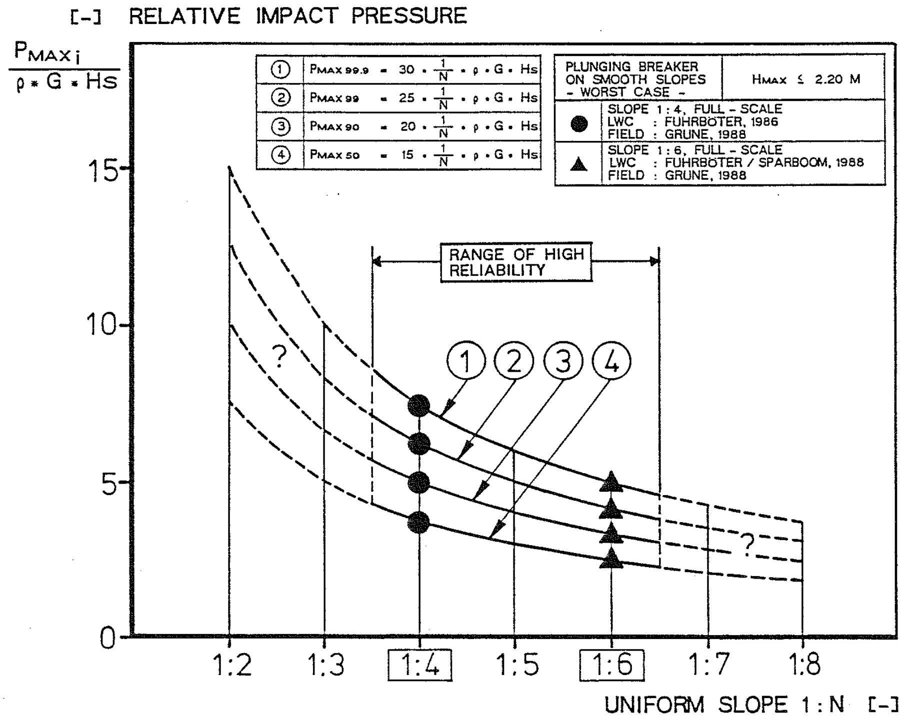
Führböter and Sparboom (1988) gave the following graph (see also chapter eight).



This graph shows the relation of the maximum impact pressure with the regular wave height. Sparboom (1991) based on Grüne (1988) made a comparison between wave height H_m (laboratory conditions) and H_s (real sea state conditions):



Sparboom (1991) gave also the relation with the slope angle.



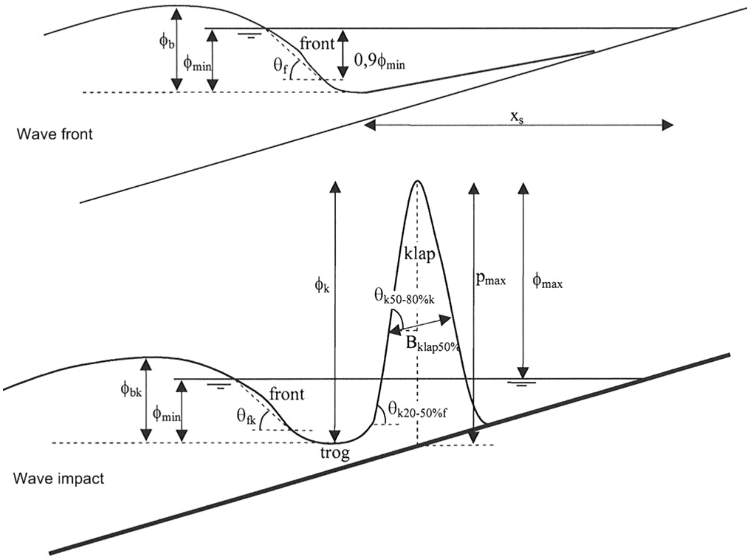
This gives that the maximum impact pressure can be found by:

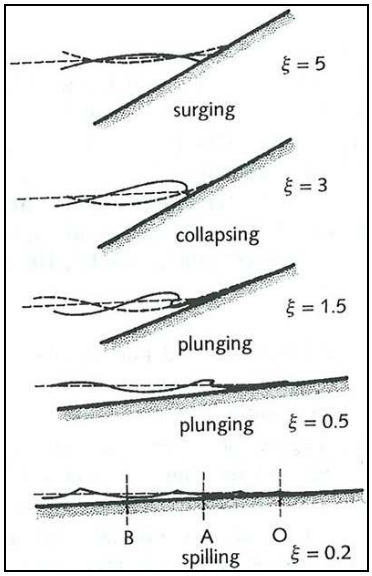
$$P_{max.max.99.9} = \frac{30}{n} \rho g H_s$$

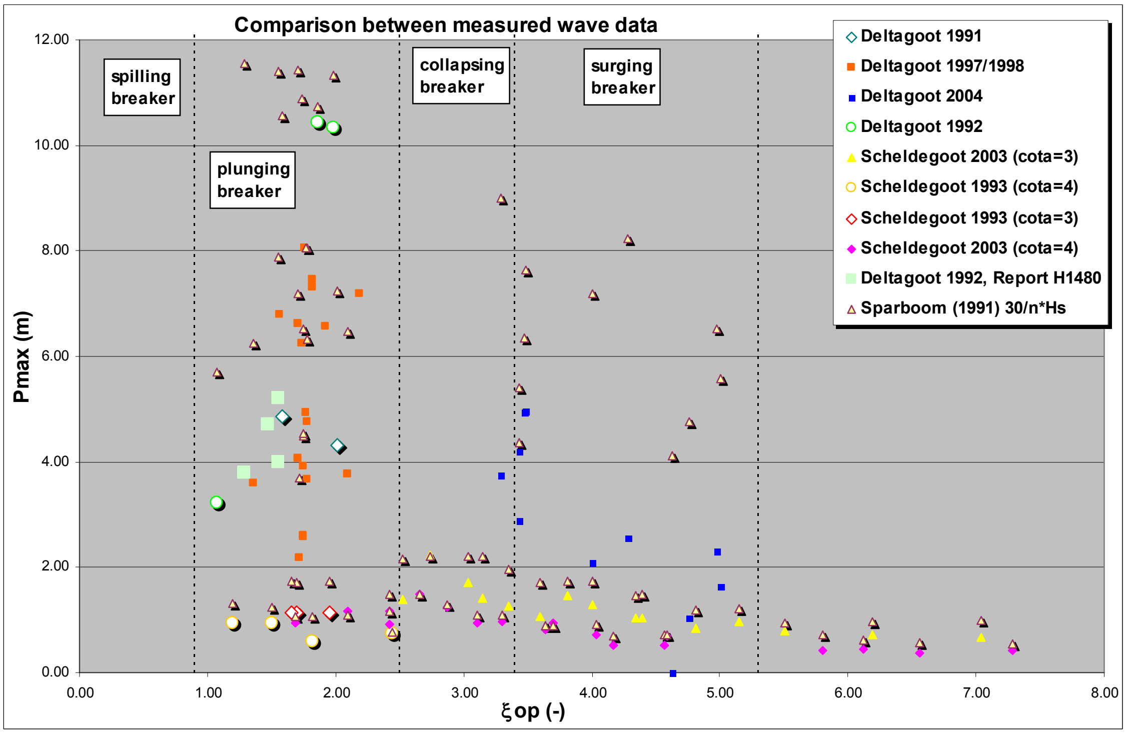
So another relation between the maximum impact pressure and the incident wave height is found. To make some conclusions the relations of Führböter and Sparboom and the ones of Klein Breteler will be compared with each other.

Klein Breteler did a lot of research into stone revetments as is shown in chapter nine and ten. Also the maximum wave impact is analyzed. Although the analysis is aimed at stone revetments a comparison is made between the data Klein Breteler used in: "KLEIN BRETELER, M.(ET AL) (2006) Kwantificering golfbelasting en invloed lange golven, Onderzoeksprogramma Kennisleemten Steenbekledingen (Conceptverslag H4421) (in Dutch), *WL | Delft Hydraulics*" and the data of Führböter and Sparboom (1988). It should be noted that the analysis done by Klein Breteler is aimed at stone revetment research. This implies that in the research described by Klein Breteler maybe another sampling rate is used than Führböter and Sparboom used.

First of all a comparison is made between the maximum impact pressure p_{max} found by Klein Breteler (2006) and the rule of thumb Sparboom (1991) gave: $P_{max.max.99.9} = \frac{30}{n} \rho g H_s$, with p_{max} as shown in the picture below. This definition of p_{max} will be used in this section.

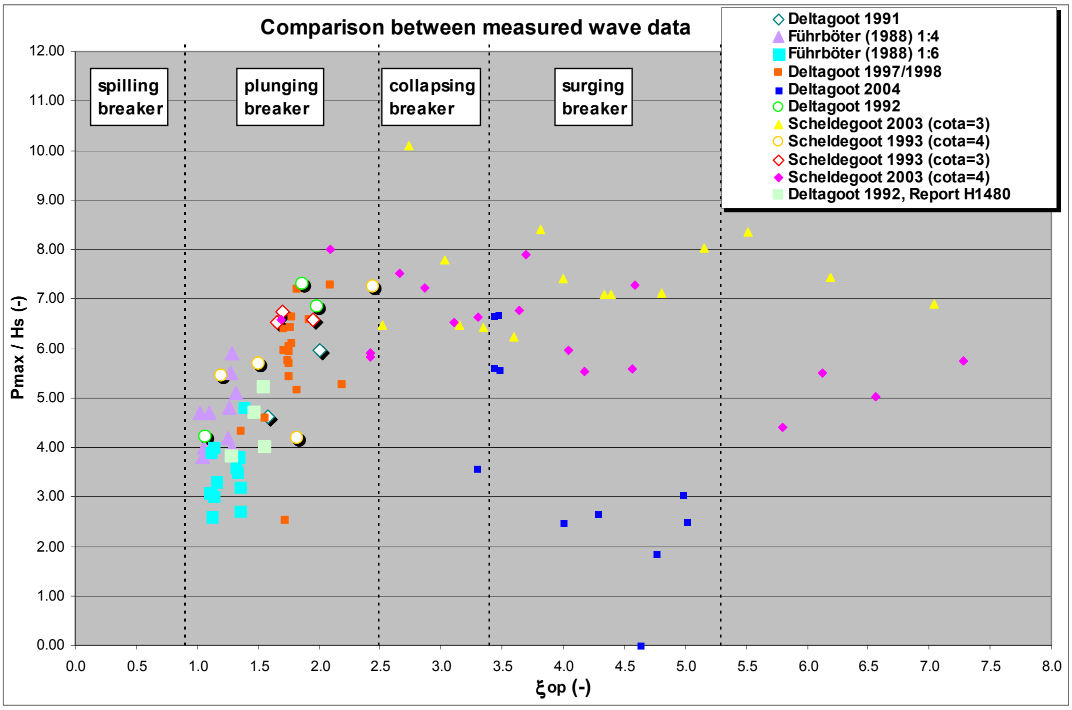






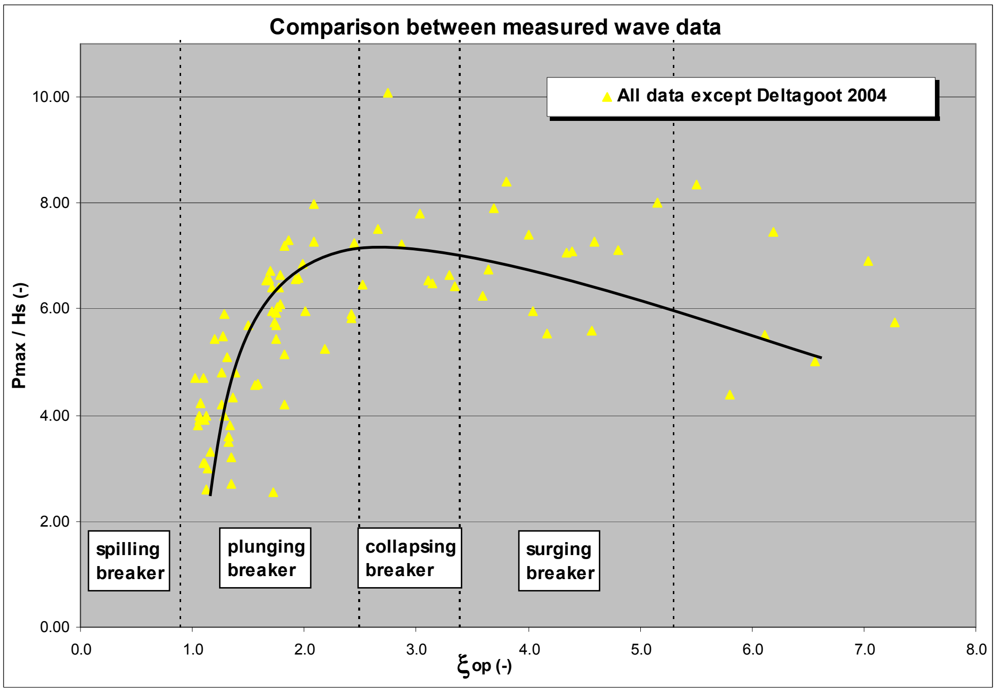
In this picture the values of p_{max}, belonging to H_s, Klein Breteler found are compared with the rule of thumb of Sparboom (1991). The data of Deltagoot 2004 are not showing the same trend as the data of the Scheldegoot 2003 experiment. This probably because the Scheldegoot experiment is a scaled experiment. Overall it looks like the rule of thumb of Sparboom (1991) gives higher maximum impact pressures and for higher breaker parameters (ξ_{op}>3.0) the differences are big.

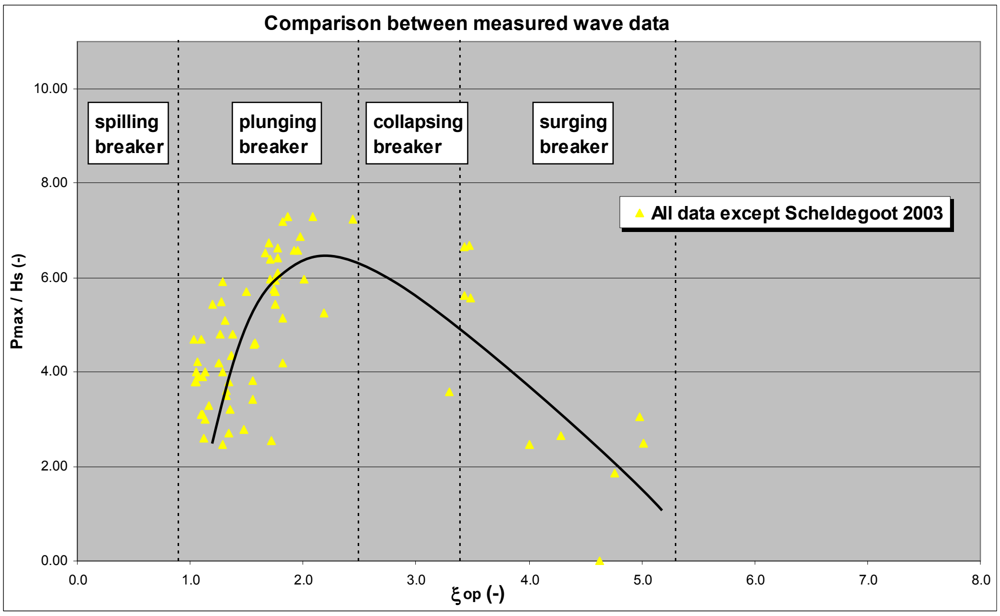
In the next picture the p_{max} value is made dimensionless by dividing it by the significant wave height. Only the $P_{max.max}$ values of the: "Führböter (1988)" data (see picture) are divided by H_m (the mean wave height) because no significant wave height is given and also the breaker parameter for this data is based on the mean wave height Hm.



The data: "Führböter (1988)" is based on regular waves whereas the wave conditions belonging to the data of Klein Breteler are as stated in the tables of chapter nine. In the experiments Klein Breteler analyzed, wave spectra are used. This can be an explanation for the lower values of Führböter (1988).

Also in this picture the data of the Deltagoot 2004 experiment is showing another trend then the data of the Scheldegoot 2003 experiment. In the next pictures a trend line is plotted, just to give a global idea of the relation between the breaker parameter and the maximum impact pressure normalized with the significant wave height.

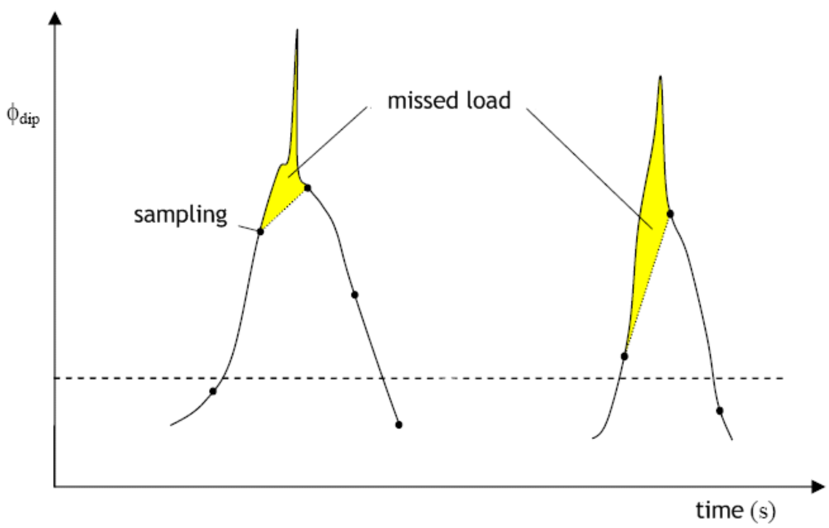




A physical explanation for this trend is the change of the wave characteristics on the slope by changing breaker parameter. With a breaker parameter higher then five almost no wave impacts are expected. Also the influence of the backwash layer is important. With a decreasing slope angle the backwash layer will increase. This results in lower p_{max} values. Furthermore, the p_{max} value is also dependent on the wave steepness which is also part of the breaker parameter (see section 12.5).

12.4 Influence of the sampling rate

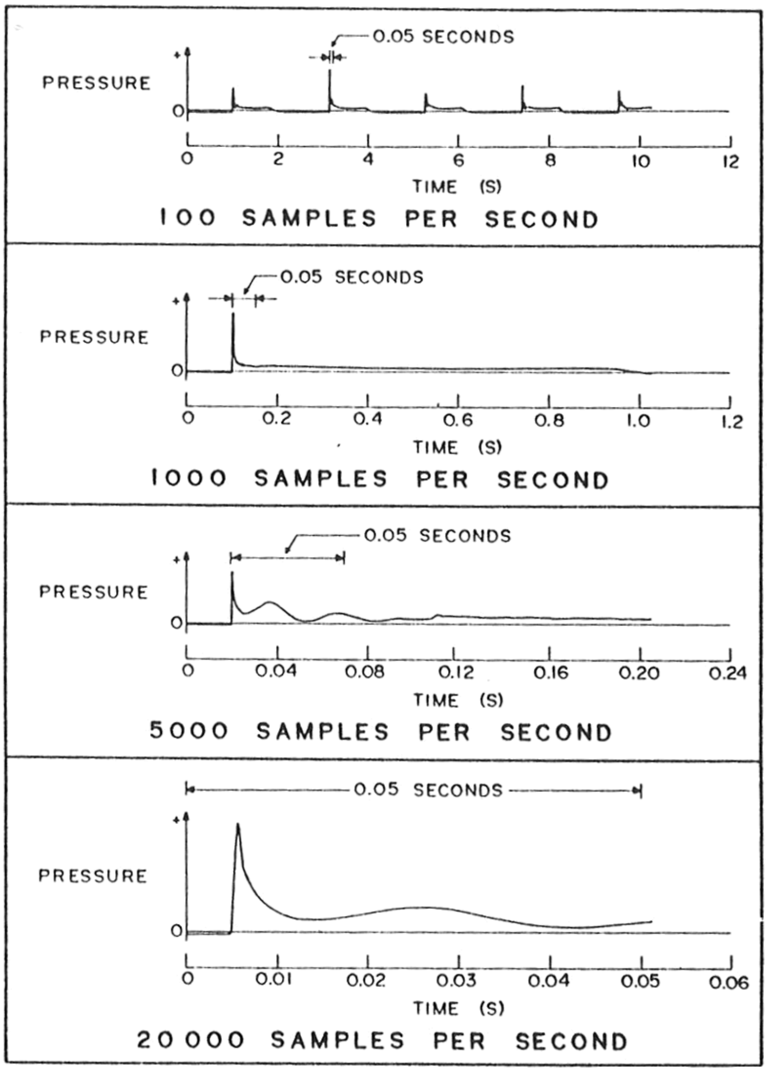
The frequency of the sampling rate determines whether the pressure peaks with duration of some milliseconds are recorded. According to Führböter and Sparboom (1988) the impact pressure time can be 10 till 60 milliseconds. This means very high sampling frequencies are needed to record the exact shape of the peak.



Van Vledder (1990) concluded in his literature survey:

- for the analysis of impact pressures, high speed pressure recording devices are needed.

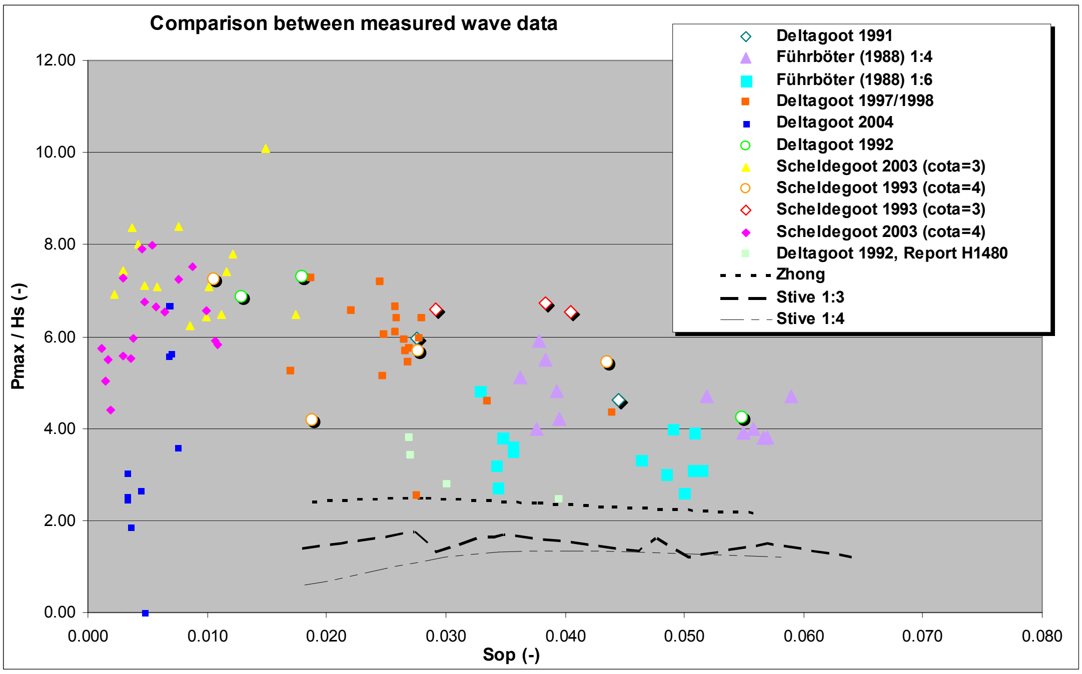
With very high sampling rate one measures the highest pressure peak but this peak will not affect the revetment due to inertia of the revetment.



The sampling rates of all the data is not analyzed, only a global comparison is sought after. Klein Breteler (2006) analyzed the maximum error because of the sampling rates for some experiments. The maximum error for the p_{max} data according to Klein Breteler is about 20%.

12.5 Dependency on wave steepness

The data of Klein Breteler (2006) is also used to show dependency on the wave steepness.



This picture confirms the trend investigated by Stive in 1984 and Zhong in 1985. Both researchers found, after a certain maximum, decreasing p_{max} values by increasing wave steepness (see chapter eight).

Also van Vledder (1990) gave this conclusion:

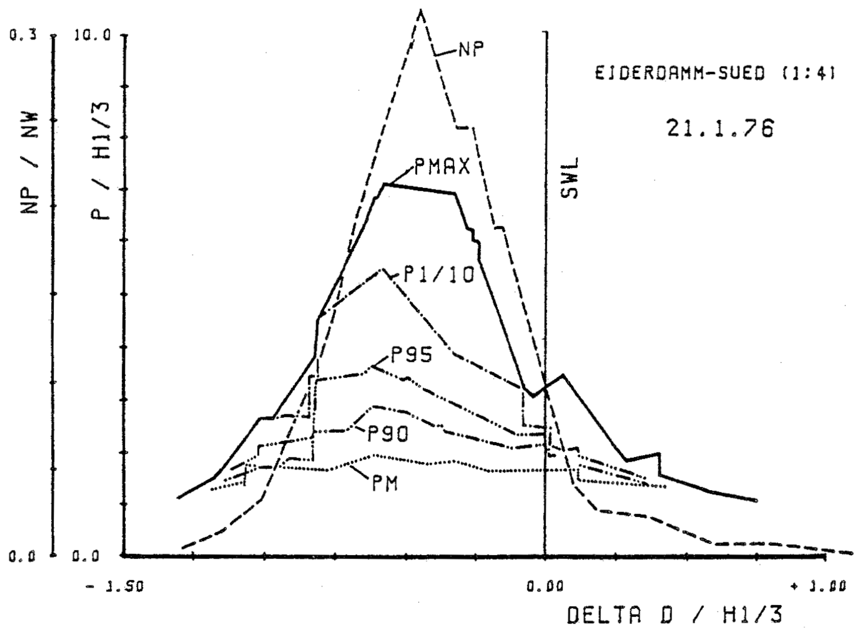
- the ratio $P_{max.max}/\rho_w gH$ decreases with increasing wave steepness after a certain maximum.

12.6 Location of the maximum impact pressure

The location of the maximum impact pressure is also sought after by many researchers. Stive, Grüne and Führböter had about the same results. Van Vledder (1990) concluded:

- most of the maximum impact pressures occur at about a distance half the incident wave height relative to still water level.

Grüne (1988) presented his findings for the Eiderdam measurements. On the horizontal axis the distance to the still water level delta d is normalized by the significant wave height. On the vertical axis NP/NW gives the ratio op peak pressures to the number of individual waves (related frequency of occurence).



Klein Breteler (2007) used a $x_{\phi max}$, the horizontal distance from where the still water line crosses the slope till the location with the highest pressure height. Klein Breteler (2007) concluded:

- location of wave impacts: $\frac{x_{\phi max} \tan\alpha}{H_s} = \min\{0.45\xi_{op} - 0.3\ ;\ 1.7\} \qquad \text{for} \quad 1 \le \xi_{op} < 6$

12.7 Width of the impact

The width of the impact is the area of impact measured in direction of the slope. Stive proposes a width of impact area of 10% of the incident wave height. In this area the maximum impact pressures will occur. Where the impact pressure is 80% of the maximum impact pressure the width will be 40% of the incident wave height and an impact pressure with a height of 50% of the maximum wave height will have an area of 80% of the incident wave height.

The program GOLFKLAP relates the width of the impact with the wave height by a Rayleigh distribution. The width varies between 0.5H and 1.5H.

According to Klein Breteler (2007), there is no relation between the width of the impact and the pressure height of the wave impact. However the width of the impact is dependent on the breaker parameter with the following relation:

$$\frac{B_{klap50\%2\%}}{H_s} = 0.96 - 0.11\xi_{op} \qquad \text{for} \quad 1 \le \xi_{op} < 5.5$$

With $B_{klap50\%2\%}$ as the width halfway the pressure height with 2% probability of exceedance (see chapter 9).

It can be concluded that nothing is known about the length of the impact, measured parallel to the slope.

12.8 Number of impacts

Van Vledder recommends to do more research in the topic of how many waves do really break and cause a wave impact. Grüne (1988) gives a ratio of peak pressures to the number of individual waves. Grüne (1988) related this ratio to the location of the maximum impact pressure (see graph section 11.6). Values for the ratio are about 0.3 for the maximum impact pressures. Also Klein Breteler (2007) gives ratios between the number of impacts and number of waves for his data. This ratio N_{impact}/N_{total} varies between 0 and 0.78 depending on the breaker parameter. The trend line for the scaled experiment (Scheldeflume) is:

$$\frac{N_{impact}}{N_{total}} = 1 - \frac{0.057}{\sqrt{s_{op}}}; \qquad \text{for} \quad 0.004 \leq s_{op} < 0.05$$

The guideline for asphalt in hydraulic engineering (TAW 1984) states that the revetment has to withstand the wave impacts of 10% of the incoming waves.

12.9 Summary

- the probability distribution of the maximum wave impact in time can be described by a log-normal distribution
- the maximum impact pressure is dependent on the slope and wave steepness; so on the breaker parameter
- the sampling rate has a big influence on measuring the peak of the impact pressures
- most wave impacts occur around half the incident wave height below SWL
- the width of the impact area is dependent on the breaker parameter and the incident wave height. There is no relation with the impact pressure
- for the number of wave impact some ratios to the total number of wave are given. Also a relation with the wave steepness is given.

12.10 Recommendations

As is summarized in this literature review a lot of research into wave impacts has been done. However some questions still remain. In this section some recommendations for further research will be done.

Nowadays with modern computers very high sampling rates for the measurements of wave impacts can be used. However it is not feasible to use the highest possible sampling rate because the high small peak will not affect the revetment anymore. Somewhere there is an optimum sampling rate.

- Research into which sampling rate is the most optimum to measure impact pressures on an asphalt revetment.

In the manual of GOLFKLAP the probability density function of the impact factor is given. Also the discrete impact factors are given in a graph. It is tried to reproduce these impact factors and the results are not totally in agreement with each other. Although the differences are small it is recommended to redefine the impact factors.

- Redefine the impact factors for the program GOLFKLAP.

It looks like the maximum impact pressures are dependent on the breaker parameter. However, the maximum impact pressures for the whole range of the breaker parameter are not investigated. More research is needed for the breaker parameter range of; $2<\xi_{op}<3.5$.

- Full scale measurements of impact pressures of breaking waves with breaker parameter; $2<\xi_{op}<3.5$

Also the number of impacts in comparison with the number of waves are probably dependent on the breaker parameter as is shown by Klein Breteler. The question remains which ratio can by used in real sea state conditions.

- Research into the number of wave impacts in real sea state conditions during a storm.

When more knowledge is obtained about the relation between the breaker parameter with the maximum impact pressures and the number of wave impact during a storm, the breaker parameter can be implemented into the program GOLFKLAP. When the breaker parameter is implemented also the dependency of the maximum impact pressure in the wave steepness is taken into account so also the wave period is of more importance.

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