Stability of stones in the surf zone

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Preface

This report presents the results of a study, performed at the Delft University of Technology, Faculty of Civil Engineering, to accomplish the requirements for the degree of Master of Science.

First I would like to thank the members of my thesis committee: Prof. ir. K. d'Angremond, Dr. ir. H.L. Fontijn and ir. G.J. Schiereck. A special thanks is owed to Dr. ir. H.L. Fontijn and ir. G.J. Schiereck for their contributions and advises during my research.

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Edwin Ottenheim
Delft, August 1997
Summary

Much research has been done on the stability of stones in breaking waves, but up to now, most of these studies were based on experiments with slopes varying from 1:1 to 1:7. The stability of stones on mild slopes, slopes not steeper than 1:10, has not yet been researched very extensively. Applications of mild slopes in practice are for example landfalls of oil pipelines and outfalls of sewage systems. The objective of this study is to improve the theoretical knowledge of the stability of stones on mild slopes in the surf zone by researching the flow in breaking waves.

The stability relations for stones on mild slopes established so far, followed the trend of experimental results quite well in a qualitative way, but the difference in stability for spilling breakers and plunging breakers was predicted too small by the relations. Probably the main reason of this imperfection is the influence of the plunging jet in a plunging breaker. Therefore, the processes which take place in plunging breakers are studied. From a study by Basco (1985) it was concluded that processes in spilling and plunging breakers are similar, albeit that the vortex systems in plunging breakers are of a much larger scale.

Experiments were carried out in the large wave flume of the Laboratory of Fluid Mechanics for a better understanding of the stability of stones on a slope subjected to wave attack. The model structure consisted of a 1:10 impermeable slope, on which a layer of stones ($D_{50} = 1.21$ cm) was laid. Only regular waves were used, because these wave are more suitable for researching the flow in a particular wave. For three waves with different wave steepnesses, incipient motion of the stones was determined. Subsequently, in the breaking regions of these waves, velocity measurements were carried out by means of LDV and video recordings. From the damage experiments it was concluded that maximum damage was located at about $h/H_o = 0.6$ and that the direction of displacement of the stones depends on the breaker parameter. Furthermore, the stone displacement in upslope direction seemed to be caused by the plunging jets of the breaking waves. The velocities in the plunging jet were equivalent for the three different waves, which is in line with the fact that these waves cause incipient motion.

The plunging jets of the breaking waves cause incipient motion of the stones. Up to now no theories were available for the stone stability in plunging jets. Therefore, an attempt was made to model the stone stability in a plunging jet. Two different models were considered, which both schematize the plunging jet on a stone as static forces on a single cubical stone. From the modelling it was concluded that the results deviate from experimental results. The missing of the turbulent fluctuations of the jet and to a less extent the dynamic characteristics of the stone stability were probably the main reasons of this deviation. Nevertheless, the modelling can improve the theoretical understanding of the stone stability in plunging jets. The numerical results of the stability relation by Izbash for uniform flow are close to the experimental results. Therefore, it seems that stone stability in a plunging jet is not as unfavourable as expected, compared to stone stability in uniform flow. The resulting stability equations for the stone stability in plunging breakers is in conformance with existing relations.
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LIST OF SYMBOLS

REFERENCES

APPENDIX I RESULTS OF THE DAMAGE EXPERIMENTS

APPENDIX II WATER LEVEL AND VELOCITY MEASUREMENTS

APPENDIX III SENSITIVITY ANALYSIS OF THE MODELS

APPENDIX IV WAVE GENERATION
1 Introduction

1.1 General

Nowadays a lot of bulk transport is carried out through pipelines, for example oil pipelines. Often these pipelines run from a location in sea to a location on land and therefore have to cross coasts. Pipelines on the sea bottom are usually protected to prevent possible damage caused by vessel anchors or erosion by currents. On the transition from sea to land, mostly a beach, also protection is needed against wave attack. The protection of oil pipelines in the surf zone is an example of what is called outfall or landfall structure protections. Other applications are for example sewage systems.

This kind of protections are often realized by laying the pipeline in a dredged trench and covering it with a stone protection consisting of a filter layer and a top layer, see Figure 1.1.

![Diagram of pipeline protection](image)

*Figure 1.1: Construction of outfall protection*

1.2 Background of research

In the past, much research has been done on the stability of stones on slopes under breaking-wave conditions. However, most of these studies were based on experiments with slopes varying from 1:1 to 1:7. The most important applications of this kind of slopes are breakwaters or dike revetments. Slopes of beaches, on the other hand, are mostly not steeper than 1:10.

Schiereck et al., 1994, established a provisional design rule for armour layers on a mild slope under wave attack, based on theoretical considerations and experiments carried out by Sistemans (1993) and Grote (1994). Only for non-breaking waves the
results were satisfying. For stone stability in breaking waves only a conservative design rule based on the experiments was found. The theoretical concept used, was not appropriate for breaking waves.

In 1996 Schiereck et al. made another attempt to derive a theoretical relation for the stability of stones in breaking waves. New experiments were carried out too by Ye (1996) with different slope angles and stone properties, as, up to now, the available experimental results were not sufficient to give reliable results for a greater range of slope and wave properties. In the theoretical approach the wave load was composed of the orbital motion and turbulent velocities due to breaking. Also a relation for the wave height distribution was involved, because in nature waves are irregular. For the stone stability the relation of Rance & Warren (1968) for stones in oscillatory flow was used.

The theoretical results followed the trends of the experimental results quite well in a qualitative way but the computed relation between the breaker parameter and the stone stability was quantitatively far from sufficient. The difference in stability with increasing breaker parameter was too small. Probably the main reason was the lacking of the vertical velocity caused by the jet in a plunging breaker. The turbulence model used could also be a weak factor.

The experimental results were in line with the experiments carried out by Sistermans and Grote and also with the Van der Meer relation (1988) for stability of stones on steep slopes.

1.3 Objective

The first objective of this study is to measure the water particle velocity in breaking regular waves in order to get a better understanding of the processes in breaking waves. These measurements will be carried out for three different wave conditions which cause incipient motion of the stones.

The second objective is to improve the theoretical knowledge of the stone stability on slopes under breaking-wave conditions by using the experimental results. Beside the orbital motion and the turbulence generated by wave breaking also an attempt will be made to include the vertical velocity of a plunging jet in the theoretical approach.

1.4 Outline of report

Chapter 2 discusses the processes taking place in a breaking wave and the results by previous researchers. Subsequently, chapter 3 presents an overview of the most important parameters used in this thesis. In the chapters 4 through 6 the wave flume experiments are discussed. Chapter 4 describes the model arrangement, chapter 5 the experiments carried out and chapter 6 presents an analysis of the results. Chapter 7 describes theoretical approaches of the processes taking place in the outer breaking region of a plunging breaker, inclusive a model of the stone stability in plunging jets. Finally, in chapter 8 conclusions will be drawn and recommendations for a continuation of this study are mentioned.
2 Literature study

A lot of research has been done on wave breaking and the stability of stone under breaking waves. In this chapter at first an overview is given of breaking wave characteristics. Subsequently the results of previous studies on the stability of rock on mild slopes are discussed.

2.1 Wave characteristics in the surf zone

2.1.1 A qualitative description of wave breaking

When waves approach the shore from deep water, gradually a change in the wave profile takes place. The wave celerity decreases, when the water becomes shallower. According to the linear wave theory the wave celerity in shallow water is:

\[ c = \sqrt{gh} \]  

(2.1)

The linear wave theory describes the oscillatory flow due to the orbital motion at the bottom in shallow water by the following relation:

\[ u_0 = \omega \frac{H}{2} \cdot \frac{1}{\sinh(kh)} \cdot \cos(\omega t) \]  

(2.2)

Wave breaking occurs, when the water particle velocity induced by the orbital motion approaches the value of the wave celerity. Wave breaking can be defined as the transformation of particle motion from irrotational to rotational, generating vorticity and turbulence. The breaking region can be divided into two parts: the outer breaking region and the inner breaking-region. In the outer breaking-region a rapid transition of the wave shape takes place. As the wave propagates further the wave develops to a uniform shape which is similar to a bore or hydraulic jump. This is called the inner breaking-region.

The processes in the breaking region are dependent on the breaker type, which is determined by the wave steepness as well as by the slope angle. A distinction can be made among four types of wave breaking based on visual observations. Proceeding from breaking to non-breaking the following breaker types can be distinguished:

spilling \(\rightarrow\) plunging \(\rightarrow\) collapsing \(\rightarrow\) surging

A spilling breaker is characterized by gradual dissipation of wave energy after breaking. In the outer breaking-region, just after the beginning of wave breaking, turbulence is mainly confined to the water surface and spreads gradually downwards as the wave propagates shorewards. A slow, gradual transition from irrotational to rotational motion takes place. This results into a wide outer breaking-region, because
the turbulence production is initially small, gradually increases and requires much time to reach equilibrium at the start of the inner breaking-region. Spilling breakers have a gradual change in wave height in the outer as well as the inner breaking region.

In a **plunging breaker** the energy dissipation is much more concentrated within a certain area and can be characterized by the formation of a clearly visible overturning jet. At breaking the front face of the wave becomes vertical and the crest curls over, forming an inner air-core and plunges into the water ahead with considerable force. A sudden, violent transition from irrotational to rotational motion takes place. In spite of the seemingly chaotic state, it is possible to identify certain flow patterns. Large-scale vortices are formed which gradually decay into smaller scales and dissipate their energy into turbulence. The turbulence generated by vortices is not confined to the surface, but spreads downwards and does not follow the propagating wave crest. In the inner breaking-region the vortices have disappeared and the waves show the same development as a spilling breaker.

Also in a **collapsing breaker** a plunging jet occurs. However, this breaking-wave type deviates from a plunging breaker, because the jet acts directly on the slope without a body of water in between.

The **surging breaker** can be seen as the transition between breaking and non-breaking waves, although it can be questioned whether this is a breaking or a standing wave. The only difference between surging and non-breaking waves is the difference in run-up on a slope.

![Diagram](image)

**Figure 2.1:** Similarity of fluid motion in spilling and plunging breakers (Basco, 1985)

In this report only spilling and plunging breakers will be extensively discussed, because only these types occur on mild slopes. Apparently great differences exist
between these wave types, but both spilling and plunging breakers generate similar patterns of motion and vortex systems. The only difference is the strength of the vortex systems, which is of a much larger scale in case of a plunging breaker. A schematic representation of this concept is shown in Figure 2.1.

Basco (1985) investigated the wave breaking phenomenon in the outer breaking region by means of visual observations. He schematized the sequence of wave breaking events, which is shown in Figure 2.2 for a strong plunging breaker. These sequence also holds for a spilling breaker but to a much smaller scale.

Figure 2.2: Schematic sequence of breaking wave events (Basco, 1985)
Basco accompanies this figure with the following text:

1. Wave breaking starts.
2. The overturning jet translates to strike the oncoming trough at the plunge point.
3. The impulse of the jet creates an initial surface disturbance (splash).
4. The impulse and weight of the jet forces it to penetrate in the oncoming trough. Backward trough flow deflects the submerged jet further downward and backward against the original wave direction. The tendency to initiate a vortex is created by the sustained shoreward particle motion at the crest of the overturning wave and the seaward trough motion combined with the overturning jet. Figure 2.3 shows this plunger vortex.
5. The trapped air-core compresses as the "green-water" wall under the crest translates horizontally (relative to the stationary observer) with air bubbles forming within the water.
6. The initially disturbed surface (splash) falls forward and forms a surface roller much like the roller of a hydraulic jump. Trough fluid moves backward while surface particles in the roller move forward with the propagating wave.
7. The plunger vortex translates horizontally and pushes on the oncoming trough to create a secondary wave disturbance and increases the size and strength of the surface roller.
8. The toe of the surface roller slides down the front face (trough) of the oncoming wave to its equilibrium position. The surface roller grows in size and generates more vorticity as a result.
9. Translation of the plunger vortex relative to a stationary observer slows down and stops as it rotates, enlarges and drifts downward. But the secondary wave disturbance it generated, continues to propagate forward.
10. The asymptotic end of the outer breaking-region is reached when the surface roller reaches its stable equilibrium state and the horizontal translation of the plunger vortex ceases in its generation of the secondary wave disturbance. This is also the beginning of the inner breaking-region.

Figure 2.3: Schematic plunging breaker (Basco, 1985)
In the outer breaking-region turbulence is produced, advected, diffused and dissipated such that a local non-equilibrium state of turbulent kinetic energy exists. This means that at any place the rate of turbulence production is not equal to the rate of dissipation. In Figure 2.1 it was shown that two primary generation zones exist, one beneath the surface roller and the other at the outer radii of the plunger vortex. Plunger vortex turbulence initially dominates over the water column for strong plunging breakers, but fades away at the point where the horizontal translation ceases. Spilling breakers are dominated by surface roller generated turbulence. The rate of this turbulence is small as compared to the initial turbulence of plunging breakers and requires more time to reach equilibrium. This is the reason why the outer breaking region for spilling breakers is wider than for plunging breakers.

### 2.1.2 Dimensionless breaker characteristics

The breaker parameter $\xi$ can be used to describe the breaker type. The transition from one breaker type to another is gradual; however, an indication of the boundaries of each breaker type can be given:

- spilling if $\xi < 0.5$
- plunging if $0.5 < \xi < 3.0$
- collapsing or surging if $\xi > 3.0$

![Diagram showing different breaker types and their corresponding $\xi$ values.](image)

*Figure 2.4: Breaker types as function of $\xi$ (Battjes, 1974)*
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Not only the type of wave breaking varies with $\xi$, but also the location of breaking and the wave height at breaking can be expressed by this parameter. Kaminsky and Kraus (1993) tried to predict incipient, depth-limited breaking of individual waves. Their results were based on a lot of laboratory experiments with slopes varying from 1:110 to 1:5. They investigated the variation of the following breaker indices under different wave conditions:

\[
g_b = \frac{H_b}{h_b} \quad (2.3)
\]

\[
\Omega_b = \frac{H_b}{H_0} \quad (2.4)
\]

\[
\beta_b = \frac{h_b}{H_0} \quad (2.5)
\]

where $\gamma_b$ is called the breaker height-to-depth index, $\Omega_b$ the breaker height index and $\beta_b$ the breaker depth index. The subscript $b$ denotes the breaking condition and the subscript 0 the deep-water condition. These three indices are algebraically related as $\beta_b = \Omega_b/\gamma_b$.

For typical field beach slopes (1:30 to 1:80) $\gamma_b$ has an average of 0.78, which is equal to the value predicted by the solitary wave theory. For steeper slopes this value is much larger and can be calculated by:

\[
\gamma_b = 120 \xi^{0.27} \quad (2.6)
\]

According to Kaminsky and Kraus (1993), $\Omega_b$ is only dependent on the deep-water wave steepness and not on the slope angle:

\[
\Omega_b = 0.46 \left( \frac{H_0}{L_0} \right)^{-0.28} \quad (2.7)
\]

The breaker depth index varies with the slope angle as well as with the wave steepness:

\[
\beta_b = 0.30 \tan \alpha^{-0.25} \left( \frac{H_0}{L_0} \right)^{-0.23} \quad (2.8)
\]

This relation should comply with $\beta_b = \Omega_b/\gamma_b$, but does not, because of the peculiar way the relations are determined (see Kaminsky and Kraus, 1993). These three equations explain about 90% of the data variability of the experiments. In paragraph 6.3 these relations are plotted with values from the experiments inserted. The relation $\beta_b = \Omega_b/\gamma_b$ is also plotted to show the difference from equation 2.8.
The amount of reflected wave energy is also dependent on the type of wave breaking. It varies from negligible reflection for spilling breakers to complete reflection in case of no breaking. Battjes (1974) made an attempt to relate the reflection to the breaker parameter. He found the following relation, which is based on experiments (see Figure 2.5):

\[ r = \begin{cases} 
0.1 \xi^2 & \text{if } r \text{ is less than 1} \\
1 & \text{otherwise}
\end{cases} \quad (2.9) \]

where \( r \) is the ratio between the reflected wave and the incident wave. However, he concluded that gentler slopes give less reflection than steeper slopes with the same value of \( \xi \). Therefore \( r \) is in fact not only dependent on the breaker parameter but also on the slope angle.

![Figure 2.5: The reflection coefficient as function of \( \xi \) (Battjes, 1974)](image)

### 2.2 Previous research on the stability of stones in the surf zone

#### 2.2.1 Regular waves experiments

Both Sistermans (1993) and Grote (1994) performed experiments with regular waves on a 1:25 slope. All breaking waves were of the spilling type. They found that the location of maximum wave attack was at \( h/H_o \approx 1 \) and that the total damage in downslope direction was higher than the total damage in upslope direction. For values \( h/H > 1 \) the stability of the stones could be described by formulae derived from stability of stone on a horizontal bottom. For values \( h/H < 1 \) these formulae could not be applied anymore. Grote (1994) concluded that beyond the location \( h/H = 1 \) the plunging or spilling effects of wave breaking explained the deviation of these
formulae. In this region the impingement of a plunging water jet and the turbulent velocity fluctuations probably cause higher velocities at the bottom than the calculated values.

Local damage levels are most appropriate for design purposes, because locally the damage to a structure can reach an unacceptable value, whereas the total damage is not alarming.

Sistermans made the results of his regular waves experiments dimensionless by transforming the damage levels to damage percentage levels and by using the dimensionless parameters $\xi$ and $H/\Delta D_{n.50}$. The resulting best-fit curve was approximated by the following formula:

$$\frac{H}{\Delta D_{n.50}} = 133 \cdot \xi^{-1.34} \cdot S_{\%\text{max}}^{0.17}$$  \hspace{1cm} (2.10)

In Figure 2.6 this relation is plotted for three damage levels; on the horizontal axis the range of $\xi$ matches with the values used in his experiments.

![Graph](image)

**Figure 2.6:** Relation between $\xi$ and $H/\Delta D_{n.50}$ for three damage levels (Sistermans, 1993)

The results of the experiments were in line with the Van der Meer relation (1988) for irregular waves: higher values of $\xi$ require heavier stones or smaller waves. One should be careful by applying this relation to other slopes than 1:25, because the relation was derived from experiments without varying the slope angle.

### 2.2.2 Irregular waves experiments

Regular waves are convenient to study the breaking process of a particular wave, but in a natural situation only irregular waves occur. Therefore irregular wave experiments were carried out by Sistermans (1993) and Grote (1994) as well as by Ye (1996). Sistermans (1993) and Grote (1994) only performed experiments on a slope with angle 1:25, but Ye (1996) also used a slope angle 1:10. Comparing the
damage distribution for irregular waves with that for regular waves led to great differences. The damage of irregular waves was less concentrated, because not each wave breaks at the same location on the slope. Further, the maximum damage for irregular waves was located in the run-up region. Great differences in run-up velocities are responsible for this damage.

Schiereck and Fontijn (1996) compared the experimental results by Ye (1996) with the empirical Van der Meer formula for plunging breakers (see Figure 2.7). They concluded that the trend was the same for both situations.

![Figure 2.7: Results of Ye compared with Van der Meer formula (Schiereck and Fontijn, 1996)](image)

### 2.2.3 Theoretical approach

Experimental results that can be explained by theory are generally better understood and decrease the chance of misuse of empirical relations. Schiereck et al. (1994), made an attempt to derive theoretical relations for the stability of stone on a mild slope under wave attack. Orbital bottom velocities were calculated with the linear wave theory. To determine the stability of stone, the approaches of Jonsson (1966)/Sleath (1978) and Rance & Warren (1968) were used. These were derived for the stability of stone on horizontal bottoms. For non-breaking waves the results were reasonably satisfying, because the situation for non-breaking waves is similar to that for oscillatory flow.

This approach proved to be unsuitable for breaking waves, because the breaking characteristics for various breaking wave types were not taken into account.

Schiereck et al. (1996), made an attempt to connect experimental results with the physical background of forces due to breaking waves. They tried to model the
Stability of stones on a slope by determining the relation between load and strength. The strength is usually described by the effective weight of the stone and friction factors. The load consists of the forces induced by breaking waves. These forces are not fully understood yet. The orbital and turbulent velocities are responsible for these forces, but the question is how to model them. Another complicating factor is the irregularity of waves in a natural situation. The orbital velocity was modelled by the linear wave theory. The turbulent velocities were modelled by making use of the approach by Battjes (1975). He coupled the production of turbulent energy to the dissipation of wave energy due to breaking:

\[ q \approx \left( \frac{D_B}{\rho_w} \right)^{\frac{1}{3}} \]  

(2.11)

where \( q \) is the turbulent velocity scale (\( q^2 = u_i u_j \), where \( u_i \) and \( u_j \) are turbulent velocity components). The dissipation of turbulent energy was derived from the analogy between a bore and a breaking wave (Battjes and Janssen, 1978):

\[ D = \frac{1}{4} \frac{Q_B}{T_p} \rho g H_m^2 \]  

(2.12)

where \( Q_B \) is the fraction of the waves that are broken, derived from:

\[ \frac{1 - Q_B}{\ln Q_B} = \left( \frac{H_{rms}}{H_m} \right)^2 \]  

(2.13)

where \( H_m \) is the maximum wave height by Miche (1944) given by:

\[ H_m = 0.88 \cdot k^{-1} \cdot \tanh(kh) \]  

(2.14)

Subsequently, the necessary stone diameter was computed with equations derived from, consecutively, the approaches by Rance & Warren and by Jonsson / Sleath. These equations read:

Rance & Warren:  
\[ D_{n50} = \frac{0.84 \cdot 2.56 \cdot (\hat{u}_b + F \cdot q)^{2.5} \cdot \sin \phi}{T \cdot (\Delta g)^{1.5} \cdot \sin(\phi - \alpha)} \]  

(2.15)

Jonsson / Sleath:  
\[ D_{n50} = \frac{0.84 \cdot (f_w \cdot \hat{u}_b^2 + F \cdot q^2)}{2 \cdot 0.956 \Delta g} \]  

(2.16)

where:  
\( \hat{u}_b \) = maximum orbital velocity at the bottom  
\( f_w \) = friction coefficient  
\( F \) = calibration factor  
\( \phi \) = angle of repose of stones
The computational results using the concept of Jonsson (1966)/ Sleath (1978) proved to give the best agreement with the experimental results. However, the difference in stability for low and high values of the breaker parameter was computed too small by both methods. An explanation for this could be the influence of the breaker type on the velocity at the bottom. A larger $\xi$ leads to a more plunging type of breaker, where due to the plunging jet high vertical velocities can occur. This plunging jet was not described in the models used during this study. Another explanation could be the turbulence model used in regarding the stability of a single stone.
3 Governing parameters

In the previous chapter a lot of parameters arose from the discussion of wave breaking and previous studies. These parameters can be divided into hydraulic and structural parameters. Both kinds of parameters are of importance for the stability of stones on mild slopes under breaking waves conditions. The hydraulic parameters consist of the water depth and wave properties such as the wave height and wave period. The structural parameters consist of slope properties such as slope angle, stone size and stone mass density. Further it is important to define the damage caused by waves independently of the dimensions of the flume.

3.1 Hydraulic parameters

3.1.1 Wave height

Waves cause the destabilizing forces exerted on stones by the orbital motion and turbulent breaking-processes. The wave height, $H$, directly influences these forces and is therefore one of the most important parameters. $H$ also determines the location of wave breaking.

In the experiments conducted during this study only regular waves have been used, which means that all wave heights have the same value. In natural situations only irregular waves exist, which are described by a spectrum. Usually $H$ is characterized by the significant wave height $H_s$, defined as the average of the highest one-third wave heights measured during a certain period.

The advantage of regular waves for a theoretical study is the possibility of phase averaging of velocity measurements, because the wave height and period are constant. Another advantage is that displaced stones in damage experiments can directly be ascribed to a certain wave. The wave height in deep water is usually noted as $H_o$, and at breaking as $H_b$. The wave height at the toe of the slope was used as reference wave height during the experiments and is written as $H_{toe}$.

3.1.2 Wave period

The wave period, $T$, is another important parameter. $T$ directly determines the wave length, $L$, and the wave celerity $c$, in deep water. These quantities are related as (deep water):

$$c = \frac{g T}{2\pi} \quad (3.1)$$
STABILITY OF STONES IN THE SURF ZONE

\[
    L_o = c \cdot T : \quad L_o = \frac{gT^2}{2\pi} \tag{3.2}
\]

For regular waves \( T \) is constant. For irregular waves the wave motion consists of a large number of different wave components. The peak period \( T_p \) is often used to characterize the irregular wave period, which is defined as the period with frequency \( 1/T_p \) where the variance-density spectrum reaches its maximum.

### 3.1.3 Water depth

In combination with the wave height and wave length, the water depth determines the location of breaking of a wave. For example, a drop in water level will result into a shift of the location of most heavy wave attack into offshore direction. This is most obvious for structures with gentle slopes. The water depth where incipient wave breaking occurs is usually denoted as \( h_o \).

### 3.1.4 Storm duration

The damage to a slope is dependent on the time span during which waves are breaking on the slope. This is called the storm duration. Commonly the storm duration of experiments is determined by the number of waves, which has the advantage that no scaling from a model to a prototype is necessary.

Sistermans (1993) used for the experiments with regular waves a storm duration of 750 waves. He concluded that most damage to a loosely packed rock bed took place during the first 250 waves, but that the development of damage after 750 waves had not yet come to an end.

### 3.2 Structural parameters

#### 3.2.1 Rock size

The rock size is an important parameter to determine the stability of the stones. A larger stone leads to less damage under the same wave conditions. Usually the rock size, for smaller rock, is described by the median sieve diameter \( D_{50} \), which is the sieve diameter through which 50% of the total weight of a sample can pass. However, for the stone stability the rock weight is determinative. Therefore the nominal rock diameter, which is related to the weight of the rock, is more appropriate to describe the rock size. The nominal diameter represents the edge of a cube with the same mass as the rock. In formula form this is:

\[
    D_n = \left( \frac{M^3}{\rho_s} \right)^{1/3} \tag{3.3}
\]
where: \( D_n \) = nominal diameter
\( M \) = mass of the rock
\( \rho_s \) = mass density of the rock

The nominal diameter is related to the sieve diameter as:

\[
D_{n\,50} = \beta \cdot D_{50}
\]  \( (3.4) \)

The shape factor \( \beta \) varies from 0.8 for flat-shaped rock to 0.9 for round-shaped rock. Angular rock, the rock used in the experiments, is angularly shaped and has a shape-factor 0.84.

### 3.2.2 Grading

The stone size of a sample of natural stone is not the same for each stone. The stone size can be measured by sieving or weighing. The percentage of weight passing through a sieve with a certain diameter can be presented as a function of this diameter. This is called a sieve curve. The grading of a sample is usually indicated by the steepness of the sieve curve, which is the ratio \( D_{50}/D_{15} \). The influence of the grading on the stone stability is negligible according to Van der Meer (1988). However, this was only tested for steep slopes. The grading will be kept constant in this thesis, so the influence on the stability will not be investigated.

### 3.2.3 Mass densities

The mass density of a stone \( (\rho_s) \) determines the weight of a stone by multiplying it with its volume. Not all kinds of rock have the same mass density. It ranges from 2300 kg/m\(^3\) for limestone to 3000 kg/m\(^3\) for heavy granite. The stability of a submerged stone is also dependent on the mass density of water \( (\rho_w) \). The weight of rock under water can be calculated by:

\[
W = D_n^3 \cdot (\rho_s - \rho_w) \cdot g
\]  \( (3.5) \)

The mass density of water can be considered as a structural as well as a hydraulic parameter, because the load on a structure is dependent on \( \rho_w \) and the strength of a structure depends on the weight of rock under water.

### 3.2.4 Slope angle

The slope angle of a structure \( (\alpha) \) is an important factor for the stability of stones on the slope. The stability reaches a minimum when \( \alpha = \phi \), the natural angle of repose of the material, and increases with decreasing slope angles.

Indirectly, the slope angle has a great influence on the way waves break on the structure which has direct effect on the stone stability.
3.3 Dimensionless parameters

Dimensionless parameters are useful to compare results of different experiments and to scale models to prototypes as far as scale effects are absent. The parameters presented in this chapter can be combined to five frequently used dimensionless parameters in coastal engineering:

- The dimensionless water depth $h/H$:
  A wave entering from deep water finally breaks when $h/H$ drops to a certain value. At the location of breaking of a wave this ratio is often noted as:

  $$
  \gamma_b = \frac{H_b}{h_b}
  \tag{2.3}
  $$

  where $\gamma_b$ is called the breaker height-to-depth index. Another dimensionless parameter dependent on the water depth and the wave height is the breaker depth index:

  $$
  \beta_b = \frac{h_b}{H_0}
  \tag{2.5}
  $$

  A combination of these dimensionless parameters leads to:

  $$
  \Omega_b = \gamma_b \beta_b = \frac{H_b}{H_0}
  \tag{2.4}
  $$

  which is called the breaker height index.

- The wave steepness, $s$:
  This is the ratio between the wave height, $H$, and the deep-water wave length, $L_0$. It has great influence on the water motion under waves. With (3.2) the wave steepness can be written as:

  $$
  s = \frac{2\pi H}{gT^2}
  \tag{3.6}
  $$

  Often $s$ is obtained as a fictitious value, viz. as the local wave height related to the deep-water wave length.

- The breaker parameter, $\xi$:
  This parameter describes the way of wave breaking on a slope and is defined by Irribarren (1950) as:

  $$
  \xi = \frac{\tan \alpha}{s}
  \tag{3.7}
  $$
3 Governing Parameters

- The relative mass density of rock, $\Delta$:
The stability of submerged rock is related to this parameter. It can be formulated as:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \quad (3.8)$$

- The stability parameter:

$$\frac{H}{\Delta D_{n50}} \quad (3.9)$$

$H$ represents the destabilizing forces by wave attack and $\Delta D_{n50}$ the stabilizing forces.

3.4 Stability

Regarding armour protections on slopes, a division can be made between static stability and dynamic stability. A static stable structure is designed in such a way that normal wave conditions can not move the stones of an armour layer. Only design conditions can subject the stones to large wave forces, which are able to move or displace individual stones. Armour layers which are designed to be static stable will probably lead, after collapsing, to severe damage to the underlying structure. An outfall protection is an example of a statically stable structure.

Dynamic stability is defined by the possibility of forming a stable profile, which can differ a lot from the initial profile. The stable profile can be schematized by profile parameters. For more information about dynamic stability, see Van der Meer (1988).

3.5 Damage

Damage of statically stable structures is defined as the displacement of rock units and can be determined by two methods. The first is comparing the profile of a structure before and after a test and the second implies simply counting the displaced stones.

With the first method the problem is to measure the difference in profile, which is also called the erosion area. Especially for little damage the erosion area is difficult to define.

The second method can be carried out in the following way. Paint the stones and lay them in strips of different colours in the test area. Subsequently, count the stones which have been displaced to another strip. The disadvantage of this method is that the stones which move within a strip can not be counted and therefore the width of a strip affects the damage level. It is important to clearly define the level of damage,
because the number of stones displaced out of a strip depends on the width of the wave flume and the length of the strip. By relating the total number of displaced stones to the number of cubes with edges $D_{n50}$ that fit in the top layer of the test area, the damage level can be made independent of the strip dimensions.

Often it is desired to locate the position on the slope with maximum damage and to determine the maximum damage level. This can be achieved by calculating the local damage levels. The local damage level is defined by the number of stones displaced out of a strip, related to the number of stones with sides $D_{n50}$ that fit in the top layer of a strip. The local damage percentage, $S_{\% \text{ loc}}$, can now be written in formula form as:

$$S_{\% \text{ loc}} = \frac{n \cdot D_{n50}^2}{A}$$

(3.10)

where: $n$ = number of stones displaced [-]
$A$ = area of a strip [m$^2$]
4 Description of model experiments

Theory is important for the understanding of processes occurring in nature. Only after verifying theoretical relations with experiments, certainty is obtained about the validity of these relations. Experiments are also useful because measurements can produce new insights into the physical processes. Therefore model tests were carried out for a better understanding of the stability of stones on a slope under wave attack.

4.1 Test facility

The experiments were carried out in the large wave flume of the Laboratory of Fluid Mechanics of the Faculty of Civil Engineering. The dimensions of the flume are: length 40 meters, width 0.8 meters and height 1.05 meters. Waves can be generated by a piston-type wave board which can only make translational movements. The wave generator can produce regular as well as irregular waves. After starting the wave board, waves are reflected against the slope and propagate to the wave board again. Re-reflection of these waves can be avoided by a reflection compensation system. With this system also long standing waves are avoided.

4.2 Model set-up and material properties

A uniform slope (1:10) was built in the flume, consisting of a sand body covered with an impermeable cement layer (see Figure 4.1). A certain roughness was given to the surface of the slope by glueing gravel on the cement layer. Subsequently stones were laid on the slope with a layer thickness of three centimetres, about twice the stone diameter. In the section where the most severe damage was expected, the material was laid down in coloured strips with a band width of 25 cm. Only one type of stone was used during the experiments, having the following properties:

\[
\begin{align*}
D_{50} & = 1.44 \text{ cm} \\
D_{15} & = 1.25 \text{ cm} \\
D_{85} & = 1.64 \text{ cm} \\
D_{n50} & = 0.84 \times D_{50} = 1.21 \text{ cm} \\
D_{85}/D_{15} & = 1.3 \\
p_s & = 2950 \text{ kg/m}^3
\end{align*}
\]

The sieve curve of the test material is given in Figure 4.2.
4.3 Wave conditions

Only tests were carried out with regular waves. The control signal for the wave board was generated by the program STIR of the software package AUKE/pc of Delft Hydraulics. The measured wave height was not the same as the wave height imposed in the STIR file. Therefore the correct wave height had to be determined iteratively by adjusting the wave height in the STIR file and the value of the SPAN of the MTS wave board controller. More information about the wave generation is given in Appendix IV.
Experiments were done with wave steepnesses 1%, 3% and 5%. The duration of the experiments was chosen as 750 times the wave period, to be able to compare the results with those of Sistermans (1993) and Grote (1994).

4.4 Measuring equipment and calibration of instruments

4.4.1 Equipment

Wave height meters

Water surface elevations were measured by conductivity-type wave height meters. A wave height meter consists of two metal rods which measure the conductivity of the water body in between. A reference electrode at the underside of the gauge corrects the effects of conductivity fluctuations caused by, for example, temperature variations.

This type of wave height meter has some strong points:
- they are easy installed and moved to other locations,
- high frequencies can be measured,
- wave heights are directly measured and not derived from any wave theory.

Also some weak points exist:
- the conductivity of water containing air bubbles is unknown, so wave height measurements in the breaking region are probably not completely correct,
- when the immersion depth of a gauge is too small, water level variations are not measured correctly anymore.

Laser Doppler Anemometer

Velocities were measured by a two-component Laser Doppler Anemometer. The principle of the Laser-Doppler Velocimetry (LDV) is the Doppler frequency shift, caused by the motion of tiny particles in the water. From this frequency shift horizontal and vertical velocities can be determined.

Strong points of the two component LDV are:
- the accuracy of the measurements,
- no calibration is needed,
- the flow is not disturbed by any instrument,
- horizontal and vertical velocities are measured at the same time at a certain position.

There are also some weak points:
- the LDV instruments are vulnerable for damage,
- the disturbance of the signal by air bubbles is large, because the laser beams cross the whole width of the flume,
the glass of the flume side-walls has to be clean and smooth, which can be a problem when using cement and stones.

Video recordings

The use of Laser-Doppler Velocimetry is restricted to water with (almost) no air bubbles. Breaking waves, especially where the overturning jet plunges into the water, contain a lot of air bubbles; so, LDV is useless in this region. Therefore another method has to be used to measure velocities in the plunging jet. A primitive but useful method is filming particle motions in breaking waves. For the particles near-neutrally buoyant paraffin balls were used, stained with a red-fluorescent dye. The particle size of these balls used during the experiments varied from 3 to 6 mm. Ultraviolet tubes were placed close to the side wall of the flume to illuminate the particles. The flume was covered with black cloth to exclude the environmental light from illuminating the water. The result of this was that only the fluorescent balls were visible on the video recordings. The concentration of the particles was such that individual particles could easily be traced on the video recordings. The camera speed was 25 frames per second, which resulted into the situation that fast moving particles were represented as stripes. The particle velocity could be derived from the length of these stripes. A stripe with a length of, for example, 3 cm implies a velocity of 0.75 m/s.

Disadvantages of this method are its inaccuracy and the laborious way to achieve results from the measurements.

4.4.2 Calibration of instruments

Wave height meters

The wave height meters had to be cleaned and calibrated every day. The cleaning was needed to ensure the conductivity of the gauge and was done with alcohol. Calibration was carried out by giving the gauge different immersion depths in still water. For each immersion depth the measured depths from the wave height meter, which was made visible by means of DASYLab (see paragraph 4.5), as well as from the probe were noted. The difference between the value of the wave height meter and the value of the probe was smaller than 1% of the immersion depths during all experiments.

Laser Doppler Anemometer

Calibration of the Laser Doppler Anemometer was not necessary because LDV is an exact measuring method. The measured velocity is not based on any quality of the water, but is directly based on the phase shift of particles in the water.

Video recordings

The velocity measurements by means of video recordings could be calibrated by measuring the velocity at a particular location by LDV as well as by video recordings themselves. The phase-averaged velocities measured by means of LDV are assumed
to be the exact mean velocities. The difference of the two measurements is caused by various reasons:

- a measured velocity by means of the video recordings is no velocity at a particular coordinate but the average velocity of a paraffin ball in the neighbourhood of that coordinate,

- the velocity is no phase average of hundred waves, but only of one or two waves,

- the paraffin balls can not follow all velocity fluctuations,

- the velocity close to the side-wall deviates from the velocity in the centre of the flume.

The velocity measurements by means of the video recordings can be compared to the LDV measurements, if there are only small differences between the two measuring methods.

4.5 Measurements and data processing

To be able to analyse the measurements the wave height meters and the Laser Doppler Anemometer were connected to a computer. The analogue signals from the meters were converted into digital signals and monitored by the data acquisition system DASYLab. Applications of this program are: saving data on disc, calculating averages, etc. The sampling frequency for the water level and velocity measurements was set at 50 Hz to be able to get information about the turbulent fluctuations.

The wave height at the toe of the slope was used as reference wave height during all experiments. In fact, this measured wave height consists of an incident wave and a reflected wave. Sistermans (1993) and Grote (1994) neglected the reflected waves, because on a slope of 1:25 only a very small part of the waves is reflected. However, for a slope of 1:10 the reflected wave can not be neglected anymore, but has to be measured too. The programs HARMO and CONASC from AUKE/pc were used to calculated the reflection coefficients of the measured waves. Therefore, two wave gauges had to be installed at the toe of the slope. The optimum distance between these wave gauges was about one fourth of the local wave length.

The wave gauges were installed on movable supports, so that the wave gauges could be moved to each location in the flume. The Laser Doppler Anemometer was installed on a rail-guided carriage with a mobile part that could move up and down (measuring bridge).

4.6 Analysing data

After the data was recorded on hard disk the data had to be analysed. The recorded velocity was analysed by splitting the velocity in a time-varying mean-flow part and in
a time-varying turbulent-velocity part. This was done by obtaining the ensemble-averaged velocities and turbulent velocity fluctuations. The recorded velocities were ensemble-averaged by phase-averaging the velocity, separated by the wave period over about hundred successive waves. The ensemble-averaged turbulent velocities were obtained by computing the standard deviation of each phase of the recorded velocities. Figure 4.3 shows the meaning of the phase of a wave. The sampling frequency of the measurements was 50 Hz, so $\Delta t = 0.02$ s.

![Figure 4.3: Phase of a wave](image)

### 4.7 Scale effects

The aim of model experiments is to use the results to describe the processes taking place in a real situation. For this reason scaling of the results has to be possible, because in this model the dimensions of stones and waves are much smaller than in a prototype situation. This is only justified if scale effects are not involved. Sleath (1978) summarized the stability measurements in oscillating flow of different authors. He found that the dimensionless shear stress, $\psi$, remains constant at a value of 0.055, if the dimensionless grain diameter $d_\ast > \pm 200$. With $d_\ast = d (\Delta g/u^2)^{1/3}$, $d$ has to be larger than roughly 7.5 mm. The grain diameter in the experiments is 12.1 mm, so no scale effects have to be expected considering the stone size.

Stive (1985) investigated the influence of the wave height on scale effects for a slope of 1:40. No significant scale effects seem to appear for wave heights ranging from 0.1 m to 1.5 m. The minimum wave height in the experiments was 0.06 m, so some scale effects can be expected. However, the conclusions by Stive were based on irregular wave experiments. A considerable part of the waves with a significant wave height of 0.1 m, relates to wave smaller than 0.06 m, so in this thesis scale effects, even for the lowest waves, are neglected.
4.8 Test procedures

The first experiments were carried out to define the wave height for three different wave steepnesses, where incipient movement of the stones occurred. Also the location of maximum wave attack was determined. Before the experiments were started first the settings of the wave board had to be adjusted iteratively, to obtain the required wave heights.

The test procedure for these experiments was as follows:

1. Put the stones in coloured strips with a band width of 25 cm.
2. Fill the wave flume until the water level is 0.6 m.
3. Install the wave gauges at the wanted locations.
4. Calibrate the wave gauges (only at the beginning of the day).
5. Follow the procedures for starting the wave board.
6. Start the monitoring file of DASYLab.
7. Stop the test after 750 waves.
8. Lower the water level below the most downslope located strip.
9. Determine the damage for each strip.

After the wave height and the location of maximum damage were determined for three wave steepnesses (1%, 3% and 5%), velocity measurements were carried out by LDV. Before these experiments could start, first some adjustments had to be made to the model. The coloured stones were removed and a layer of non-coloured stones was fixed on the slope applying cement mortar. This was done to avoid profile development as well as to maintain a certain roughness for the slope. Before these experiments could start, first the sides of the flume had to be cleaned to get a clear laser signal.

![Diagram](image)

**Figure 4.4:** Model arrangement of LDV measurements

Also the water level had to be lowered to 0.55 m because the region with most severe wave attack had to be re-located between two supports of the flume. Figure 4.4 shows the model arrangement.
To improve the signal of the LDV, particles were seeded to the water to scatter the laser light. After achieving a good Doppler signal the experiments were carried out, following the next procedures.

1. Fill the flume until the required water level is reached (0.55 m).
2. Clean and calibrate the wave gauges.
3. Install the wave gauges at the toe of the slope and at the location where the velocity measurement will be carried out.
4. Switch on the Laser Doppler Anemometer.
5. Start the wave board, following the procedures.
6. Monitor the wave height with DASYLab.
7. Wait about ten minutes until the wave height reaches a constant value.
8. Stop the wave board if the measured wave height deviates too much from the required wave height, and start again at point 5.
9. Set the LDV at the wanted position and check the Doppler signal.
10. When the signal is not good due to scratches on the glass, re-locate the LDV to a slightly different location.
11. When the signal is good, start the measuring file of DASYLab.
12. Stop the file after at least 100 waves.
13. Stop the wave board.

Application of Laser Doppler Velocimetry was limited to breaking waves without or almost without air bubbles. In the breaking wave region with many air bubbles, velocity measurements were carried out by means of video recordings. The procedure for these measurements was a follows.

1. Cover the measuring section of the wave flume with black cloth.
2. Place the UV-tubes close to the side-wall of the flume.
3. Place the video camera at such position that a 20 - 30 cm wide part of the breaking region is shown on the TV screen.
4. Start the wave board, following the procedures.
5. Wait till the waves have reached an equilibrium.
6. Add the paraffin particles to the water.
7. Start the video recordings.
8. Stop the video recordings when the particles are spread over the flume too much.
9. Velocity measurements can now be carried out by following the particles, watching image by image of the video recording.
5 Experimental results

In this chapter an overview is given of the experimental results. First the results are presented for the determination of the three wave heights at which incipient motion of the stones occurred. Subsequently, an overview of the velocity measurements for these particular waves is presented. Finally the development of the wave height in the breaking region is presented for the three waves.

5.1 Damage experiments

Movement of stones was defined as the re-location of a stone from one strip to another strip. For incipient movement only the strip with maximum damage is of importance. Therefore, only the maximum damage for the performed experiments is presented (Table 5.1). In this table, taking for example experiment H6s1, H6 stands for a required wave height at the toe of 6 cm and s1 for a required wave steepness of 1%. The number of waves was chosen to be 750, to be able to compare the results with the results of Sistermans (1993) and Grote (1994). The location of the strips is given in Table 5.2 and in Figure 5.1.

<table>
<thead>
<tr>
<th>Experiment</th>
<th>T (s)</th>
<th>H_{loc, meas}</th>
<th>s_{0, meas}</th>
<th>max. dam.</th>
<th>Strip number</th>
</tr>
</thead>
<tbody>
<tr>
<td>H6s1 (1)</td>
<td>1.96</td>
<td>5.9</td>
<td>1.0</td>
<td>16</td>
<td>7</td>
</tr>
<tr>
<td>H6s1 (2)</td>
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<td>6.0</td>
<td>1.0</td>
<td>10</td>
<td>7 / 8</td>
</tr>
<tr>
<td>H7s1</td>
<td>2.12</td>
<td>7.1</td>
<td>1.0</td>
<td>113</td>
<td>7</td>
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<td>1.0</td>
<td>120</td>
<td>7</td>
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<td>10.7</td>
<td>1.1</td>
<td>349</td>
<td>6</td>
</tr>
<tr>
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<td>1.1</td>
<td>333</td>
<td>6</td>
</tr>
<tr>
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<td>1.0</td>
<td>231</td>
<td>6</td>
</tr>
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<td>10.0</td>
<td>3.0</td>
<td>11</td>
<td>7</td>
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<td>16.1</td>
<td>3.0</td>
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<td>23.7</td>
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</table>

Table 5.1: Performed damage experiments
Table 5.2: Location of the strips

<table>
<thead>
<tr>
<th>Strip number</th>
<th>Min. X-coord. (m)</th>
<th>Max. X-coord. (m)</th>
<th>Min. depth (cm)</th>
<th>Max. depth (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.325</td>
<td>3.575</td>
<td>19.25</td>
<td>21.75</td>
</tr>
<tr>
<td>2</td>
<td>3.575</td>
<td>3.825</td>
<td>16.75</td>
<td>19.25</td>
</tr>
<tr>
<td>3</td>
<td>3.825</td>
<td>4.075</td>
<td>14.25</td>
<td>16.75</td>
</tr>
<tr>
<td>4</td>
<td>4.075</td>
<td>4.325</td>
<td>11.75</td>
<td>14.25</td>
</tr>
<tr>
<td>5</td>
<td>4.325</td>
<td>4.575</td>
<td>9.25</td>
<td>11.75</td>
</tr>
<tr>
<td>6</td>
<td>4.575</td>
<td>4.825</td>
<td>6.75</td>
<td>9.25</td>
</tr>
<tr>
<td>7</td>
<td>4.825</td>
<td>5.075</td>
<td>4.25</td>
<td>6.75</td>
</tr>
<tr>
<td>8</td>
<td>5.075</td>
<td>5.325</td>
<td>1.75</td>
<td>4.25</td>
</tr>
</tbody>
</table>

Figure 5.1: Location of the strips

The X-coordinate is defined as the location on the slope, with the toe of the slope situated at X = 0 and the still water line at X = 5.500.

Stones were displaced in upslope direction as well as in downslope direction. Differences could be observed between experiments with different wave steepnesses. The experiments with a wave steepness of 1% resulted in displacement of stones almost only in upslope direction. On the other hand a wave steepness of 5% resulted in an equal distribution of damage in upslope and downslope direction. The results of experiments with a wave steepness of 3% were in between the results of 1% and 5%.

The experiments of Sistermans (1993) showed that the total damage in downslope direction was higher than in upslope direction. However, those experiments were carried out on a slope of 1.25, resulting in lower $\xi$-values.

Considering Sistermans' experiments and the new experiments, it seems that the breaker parameter has influence on the displacement of stones in downslope or upslope direction. More experiments have to be carried out to check whether this is true.
The regular wave experiments of Sistermans (1993) and Grote (1994) (slope 1:25) showed the trend that maximum damage was located at $h/H_0 = 1$. In the experiments carried out during this research (slope 1:10), however, $h/H_0$ was about 0.6. This infers that the slope angle probably has great influence on this ratio.

In Figure 5.2 the maximum damage is plotted against the wave height for three wave steepnesses. Trend lines are plotted to show the expected trend that higher wave steepnesses require a higher wave to cause the same level of damage.

![Graph showing the relationship between wave height and number of stones displaced for different steepnesses.](image)

**Figure 5.2:** Maximum damage related to the wave height and the wave steepness

Sometimes a higher wave inflicts less maximum damage than a smaller wave with the same wave steepness, for example the ones indicated with an arrow in Figure 5.2. This is caused by the way the maximum damage is determined. If most severe wave attack is located in between two adjacent strips, the maximum damage will be lower than if the location of most severe wave attack is situated in the middle of a strip. In the first case the maximum damage is, as it were, spread over two strips. Smaller strip lengths would probably have avoided this phenomenon.

### 5.2 Defining of wave heights for incipient motion

The maximum damage is of course also dependent on the width of the flume and the length of a strip. To avoid this dependency, the damage is related to the number of stones in the top layer of a strip. As already described in paragraph 3.5 this is defined as the number of stones with an area $D_{50}$ that fit in the area of a strip. In formula form the local damage percentage is noted as:
\[
S_{\%} = \frac{n}{A} D_{n50}^2
\]  
(3.10)

The flume width is 80 cm and the strip length is 25 cm, resulting in a number of 1366 stones with \(D_{n50} = 1.21 \text{ cm}\) in the top layer of a strip. Subsequently, the maximum damage can be expressed as the percentage of the stones in the top layer of a strip, that are displaced to other strips. This is denoted by \(S_{\%_{\text{max}}}\). Of course this percentage is dependent on the length of a strip. Larger strip lengths result in smaller percentages. The strip length was chosen to be 25 cm, in order to be able to compare the results with the experimental results of Sistermans (1993) and Grote (1994).

Theoretically for incipient motion no damage is allowed, so \(S_{\%_{\text{max}}}\) then has to be zero. In practice incipient motion is arbitrarily defined as a certain damage percentage: in this thesis as 1\%. This means a displacement of about 13 stones out of the strip with maximum damage. The wave heights which cause incipient motion of the stones are determined by defining the waves which cause a displacement of approximately 13 stones at maximum out of a strip. For wave steepnesses 1\%, 3\% and 5\%, wave heights of 6 cm, 10 cm and 14 cm cause a maximum damage of 13, 11 and 15 stones, respectively. The maximum damage for the wave with a wave height of 6 cm seems to be located in between two adjacent strips. Because of this, only considering the maximum damage seems to be inadequate to define incipient motion. However, from visual observations it is concluded that these waves cause about the same damage at the location of severest wave attack. Therefore, in this thesis, these waves are considered to be the waves which cause incipient motion of the stones.

5.3 Water particle velocity measurements

5.3.1 Laser Doppler velocity measurements

Knowledge of the velocities in breaking waves is important for the understanding of the physical processes which cause damage to a slope. Therefore, water particle velocity measurements were carried out for the three different waves at which incipient motion of the stones occurred. An overview of the location of the measurements is given in the Tables 5.3, 5.4 and 5.5. The z-coordinates are chosen to be positive going down from the mean water level. In the region nearby the strip with incipient movement of the stones, it was tried to measure the velocity as close as possible to the bottom. In practice this was about one centimetre above the bottom.
### Table 5.3: Location of velocity measurements with $H_{ioe} = 6$ cm and $s = 1\%$

<table>
<thead>
<tr>
<th>$H_{ioe}$ (cm)</th>
<th>$H_{x,ioe}$ (cm)</th>
<th>X -coordinate (m)</th>
<th>z -coordinate (cm)</th>
<th>Water depth (cm)</th>
</tr>
</thead>
<tbody>
<tr>
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### Table 5.4: Location of velocity measurements with $H_{ioe} = 10$ cm and $s = 3\%$

<table>
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<th>$H_{ioe}$ (cm)</th>
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### Table 5.5: Location of velocity measurements with $H_{toe} = 14$ cm and $s = 5\%$

<table>
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<th>$H_{toe}$ (cm)</th>
<th>$H_{X,loc}$ (cm)</th>
<th>$X$ -coordinate (m)</th>
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</tr>
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<td>9.0</td>
<td>10.0</td>
</tr>
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<td>9.5</td>
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<td>6.9</td>
<td>4.62</td>
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<td>8.8</td>
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</table>

Figure 5.3 shows a LDV measurement, where $u$ is the velocity component parallel to the slope and $w$ the velocity component perpendicular to the slope. $u$ is positive in upslope direction and $w$ is positive upwards. The velocity is obtained by phase-averaging the instantaneous velocity over about hundred successive waves. From now on $u$ is simply called the horizontal velocity and $w$ the vertical velocity. Appendix II contains the other velocity measurements at the locations given in Tables 5.3 through 5.5.

![Graph](image_url)

**Figure 5.3:** Velocity measurement
Unfortunately no LDV measurements could be carried out in the time span of the wave where the plunging jet reaches the bottom, because of too many air bubbles in the water. Figure 5.4 shows a LDV signal where the parts of the signal indicated with arrows can be ascribed to signal drop-outs. The signal component in this figure is shown as it is measured with the Laser Doppler Anemometer. It has not yet been converted into a horizontal and a vertical velocity component. The signal drop-out implies that the signal’s voltage level remains almost constant if the signal intensity drops below a certain value. When the intensity of the signal increases again, a sudden jump occurs to the first measured value after the drop-out.

![Figure 5.4: Drop-out of LDV signal](image)

### 5.3.2 Video recording of particle motions

To obtain information about the water particle velocities in the region where air bubbles restrict the LDV measurements, supplementary velocity measurements were carried out by means of video recording. However, as described in Chapter 4, this kind of velocity measurements is probably not very accurate. To determine the accuracy, the measurements were carried out at some locations where LDV measurements had been carried out already. The difference between the measured velocities of the two kinds of measurements can be ascribed to reasons listed in paragraph 4.4.2. Figure 5.5 shows this comparison for one wave, in the region of severest wave attack.
Figure 5.5: Comparison of velocity measurements by means of LDV and video recording

The conformity of the two kinds of measurements is remarkably good. Therefore, it is probable that velocity measurements by means of video recording represent the water particle velocity reasonably well. The appearance of the high downwards directed velocities in the drop-out phase of the LDV measurements shows the fact that the plunging jet reaches the bottom in this phase.

Figure 5.6, 5.7 and 5.8 show velocities for the three different wave conditions in the region of severest wave attack. For the longest wave (s = 1%) also LDV measurements could be carried out, except for the plunging-jet phase. Other velocity measurements derived from the video recordings are given in Appendix II.

Figure 5.6: Velocity in the region of severest wave attack for \( H_{10} = 6 \text{ cm} \) and \( s = 1\% \)
Figure 5.7: Velocity in the region of severest wave attack for $H_{10s} = 10$ cm and $s=3\%$ (only video recording measurements)

Figure 5.8: Velocity in the region of severest wave attack for $H_{10s} = 14$ cm and $s=5\%$
5.4 Water level measurements

During the LDV measurements, water level variations were recorded at the toe of the slope and at the locations of the velocity measurements. The measured waves consist of an incident wave and a reflected wave. The reference wave height used in the description of the experiments is the incident wave height at the toe of the slope. The reflection coefficient is defined as the ratio between the reflected and the incident wave height. The averaged measured values of the reflection coefficient are shown in Table 5.6. In Figure 5.9 these measured values are inserted in the $r,\xi$-relation by Battjes (1974). From this figure it can be concluded that the reflection coefficients coincide with previous measurements and with the $r,\xi$-relation by Battjes.

<table>
<thead>
<tr>
<th>Wave height (cm)</th>
<th>Wave period (sec)</th>
<th>$\xi$ (-)</th>
<th>$r$ (-)</th>
</tr>
</thead>
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<td>0.050</td>
</tr>
<tr>
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<td>0.040</td>
</tr>
<tr>
<td>14</td>
<td>1.34</td>
<td>0.45</td>
<td>0.046</td>
</tr>
</tbody>
</table>

Table 5.6: Reflection coefficients

![Reflection coefficient graph](image)

Figure 5.9: Reflection coefficient $r$ as function of $\xi$

Also additional water level measurements were carried out to determine the exact location of wave breaking and to follow the development of the wave height in the breaking region. Figure 5.10, 5.11 and 5.12 contain the development of the wave height for the three different waves.
Figure 5.10: Wave height development in the breaking region for $H_{ioe} = 6$ cm and $s = 1\%$

Figure 5.11: Wave height development in the breaking region for $H_{ioe} = 10$ cm and $s = 3\%$
Figure 5.12: Wave height development in the breaking region for $H_{10s} = 14$ cm and $s = 5\%$

<table>
<thead>
<tr>
<th>$H_{10s}$ (cm)</th>
<th>$s_0$ (-)</th>
<th>$H_b$ (cm)</th>
<th>$X_b$ (m)</th>
<th>$h_b$ (cm)</th>
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<td>16.6</td>
<td>3.47</td>
<td>20.3</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Table 5.7: Wave heights at breaking

The location of wave breaking is defined as the location where the wave height reaches its maximum. Table 5.7 contains the breaking wave heights and locations for the three waves.

Beyond the breaking point it is more difficult to clearly define a wave. It is still possible to distinguish a wave period, but the passing of the wave top is difficult to see, because of the splashing of water in a plunging breaker. When phase-averaging the water level measurements near the plunging point of the wave, two wave tops can be distinguished. The first is due to the splashing and the second is the normal wave top. The first top is more apparent for the longer waves, which is caused by the stronger plunging characteristics. It can be even higher than the actual wave as is shown in Figure 5.13, where $\zeta$ is the water level elevation.
Figure 5.13: Water level variation in the plunging region (s = 1%)
STABILITY OF STONES IN THE SURF ZONE
6 Analysis of experimental results

In the previous chapter the results from the experiments were presented. To use these results to draw conclusions, the validity of the experiments has to be guaranteed. In this chapter the results are analysed and compared with theories of previous researchers by making use of dimensionless parameters. This was only possible for the results of the damage experiments and the wave height measurements, since no theories are available for velocities in plunging breakers. Further, it is made plausible that phase-averaging of the velocities is justified and that the deviations of the mean flow can be ascribed to turbulent fluctuations.

6.1 Analysing results for incipient motion

Sistermans (1993) derived dimensionless relations for the stability of stone under regular-wave attack, by making use of the dimensionless breaker and stability parameters. For several maximum damage percentages, relations were expressed as function of these two parameters. In Figure 6.1 values measured by Sistermans (1993) as well as the new experimental results are inserted for a damage percentage of 1%, in this thesis called incipient motion. The trend is very clear: waves with smaller $\zeta$ require a higher wave to cause incipient motion. The values of the stability parameter for the new experiments are higher than the-by means of curve fitting-obtained relation (equation 2.10). This might have several causes: first, curve fitting was done for a few values only, secondly, the values of Sistermans were obtained for spilling breakers only. No values were known for higher $\zeta$ ($\zeta > 0.25$), so curve fitting in this region might not be justified.

![Figure 6.1: $H/\Delta D_{50}$ as function of $\zeta$](image)

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Pilarczyk and Den Boer (1983) produced stability formulae which were derived from regular wave experiments with slope angles varying from \( \cot \alpha = 1.5 \) to \( \cot \alpha = 5 \). For plunging breakers the formula reads:

\[
\frac{H}{\Delta D} = 2.25 \cdot \xi^{-0.5} \cdot S_R
\]

(6.1)

where:
\[ S_R = \tan \phi \cos \alpha + \sin \alpha \]
\[ \phi = \text{angle of repose} \]
\[ \alpha = \text{slope angle} \]

This relation is also plotted in Figure 6.1. For \( \xi = 1 \) the measured value matches with this relation. Smaller \( \xi \) lead to underestimation of the stability parameter. This has probably two reasons: first, the relation was based on experiments which contained no measurements for \( \xi \)-values smaller than 0.7. Secondly, spilling features have an advantageous effect on the stability of stones, leading to a higher stability parameter.

From the experiments it was concluded that waves with a higher \( \xi \) cause a mainly upslope displacement of the stones. This is in line with the fact that plunging breakers tend to build up beaches.

6.2 Analysing the velocity measurements

6.2.1 Mean velocities

Motion of stones in a breaking wave is caused by the turbulent flow pattern, which consists of the mean flow and the turbulent fluctuations. The mean flow is obtained by phase-averaging the measured velocities of a test period. This is only justified if the mean flow is constant during a test period. By first splitting the test period (about 100 waves) in ten equal parts, each with about 10 waves, and subsequently phase-averaging the velocity of each part, the variation of the mean flow can be made clear. In Figure 6.2 this is shown for one test. It can be concluded that the variation of the mean flow is only very small.
Figure 6.2:  Variation of mean velocities

The locations where the velocity measurements were carried out, can be subdivided into two parts. The first part stretches from the breaking point up to the location where air bubbles reach the bottom (see Figure 6.3). Of course in this part air bubbles are already present in the water due to wave breaking. However, air bubble formation starts at the water surface and gradually moves down to the bottom. Figure 6.4 represents the development of the maximum horizontal bottom velocities in the first part of the breaking region. Also results from the linear wave theory are inserted in this figure (the measured wave height is used to calculate the orbital velocity). It is clearly shown that the linear wave theory is not capable of calculating the velocities in a breaking wave.

Figure 6.3:  Region in which LDV measurements were performed
Figure 6.4: Comparison between the linear wave theory and the velocity measurements at the bottom ($H_{1/3} = 6 \text{ cm}, s = 1\%$)

In the second part air bubbles have reached the bottom and obstruct the LDV measurements. Wave plunging takes place in this part and therefore LDV measurements in the plunging jet were impossible.

From the additional measurements by means of video recording it can be concluded that all three waves show the same development in time with respect to the velocities in the area of severest wave attack. The presence of a plunging jet is clearly shown by the combined large horizontal and vertical velocities. The magnitudes of the velocities in the plunging jet diverge only little for the three wave conditions, which is in line with the fact that all waves cause incipient motion of the stones. All waves show some plunging characteristics, but the longer the wave the stronger is the plunging jet related to the deep-water wave height.

In the Figures 6.5, 6.6 and 6.7 particle trajectories in the plunging jet of the three waves are presented. The phase of the wave in which a particle appears is indicated with tick marks. The cross marks, for example, mean that these particles are in the same phase of the wave. In all figures particle trajectories are drawn for several waves, so in the same phase does not necessarily mean in the same wave. The direction of the plunging jet seems to be similar for all wave conditions. However, for the shortest wave, the jet seems to turn a little into horizontal direction when proceeding towards the slope.
**Figure 6.5:** Particle trajectories in the plunging jet with $H_{100} = 6$ cm and $s = 1\%$

**Figure 6.6:** Particle trajectories in the plunging jet with $H_{100} = 10$ cm and $s = 3\%$
Care should be taken when analysing the numerical values of the velocities in the plunging jet. In this phase of the wave considerable velocity fluctuations take place due to turbulence. Besides, for all three wave conditions only a few values of the velocity are known in this phase, so it is not sure whether the measured maximum velocities coincide with the real maximum velocities. Furthermore, the location of severest wave attack could not be determined exactly because of the strip dimensions. The locations of the velocity measurements are chosen arbitrarily in the region of severest wave attack, so it is not sure whether the measured maximum velocities are the highest in this region or not.

6.2.2 Turbulent velocities

The turbulent fluctuations are considered to be the deviations from the phase averaged velocities. This is only justified if the phase-averaged velocities remain constant during a test period, which is the case. In Figure 6.8 vertical as well as horizontal phase-averaged turbulent velocities are shown at a certain position. Appendix II contains all measured turbulent velocities. Turbulent velocities are highest immediately after the wave front has passed, and decline afterwards. A difference can be noted among the three waves. Especially for the waves with a steepness of 5% it can be seen that the turbulent energy does not fade away between two successive waves (see Appendix II). This was also found by Ting and Kirby (1996). They observed that the dissipation rate in spilling breakers is slow and therefore turbulent energy is more uniform in time. A wave with a steepness of 5% is in between a spilling and a plunging breaker, so spilling features can be expected for this wave.
The turbulence rate near the bottom increases gradually when the wave approaches smaller depths, which is shown in Figure 6.9 for the horizontal turbulent velocities. No turbulent velocities could be measured in the plunging jet, but it is expected that the turbulence rate reaches its maximum there.

**Figure 6.8:** Phase-averaged turbulent velocities ($s = 1\%$)  
($SD <u>$ means: the standard deviation of the horizontal velocity)

**Figure 6.9:** Maximum turbulent velocities near the bottom ($H_{te} = 6\; cm$, $s = 1\%$)
6.3 Analysing water level measurements

For regular-wave experiments it is important that the waves are truly regular. The wave height as well as the wave period has to be constant during a test period. Figure 6.10 shows the measured water level variations at the toe of the slope for a test period. About 100 wave periods are plotted in this figure. The band width of the water level variation is very small, so it can be concluded that only minor variation in wave characteristics occurs. This justifies phase-averaging of the water levels.

Wave parameters are often made dimensionless to compare different waves with each other. In Chapter 3 relations between dimensionless parameters are mentioned as used by Kaminsky and Kraus (1993). Figure 6.11, 6.12 and 6.13 present these empirical relations and the measured values of this thesis (\( \zeta \) is the water level elevation).

![Figure 6.10: Variation of the water level elevations at the toe of the slope](image)

\( H_{\text{toe}} = 10 \text{ cm, } s = 3\% \)
Figure 6.11: \( \gamma_b \) as function of \( \zeta \)

Figure 6.12: \( \beta_b \) as function of \( H_0/L_0 \)
Figure 6.13: $\Omega_b$ as function of $H_0/L_0$

The empirical relations predict the measured values quite well except for one value of the breaker-depth index, $\beta_b$. The wave with the largest steepness gives a too high value for $\beta_b$. Whether this is caused by the variability of the empirical relation or by the fact that $H_{10e}$ deviates from $H_0$ is difficult to say. $H_0$ differs at maximum a few percent from $H_{10e}$, so probably the deviation can be explained by the variability.
7 Theoretical approach of the stone stability

Most damage to a stone layer, which is attacked by plunging waves, takes place where the plunging jet reaches the bottom. In this chapter an attempt is made to explain the stone stability in plunging jets by theory. Two models are discussed, which consider the static stability of a single stone in a uniform plunging jet. These models are compared with the stability relation by Izbash.

7.1 Qualitative description of stone stability in the surf zone

As stated before, most damage takes place in the region where the plunging jet hits the rock layer. This is in the outer-breaker region, where large-scale vortices occur. The turbulent processes in this region are completely different from the quasi-steady breaking wave mode in the inner-breaking region. It is therefore not justified to compare the dissipation of wave energy with that in a bore or hydraulic jump.

Ting and Kirby (1995) assumed that in breaking waves sediment is stirred up by turbulence and transported by the mean flow. Whether this is also valid for stones has to be questioned.

From the experiments it was concluded that the longer waves, with more plunging characteristics, cause an upslope-directed transport of stones. For these waves the turbulence was concentrated in the phase at which the mean flow was directed upslope, namely when the plunging jet hits the bottom. No stones were displaced in downslope direction, from which it can be concluded that the downslope flow is not strong enough to cause incipient motion of the stones.

The temporal variation of the turbulent velocities for the shorter waves, with some spilling characteristics, was much smaller and the wave-period-averaged flow at the bottom was directed downslope. Stones were displaced into upslope as well as into downslope direction. The maximum flow velocity in downslope direction was of about the same magnitude for the three wave types, but the turbulent velocities were much higher for shorter waves compared to the longer waves, when the flow was directed downslope. Probably the flow in downslope direction, with large turbulent fluctuations, is responsible for the downslope displacement of the stones.

From the experiments by Sistermans (1993) with spilling breakers, it was concluded that downslope displacement of the stones dominated upslope displacement of the stones.

It seems that plunging breakers cause an upslope-directed transport of stones and spilling breakers a downslope-directed transport. Therefore, the assumption of Ting and Kirby (1995), that plunging waves tend to build up beaches, seems to be valid for sand as well as for stones.
7.2 Influence of plunging jet on stone stability

From the velocity measurements it was concluded that similar velocities of about 1 m/s occur in the plunging jets of the three investigated waves. Also the angles of incidence of the jets were similar. This is in line with the fact that these waves cause incipient movement of the stones. It seems that large pressure fluctuations in the plunging jet are responsible for the upslope displacement of the stones in the area of severest wave attack. No theories are available for the stability of stone in plunging jets, so it is difficult to explain why a plunging jet in a breaking wave causes instability of stones. In cooperation with the MSc-student R.Hakenberg, an attempt has been made to model the stability of stones in jets to increase the understanding of the physical processes in a jet. Two different models are discussed in this chapter: the static model and the simplified static model (see also Hakenberg (1997)). The models are compared with the relation by Izbash (1930) for uniform flow, which is a tool for first approximation of stone stability when the circumstances are not very clear:

\[ D_{n50} = \frac{0.7 \cdot u^2}{2 \Delta g} \]  

(7.1)

7.3 The static model

7.3.1 Features

The static model schematizes the stability of stones in a plunging jet starting from the load, consisting of static forces, on a single stone. The single stone is schematized as a cubical stone with sides \( d \), which protrudes above the adjacent stones (see Figure 7.1). \( d \) is supposed to be equal to \( D_{n50} \). The distance of the cubical stone to the adjacent stone is \( \delta \), and the protrusion is \( p \).

The second stone layer, the stone layer below the top layer, is schematized as an impenetrable surface, because the resistance to penetration of the jet is large, because of the small spaces between the stones. The forces on the stone are generated by a jet with a uniform flow velocity \( u \), which hits the stone at an angle \( \beta \).

The uniform jet is considered to be much wider than the stone dimensions.

![Figure 7.1: Schematization of the stone layer and the obliquely impinging jet](image)

Figure 7.1: Schematization of the stone layer and the obliquely impinging jet
The forces on the single stone are proportional to the pressure generated by the jet:

\[ F = 0.5 \cdot \rho_w \cdot u^2 \cdot C \cdot A \]  

(7.2)

where:
- \( \rho_w \) = mass density of water [kg/m\(^3\)]
- \( u \) = flow velocity in the jet [m/s]
- \( C \) = coefficient [-]
- \( A \) = surface of the single stone which is hit by the jet [m\(^2\)]

The load on the stone is schematized by the next forces (see Figure 7.3):
- \( F_D \): drag force on the side of the cubical stone
- \( F_{down} \): force on the top side of the cubical stone
- \( F_Y \): vertical force of reaction
- \( F_W \): (horizontal) friction force
- \( W \): submerged weight of the stone
- \( F_L \): lift force due to the generation of pressure under the cubical stone

The pressure under the stone is generated by the reflection of the jet against the bottom. This pressure is increased by extra pressure generation due to the restriction of the flow of the water under the stone.

For the determination of the lift force two situations can be distinguished, dependent on \( \beta \), \( p \) and \( \delta \):

Situation 1: The impinging jet turns downwards by influence of the left side of the cubical stone. The turned jet reflects against the bottom and hits the bottom side of the stone, generating an upwards directed force.

Situation 2: The same situation as for situation 1, plus an extra component for the direct reflection of the jet against the bottom.

The transition between both situations is represented in Figure 7.2. This transition takes place if \( \tan \beta = (d-p)/\delta \). Figure 7.3 represents the schematization of forces for situation 1 and situation 2.

![Figure 7.2: Transition between situation 1 and situation 2](image-url)
7.3.2 Forces on the cubical stone

Force on the side of the stone, $F_D$ (drag force)

Situation 1:

$$F_D = 0.5 \cdot \rho \cdot (u \cdot \cos \beta)^2 \cdot C_D \cdot R \cdot d$$

where: $R$ = contact height of the jet on the side of the cubical stone

Substitution of:

$$R = p + \delta \cdot \tan \beta$$
$$p = C_b \cdot d$$
$$\delta = C_\delta \cdot d$$

$$\Rightarrow F_D = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_D \cdot (C_b \cdot \sin \beta + C_\delta \cdot \cos \beta) \cdot \cos \beta \quad (7.3)$$

In situation 2, $R$ is equal to $d$, leading to:

$$F_D = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_D \cdot \cos^2 \beta \quad (7.4)$$

Force on the top side of the cubical stone, $F_{\text{down}}$ (down force)

$$F_{\text{down}} = 0.5 \cdot \rho \cdot (u \cdot \sin \beta)^2 \cdot d^2 \cdot C_{\text{down}} = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_{\text{down}} \cdot \sin^2 \beta \quad (7.5)$$
Lift force on the bottom side, \( F_L \)

**Situation 1:**

\[
F_L = 0.5 \cdot \rho \cdot u^2 \cdot C_{L1} \cdot b_L \cdot d
\]

Substitution of (see Figure 7.4):

\[
b_L = 0.5 \cdot B \cdot (1 + \sin \beta)
\]

(derived from the impulse balance of a jet impinging on a plane surface)

\[
B = R \cdot \cos \beta \\
R = \rho \cdot \delta \cdot \tan \beta \\
\rho = C_p \cdot d \\
\delta = C_s \cdot d
\]

\[
\Rightarrow B = (C_s \cdot \sin \beta + C_p \cdot \cos \beta) \cdot d
\]

\[
\Rightarrow F_L = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_{L1} \cdot (C_s \cdot \sin \beta + C_p \cdot \cos \beta) \cdot 0.5 \cdot (1 + \sin \beta) \tag{7.6}
\]

In situation 2 the lift force consists of two components:

\[
F_L = F_{L1} + F_{L2}
\]

where:

**Component 1:** \( F_{L1} = 0.5 \cdot \rho \cdot u^2 \cdot C_{L1} \cdot b_L \cdot d \)

Substitution of (see Figure 7.5)

\[
b_L = 0.5 \cdot B_L \cdot (1 + \sin \beta) \\
B_L = R \cdot \cos \beta \\
R = \text{constant} = d
\]

\[
\Rightarrow F_{L1} = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_{L1} \cdot \cos \beta \cdot 0.5 \cdot (1 + \sin \beta) \tag{7.7}
\]
Component 2: \( F_{l2} = 0.5 \cdot \rho \cdot u^2 \cdot C_{l2} \cdot B_2 \cdot d \)

Substitution of (see Figure 7.5):

\[
\begin{align*}
B_2 &= b_2 \cdot \sin \beta \\
b_2 &= \delta - x_1 + x_2 \\
x_1 &= \frac{d}{\tan \beta} \\
x_2 &= \frac{\rho}{\tan \beta}
\end{align*}
\]

\[
\Rightarrow \quad F_{l2} = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_{l2} \cdot (C_\rho \cdot \sin \beta + (C_\rho - 1) \cdot \cos \beta)
\]  

(7.8)

Friction force:
The friction on the cubical stone and its adjacent stones depends on the vertical load on the stone. From the vertical equilibrium of the stone it follows:

\[
\sum V = 0 \quad \iff \quad F_y = F_{\text{down}} + W - F_L
\]

The friction force equals:

\[
F_w = \mu \cdot F_y
\]

\[
\Rightarrow \quad F_w = \mu \cdot [F_{\text{down}} + W - F_L]
\]  

(7.9)

Submerged weight of the stone:

\[
W = (\rho_s - \rho_w) \cdot g \cdot d^3
\]  

(7.10)

7.3.3 Horizontal equilibrium of the static model

Situation 1

\[
\sum H = 0 \quad \iff \quad F_D - F_v = 0
\]

\[
\Rightarrow \quad F_D - \mu \cdot [F_{\text{down}} + W - F_L] = 0
\]

\[
\Rightarrow \quad d = \frac{u^2}{2\mu \Delta g} \cdot [\text{Term 1} - \text{Term 2} + \text{Term 3}]
\]  

(7.11)
where:

**Term 1**
Term for the drag force: \( C_D \cdot \cos \beta \cdot (C_s \sin \beta + C_p \cos \beta) \)

**Term 2**
Term for down force: \( \mu \cdot C_{down} \cdot \sin^2 \beta \)

**Term 3**
Term for the lift force: \( 0.5 \cdot \mu \cdot C_{L1} \cdot (C_s \sin \beta + C_p \cos \beta) \cdot (1 + \sin \beta) \)

**Situation 2**

\[
\sum H = 0 \Rightarrow F_D - F_W = 0
\]

\[
\Rightarrow F_D - \mu \cdot [F_{down} + W - F_z] = 0
\]

\[
\Rightarrow d = \frac{u^2}{2 \mu \Delta g} \cdot [\text{Term 1} - \text{Term 2} + \text{Term 3;1} + \text{Term 3;2}] \quad (7.12)
\]

where:

**Term 1**
Term for the drag force: \( C_D \cdot \cos^2 \beta \)

**Term 2**
Term for down force: \( \mu \cdot C_{down} \cdot \sin^2 \beta \)

**Term 3;1**
Term for the lift force component by deflection against the side of the stone:
\( \mu \cdot 0.5 \cdot C_{L1} \cdot \cos \beta \cdot (1 + \sin \beta) \)

**Term 3;2**
Term for the lift force component by direct reflection against the bottom:
\( \mu \cdot C_{L2} \cdot (C_s \sin \beta + (C_p - 1) \cdot \cos \beta) \)

**7.3.4 Moment equilibrium of the static model**

**Defining the location of the point of rotation**

The point of rotation is located on the right side of the cubical stone, see Figure 7.6. The distance of the point of rotation to the right-handed bottom angular point is \( x \cdot d \). The point of rotation is supposed to be located in between 0 and 0.5\( \cdot d \).
Defining the moment arms

Down force on the top side of the stone
This force acts on the centre of the top side of the stone. Therefore, the moment arm is 0.5·d.

Lift force
The location on which the lift force acts is unknown, but it is supposed to be in between 0.5·d and d (see Figure 7.6).

![Figure 7.6: Moment arms](Image)

Drag force on the side of the stone
This force acts in the centre of the contact surface \((R)\) of the side of the stone. The distance \(b·d\) is flexible in situation 1 according to:

\[ b·d = (1 - 0.5·C_b·\tan\beta - 0.5·C_p - x)·d \]

In situation 2 the contact surface covers the entire side of the cubical stone. The moment arm is:

\[ b·d = (0.5 - x)·d \]

Friction force
Considering the equilibrium of moment, the point of rotation is the only contact point of the stone. The friction force is supposed to act in this point, which implies that the friction force has no influence on the equilibrium of moment.

The submerged weight of the stone
The weight acts in the centre of the stone leading to a moment arm 0.5·d.

Situation 1

\[ \sum M = 0 \quad \Rightarrow \quad F_D · b·d + F_L · a·d - F_{down} · 0.5d - W · 0.5d = 0 \]

\[ \Rightarrow \quad d = \frac{u^2}{\Delta g} \cdot [Term1 - Term2 + Term3] \quad (7.13) \]
7 THEORETICAL APPROACH OF THE STONE STABILITY

where:

**Term 1**
Term for the moment due to the drag force: \( C_D \cdot \cos \beta \cdot (C_s \cdot \sin \beta + C_p \cdot \cos \beta) \cdot b \)

**Term 2**
Term for the moment due to the down force: \( 0.5 \cdot C_{down} \cdot \sin^2 \beta \)

**Term 3**
Term for the moment due to the lift force: \( 0.5 \cdot C_{L1} \cdot (C_s \cdot \sin \beta + C_p \cdot \cos \beta) \cdot (1 + \sin \beta) \cdot a \)

**Situation 2**

\[
\sum M = 0 \quad F_D \cdot b \cdot d + F_L \cdot a \cdot d - F_{down} \cdot 0.5d - W \cdot 0.5d = 0
\]

\[d = \frac{u^2}{\Delta g} \cdot \left[ Term1 - Term2 + Term3;1 + Term3;2 \right] \quad (7.14)

where:

**Term 1**
Term for the moment due to the drag force: \( C_D \cdot \cos^2 \beta \cdot (0.5 - x) \)

**Term 2**
Term for the moment due to the down force: \( 0.5 \cdot C_{down} \cdot \sin^2 \beta \)

**Term 3;1**
Term for the moment of the lift force component due to reflection against the side of the stone: \( a \cdot (0.5 \cdot C_{L1} \cdot \cos \beta \cdot (1 + \sin \beta)) \)

**Term 3;2**
Term for the moment of the lift force component due to direct reflection against the bottom: \( a \cdot C_{L2} \cdot (C_s \sin \beta + (C_p - 1) \cdot \cos \beta) \)

7.3.5 Coefficients in the equations

**Geometric coefficients**

- \( C_s \) en \( C_p \): Both coefficients represent the irregular shape of the stone layer. \( C_s \) represents the space between adjacent stones and \( C_p \) represents the protrusion of the stones.

- \( a \) en \( b \): See Figure 7.6.
Mechanical coefficient

\( \mu \)  
Represents the friction between stones. For rip-rap this is: \( \mu \sim 0.6-0.8 \).  

Hydraulic coefficients

C_D, C_{down}, C_{L1}, and C_{L2} are coefficients for successively the drag force, the down force and the subdivided lift forces. These coefficients are used to take into account uncertainties about the flow around stones and the pressure set-up.  
The coefficient of the lift force for reflection, at first against the side of the stone \((C_{L1})\), is supposed to be half the coefficient for the lift force in the case of direct reflection against the bottom \((C_{L2})\). For the last case, the jet is directly reflected against the bottom without being deflected by the side of the stone. This probably increases the influence on the lift force.

7.4 Simplified static model

7.4.1 Difference from the static model

The static model consists of two relations: the first is valid if \( \tan \beta < (d-p)/\delta \) and the second is valid if \( \tan \beta > (d-p)/\delta \). In reality no distinction can be made between the two situations. Therefore, this distinction is left out of consideration in the simplified model. In the simplified model only one relation exists, which is valid for \( \beta = 0^\circ \) to \( 90^\circ \). In this model the virtual contact height \( R \), is introduced, which is used to calculate the intrusion of the jet into the stone layer (see Figure 7.7). The length of \( R \) is \( \delta \cdot \tan \beta + p \). The virtual contact height is bounded by \( S \) and \( S' \), which are defined as the points of intersection of the jet and the imaginary parallel line through the left side of the cubical stone. The virtual contact height is of course not a real contact height because the real contact height can not be larger than the side of the cubical stone. It is only used to calculate the influence of the angle of incidence of the jet on the drag force.

Another simplification is the schematization of the lift force. The lift force in the static model consists of complex terms which are uncertain. Therefore, a simple term, which is only dependent on the vertical component of the jet, is probably sufficient to model the lift force.

The irregularity of the stone layer is difficult to quantify and therefore different values for \( C_\theta \) and \( C_p \) are of no use, because both coefficients represent the irregularity. So from now on \( C_\theta \) and \( C_p \) are equal and called \( C_{rock} \).
7 Theoretical approach of the stone stability

Figure 7.7: The impinging jet in the simplified model

7.4.2 Forces on the cubical stone

Force on the side of the stone, $F_D$ (drag force)

$$F_D = 0.5 \cdot \rho \cdot (u \cdot \cos \beta)^2 \cdot C_D \cdot C_R \cdot d^2$$

where: $C_R = R/d$

Substitution of:

$$C_R = \frac{p + \delta \cdot \tan \beta}{d}$$

$p = C_p \cdot d$

$\delta = C_s \cdot d$

$C_p = C_s = C_{rock}$

$$\Rightarrow F_D = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_D \cdot C_{rock} \cdot (\sin \beta + \cos \beta) \cdot \cos \beta$$  \hspace{1cm} (7.15)

Force on the top side of the cubical stone, $F_{down}$ (down force)

$$F_{down} = 0.5 \cdot \rho \cdot (u \cdot \sin \beta)^2 \cdot d^2 \cdot C_{down} = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_{down} \cdot \sin^2 \beta$$  \hspace{1cm} (7.16)

Lift force on the bottom side, $F_L$

$$F_L = 0.5 \cdot \rho \cdot (u \cdot \sin \beta)^2 \cdot d^2 \cdot C_L = 0.5 \cdot \rho \cdot u^2 \cdot d^2 \cdot C_L \cdot \sin^2 \beta$$  \hspace{1cm} (7.17)
Stability of Stones in the Surf Zone

Friction force
The friction between the cubical stone and its adjacent stones depends on the vertical load on the stone:

\[ F_w = \mu \cdot [F_{down} + W - F_L] \quad (7.18) \]

Submerged weight of the stone

\[ W = (\rho_s - \rho_w) \cdot g \cdot d^3 \quad (7.19) \]

7.4.3 Horizontal equilibrium of the simplified model

\[ \sum H = 0 \quad \Leftrightarrow \quad F_D - F_w = 0 \]

\[ \Leftrightarrow \quad F_D - \mu \cdot [F_{down} + W - F_L] = 0 \]

\[ \Rightarrow \quad d = \frac{u^2}{2\mu \Delta g} \cdot [\text{Term } 1 - \text{Term } 2 + \text{Term } 3] \quad (7.20) \]

where:

Term 1
Term for the drag force: \( C_D \cdot C_{rock} \cdot \cos \beta \cdot (\sin \beta + \cos \beta) \)

Term 2
Term for the down force: \( \mu \cdot C_{down} \cdot \sin^2 \beta \)

Term 3
Term for the lift force: \( \mu \cdot C_L \cdot \sin^2 \beta \)

7.4.4 Moment equilibrium of the simplified model

From the sensitivity analysis of the static model (Appendix III) it follows that the location of the point of rotation has minor influence on the calculated stone size. For convenience the location of the point of rotation is chosen at the right bottom-angular-point of the cubical stone, so \( x = 0 \).

\[ \sum M = 0 \quad \Leftrightarrow \quad F_D \cdot b \cdot d + F_L \cdot a \cdot d - F_{down} \cdot 0.5d - W \cdot 0.5d = 0 \]

\[ \Rightarrow \quad d = \frac{u^2}{\Delta g} \cdot [\text{Term } 1 - \text{Term } 2 + \text{Term } 3] \quad (7.21) \]
where:

Term 1
Term for the moment due to the drag force: \( C_D \cdot C_{\text{rock}} \cdot b \cdot \cos \beta \cdot (\sin \beta + \cos \beta) \)

Term 2
Term for the moment due to the down force: \( 0.5 \cdot C_{\text{down}} \cdot \sin^2 \beta \)

Term 3
Term for the moment due to the lift force: \( C_L \cdot a \cdot \sin^2 \beta \)

### 7.4.5 Coefficients in the equations

**Geometric coefficients**

- \( C_{\text{rock}} \)  
  Same assumptions as for the static model.
- \( a \) en \( b \)  
  Measure for the moment arm for the lift force and the drag force, respectively.

**Mechanical coefficient**

- \( \mu \)  
  Same assumptions as for the static model.

**Hydraulic coefficients**

- \( C_D \), \( C_{\text{down}} \) en \( C_L \)  
  Coefficients for successively the drag force, down force and the lift force.

### 7.5 Assumptions of the values of the coefficients

For the practicability of the \( \beta \)-d-equations of the static models, values have to be allocated to the coefficients. The next assumptions are made for the determination of these values:

\( C_D = C_{\text{down}} = 1 \)

The pressures on the top side and the left side of the stone are supposed to be equal to the jet's pressure. No extra pressure is taken into account for possible hindered flow-off.

\( C_L = 2 \) (\( = C_{\text{lift}} = 0.5 \cdot C_{\text{L2}} \))

The lift coefficient's value is chosen larger than the drag coefficient's value, because accumulation of pressure is expected at the bottom side of the stone by hindered flow of the water.

\( a = 0.5 \)

The lift force is supposed to act in the middle of the bottom side of the stone.
$b=0.5$
In the simplified static model the drag force is supposed to act in the centre of the left side of the stone. In the static model $b$ depends on the angle of incidence of the jet and is not constant.

$x=0$
The location of the point of rotation is chosen at the right bottom angular point of the cubical stone, because in this way $x$ is zero and thus vanishes from the equation. This also leads to a conservative value of the calculated stone size.

$\mu=0.7$
This value comes from CUR (1995), p. 5-185 and can be applied for rip-rap.

$C_d=C_p=C_{rock}=0.5$
The irregularity of the stone layer is difficult to quantify and therefore different values for $C_d$ and $C_p$ are of no use, because both coefficients represent the irregularity. From now on $C_d = C_p = 0.5$ and is indicated by $C_{rock}$. This means that the stone in question protrudes $0.5 \cdot d$ and the irregularity of the sides of the stone causes a space of $0.5 \cdot d$ between adjacent stones.

The values of these coefficients are uncertain, so therefore a sensitivity analysis of these values is carried out. Appendix III contains the results of this analysis.

7.6 Comparison between the static models and the relation by Izbash

Results of both the models and the relation by Izbash (1930) are plotted in Figure 7.8. Generally the static models yield a larger stone size than the relation by Izbash. For both models the stone size calculated by horizontal equilibrium is decisive for almost all angles of incidence. For $\beta < 45^\circ$, great differences can be seen between both static models, which is caused by the smaller lift force in the simplified model for small angles of incidence of the jet.
Figure 7.8: Comparison between the static models and the Izbash relation

7.7 Limitations of the models

- The stone stability is only calculated for static horizontal and static moment equilibrium. In reality dynamic responses of a single stone may lead to other (dynamic) failure mechanisms.
- Both models do not take into account the influence of turbulence which causes pressure fluctuations.
- The width of the jet is limited, so the flow velocity and the angle of incidence above the stone layer are not constant.
- The values of the coefficients in the equations are uncertain, because the load and the influence of the shape of the stone on the stone stability are unknown.
- The static model has two different expressions for the calculation of the stone size for $0^\circ<\beta<90^\circ$, dependent on $\beta$, $C_s$ and $C_p$. The transition between both expressions can not be physically explained.
- In situation 2 of the static model the lift force is composed of two components. In reality these components are difficult to distinguish, because a stone is not cubical.
7.8 Conclusions for the modelling

Both models lead to relations which should be calibrated by experiments, otherwise the results can not be used for design purposes. Hakenberg (1997) performed experiments for obliquely impinging jets on a rock layer and concluded that for $50^\circ \leq \beta \leq 85^\circ$, the angle of incidence of the jet has only little influence on the damage to the rock layer. The \( \beta \)-d-relations of the static model seems to be fairly correct in a qualitative way, but whether the numerical results are correct has to be questioned. For jets with small angles of incidence (\( \beta \leq 50^\circ \)) no experiments were carried out, so the \( \beta \)-d-relations can not be calibrated for this range.

The \( \beta \)-d-relations of the simplified model are more convenient to apply than the relations for the static model, so these relations are used to derive a stone stability formula for plunging jets in breaking waves. The \( \beta \)-d-relation for horizontal equilibrium results into the largest stone size for all angles. Therefore, this relation is considered to be normative. With \( d = D_{n50} \), \( C_D = 1 \), \( C_{\text{rock}} = 0.5 \) and \( C_L = 2 \), (7.20) can be rewritten as:

\[
D_{n50} = \frac{u^2}{2\mu \Delta g} \cdot \left[ 0.5 \cdot \cos \beta \cdot (\sin \beta + \cos \beta) + \mu \cdot \sin^2 \beta \right] 
\]  
(7.22)

7.9 Stability relation for stones in a plunging jet of a breaking wave

The \( \beta \)-d-relations in this chapter are derived for uniform impinging jets on a horizontal rock layer, but in depth-limited breaking waves the rock layer is inclined. This results into an increase in the stability of the stones in a plunging jet, because stones are displaced in upslope direction and are exposed to gravity, which has a component opposite to the direction of movement of the stones. The positive influence of the slope angle on the stone stability can be expressed by the following relation:

\[
D_{\text{slope}} = \frac{\sin(\phi - \alpha)}{\sin \phi} \cdot D_{\text{plane}} 
\]  
(7.23)

where:
\( \phi \) = angle of repose of stones
\( \alpha \) = slope angle
\( D_{\text{slope}} \) = stone size on a sloping structure
\( D_{\text{plane}} \) = stone size on a plane structure

Combination of (7.22) and (7.23) leads to:

\[
D_{n50} = \frac{u^2}{2\mu \Delta g} \cdot \left[ 0.5 \cdot \cos \beta \cdot (\sin \beta + \cos \beta) + \mu \cdot \sin^2 \beta \right] \cdot \frac{\sin(\phi - \alpha)}{\sin \phi} 
\]  
(7.24)
From Figures 6.5 to 6.7 it can be concluded that velocities of 1 m/s and angles of incidence of 40° to 50° are characteristic for the plunging jet. With \( u = 1 \text{ m/s}, \beta = 45°, \mu = 0.7, \Delta = 1.95, g = 9.81 \text{ m/s}^2, \phi = 40° \) and \( \alpha = 5.71° \), (7.24) results into: \( D_{n50} = 2.8 \text{ cm} \). This equation leads to a larger stone size than the size of the stones used in the experiments. Apart from the fact that the model is not perfect, also uncertainties exist about the input of the model. For example the velocity in the jet, \( u \), is not constant. At the bottom this velocity is smaller than a few centimetres above the bottom. It is not sure at which height the velocity has to be chosen, to calculate the real pressure on a stone.

The stability relation by Izbash can also be adapted to sloping structures, by combination of (7.1) and (7.23):

\[
D_{n50} = \frac{0.7 \cdot u^2 \cdot \sin(\phi - \alpha)}{2\Delta g \cdot \sin \phi} \quad (7.25)
\]

This relation results into a stone size, \( D_{n50} = 1.6 \text{ cm} \), which is much closer to the experimental size than the stone size computed by equation 7.24. Therefore, it seems that the stone stability in a plunging jet with a very turbulent flow is not more unfavourable than the stone stability in uniform flow. This might be caused by the fact that the high velocity in a plunging jet only takes place during a very short time, which leads to inertia effects.

The velocity in the plunging jet is of course dependent on the wave height and also on the breaker type. However, still no wave theories exist which can calculate the velocity of a plunging jet in a breaking wave. An empirical relation between the wave characteristics and the velocity in the plunging jet can be written as:

\[
u = i \cdot \sqrt{g \cdot H_{\text{hoe}} \cdot \xi^j} \quad (7.26)
\]

where: \( i, j \) = coefficients

According to Battjes (1975) the parameter \( \sqrt{g \cdot H} \) is a logical choice for the scaling quantity for the particle velocity. By means of the experimental results, the next values can be found for the coefficients: \( i = 1.3 \) and \( j = 0.5 \). Equation 7.26 can now be rewritten as:

\[
u = 1.3 \cdot \sqrt{g \cdot H_{\text{hoe}} \cdot \xi} \quad (7.27)
\]

This equation is based on the experimental results for only three different waves, so it is very uncertain whether this equation can be used in general.

For design purposes, the relation by Izbash seems, up to now, more suitable than the relation derived from the static modelling. Combination of (7.25) and (7.27) leads to:
\[ D_{n50} = \frac{0.9 \cdot H_{lpe} \cdot \xi}{2\Delta} \cdot \frac{\sin(\phi - \alpha)}{\sin \phi} \]  

(7.28)

This relation can be rewritten as:

\[ \frac{H_{lpe}}{\Delta D_{n50}} = 2.2 \cdot \frac{\sin \phi}{\sin(\phi - \alpha)} \cdot \xi^{-1} \]  

(7.29)

In Figure 7.9 this relation is compared to the relation by Pilarczyk and Den Boer (1983), which was based on experiments with \( \xi > 0.7 \) and slopes with an angle of at least 1:5. Equation 7.29 is not tested for \( \xi > 1 \), but the conformity with Pilarczyk and Den Boer is quite good. Increasing the slope angle in equation 7.29 to 1:5 improves the conformity for larger \( \xi \), which can be expected because for \( \xi > 1 \) slopes are usually steeper than 1:10.

More experiments have to be carried out to find out whether the equations 7.28 and 7.29 can be used for a greater range of wave and structural characteristics.

**Figure 7.9:** Comparison between equation 7.29 and the relation by Pilarczyk & Den Boer
8 Conclusions and recommendations

8.1 Conclusions

From the results, presented in this report, the following conclusions can be drawn:

- The strip length is chosen too large to determine the maximum damage by means of the strips only. For some waves maximum damage is located in between two adjacent strips, leading to a too small maximum-damage percentage.

- Maximum damage is located at the position where the plunging jet reaches the bottom, which is at \( h/H_{toe} \approx 0.6 \). This position differs from the position found by Sistermans (1993), which was located at \( h/H_{toe} \approx 1 \). The result of Sistermans, however, was obtained for experiments on a slope 1:25. Therefore, the slope angle seems to have influence on the location of maximum damage.

- Decreasing the wave steepness leads to more displacement of stones in upslope than in downslope direction. From the experiments by Sistermans (1993) it was concluded that more stones were displaced in downslope direction. All breaking waves in the experiments of Sistermans were of the spilling-breaker type. Therefore, it seems that the breaker type has influence on the direction of displacement of the stones: spilling breakers lead to downslope displacement and plunging breakers lead to upslope displacement of the stones. This is also in line with the tendency that plunging breakers build up sandy beaches.

- The Laser Doppler Anemometer used in the experiments is not capable of measuring the velocity in the plunging jet of a breaking wave, because air bubbles in the water hinder the laser beams.

- The velocities and the directions of the plunging jets of the three waves are similar, which is in line with the fact that these waves cause incipient motion of stones at the location where the plunging jet reaches the bottom.

- The static models, which are described in chapter 7, do not predict the proper stone size for incipient motion in a plunging jet.

- The stability relation by Izbash for uniform flow, leads to approximately the same stone size as the stone size used in the experiments. From this it can be concluded that a plunging jet of a breaking wave does not inflict more damage to a stone layer than uniform flow with the same flow velocity.

- Equations 7.28 and 7.29 which are partly composed of the relation by Izbash and experimental results, seem to be suitable to calculate the stone stability in the plunging jet of breaking waves.
8.2 Recommendations

In this study a start is made to explain theoretically the stability of stones in breaking waves. For a continuation of this study, the following recommendations are given:

- Decreasing the strip length of damage experiments can facilitate the determination of the quantity and the location of maximum damage.

- Knowledge about the velocity field near the plunging jet can be improved by making use of other measuring equipment. An immersable Laser Doppler Anemometer can probably better measure velocities in the plunging jet, because the laser beam has to cross the water over a much smaller distance compared to the two-component Laser Doppler Anemometer used in the experiments. Another measuring method, which can improve knowledge of the velocity field, is Particle Image Velocimetry. This measuring method is more suitable and convenient to acquire a complete velocity field as compared to LDV, which is more suitable to measure the velocity and turbulent fluctuations in one point. The measuring method by means of video recordings, which was used in the experiments, is in fact a primitive kind of Particle Image Velocimetry, but is unsuitable to obtain a complete velocity field.

- Forces of a plunging jet acting on a stone are proportional to the pressure in the jet. Pressure measurements at the bottom should be carried out, to improve the knowledge about these forces.

- More experiments have to be carried out to determine the influence of the angle of incidence of the jet on the stone stability. This can improve the understanding of the forces of a jet acting on a stone layer.

- Turbulent velocity fluctuations in the plunging jet should be modelled, because these fluctuations cause pressure fluctuations, which can reduce the stability of a stone.

- Modelling the stability of stones in a plunging jet should be done by considering static equilibrium as well as dynamic equilibrium.

- More measurements have to be carried out to find out whether the stability relations of chapter 7 can be used for a larger range of slope angles and wave characteristics.
List of symbols

\( a \) = dimensionless moment arm of the lift force \([-]\)
\( A \) = area of a strip \( [m^2] \)
\( b \) = dimensionless moment arm of the drag force \([-]\)
\( b_{1,2} \) = measures for the impinging jet \( [m] \)
\( B_{1,2} \) = measures for the impinging jet \( [m] \)
\( c \) = wave celerity \( [m/s] \)
\( C_D \) = coefficient for the drag force \([-]\)
\( C_{\text{down}} \) = coefficient for the down force \([-]\)
\( C_L \) = coefficient for the lift force \([-]\)
\( C_{L1} \) = coefficient for the first component of the lift force \([-]\)
\( C_{L2} \) = coefficient for the second component of the lift force \([-]\)
\( C_p \) = dimensionless protrusion \([-]\)
\( C_{\text{stone}} \) = coefficient for the irregularity of the rock layer \([-]\)
\( C_s \) = dimensionless distance between adjacent stones \([-]\)
\( d \) = side of a cubical stone \( [m] \)
\( d_0 \) = stone size for horizontal flow with \( \Sigma H = 0 \) \( [m] \)
\( d_{lzbash} \) = stone size calculated with relation by lzbash \( [m] \)
\( D \) = stone diameter \( [m] \)

index: 15 = 15% value of sieve curve
50 = 50% value of sieve curve
85 = 85% value of sieve curve
\( n \) = nominal diameter
\( \text{plane} \) = stone size on a horizontal plane
\( \text{slope} \) = stone size on a slope

\( D_1 \) = dimensionless stone diameter \([-]\)
\( D_B \) = dissipation of wave energy due to breaking \( [N s^{-1} m^{-1}] \)
\( f_w \) = friction coefficient \([-]\)
\( F \) = calibration factor \([-]\)
\( F_D \) = drag force on the side of the cubical stone \( [N] \)
\( F_{\text{down}} \) = force on the top side of the cubical stone \( [N] \)
\( F_L \) = lift force on the bottom side of the cubical stone \( [N] \)
\( F_{L1} \) = first component of the lift force \( [N] \)
\( F_{L2} \) = second component of the lift force \( [N] \)
\( F_W \) = horizontal friction force \( [N] \)
\( F_V \) = vertical reaction force \( [N] \)
\( g \) = gravitational acceleration \( [m/s^2] \)
\( h \) = water depth \( [m] \)
\( h_b \) = water depth at wave breaking \( [m] \)
\( H \) = wave height \( [m] \)
\( H_b \) = wave height at breaking \( [m] \)
\( H_d \) = deep-water wave height \( [m] \)
\( H_m \) = maximum (irregular) wave height \( [m] \)
\( H_{rms} \) = root mean square of the wave height \( [m] \)
\( H_s \) = significant wave height \( [m] \)
\( H_{\text{loc}} \) = wave height at the toe of the slope  [m]
\( H_{X,\text{loc}} \) = wave height at the location of velocity measurements  [m]
\( k \) = wave number, 2\( \pi \)/\( L \)  [1/m]
\( L \) = wave length  [m]
\( L_0 \) = deep-water wave length  [m]
\( M \) = mass of a stone  [kg]
\( n \) = number of stones displaced  [-]
\( p \) = protrusion of a stone  [m]
\( q \) = turbulent-velocity scale  [m/s]
\( Q_B \) = fraction of the waves that are broken  [-]
\( r \) = reflection coefficient  [-]
\( R \) = contact surface on the side of the cubical stone  [m\(^2\)·m\(^{-1}\)]
\( Re \) = Reynolds number  [-]
\( s \) = wave steepness, \( H/L_0 \)  [-]
\( S_R \) = stability coefficient  [-]
\( S_{\%} \) = damage percentage  [%]
\( S_{S,\text{loc}} \) = local damage percentage  [%]
\( S_{S,\text{max}} \) = maximum local damage percentage  [%]
\( t \) = time  [s]
\( T \) = wave period  [s]
\( T_p \) = wave period with maximum wave energy  [s]
\( u \) = flow velocity of the impinging jet (chapter 7)  [m/s]
\( u \) = flow velocity component parallel to the slope  [m/s]
\( \langle u \rangle \) = mean velocity parallel to the slope  [m/s]
\( u_b \) = orbital velocity at the bottom  [m/s]
\( \bar{u}_b \) = maximum orbital velocity at the bottom  [m/s]
\( w \) = flow velocity component perpendicular to the slope  [m/s]
\( \langle w \rangle \) = mean velocity perpendicular to the slope  [m/s]
\( W \) = weight of rock under water  [N]
\( x \) = dimensionless location of the point of rotation  [-]
\( X_{1,2} \) = assistance measures  [m]
\( X \) = location on slope  [m]
\( X_b \) = location of wave breaking  [m]
\( z \) = distance downward from the mean water level  [m]
\( \alpha \) = slope angle of structure  [degrees]
\( \beta \) = angle of incidence of the jet  [degrees]
\( \beta \) = shape factor  [-]
\( \beta_b \) = breaker depth index, \( h_b/H_0 \)  [-]
\( \gamma_b \) = breaker height-to-depth index, \( H_0/h_b \)  [-]
\( \delta \) = distance between adjacent stones  [m]
\( \Delta \) = relative density, \((\rho_s-\rho_w)/\rho_w\)  [-]
\( \zeta \) = water level elevation  [m]
\( \mu \) = friction coefficient  [-]
\( \nu \) = kinematic viscosity  [m\(^2\)/s]
\( \xi \) = breaker parameter, tan \( \alpha/\sqrt{s} \)  [-]
\( \rho_s \) = mass density of stones  [kg/m\(^3\)]
$\rho_w = \text{mass density of water} \quad \text{[kg/m}^3\text{]}$

$\phi = \text{angle of repose of stones} \quad \text{[degrees]}$

$\psi = \text{dimensionless stability parameter} \quad [-]$  

$\omega = \text{angular frequency in waves, } 2\pi/T \quad \text{[1/s]}$

$\Omega_b = \text{breaker-height index, } H_b/H_0 \quad [-]$
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*Stability of rock on beaches*
MSc-thesis, Delft University of Technology, The Netherlands
Appendix I

Results of the damage experiments

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<th>Experiment H6s1 (1)</th>
<th>$H_{ice} = 5.9$ cm</th>
<th>$L_o = 6$ m</th>
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### Stability of Stones in the Surf Zone

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1.2
## Appendix I

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I.3
### Stability of Stones in the Surf Zone

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$H_{ioe} = 23.7\text{ cm}$  
$L_0 = 4.8\text{ m}$  
$T = 1.75\text{ s}$
Appendix II

Water level and velocity measurements

This appendix contains the measurements which were carried out in the surf zone for the three investigated waves. From page II.2 through II.14 the measurements for the waves with wave steepness 1% are presented. Subsequently, from page II.15 through II.30 the measurements with a steepness of 3% are presented. Finally, from page II.31 through II.43 the measurements for the waves with steepness 5% are presented. On each page, successively the water level variation, the mean flow and the turbulent velocities are presented. For the positions where no LDV measurements could be carried out, only velocity measurements by means of video recordings are presented.
APPENDIX II

Water level, $H = 6$ cm, $T = 1.96$ s, $X = 4.95$ m, $z = 4$ cm

![Water level graph](image)

Mean velocity

![Mean velocity graph](image)

Turbulence

![Turbulence graph](image)
Waterlevel, $H = 10$ cm, $T = 1.50$ s, $X = 4.50$ m, $z = 9$ cm

Mean velocity

Turbulence
Waterlevel, $H = 10$ cm, $T = 1.50$, $X = 4.65$ m, $z = 7.5$ cm

Mean velocity

Turbulence
Stability of stones in the surf zone

Water level, \( H = 10 \text{ cm}, T = 1.50 \text{ s}, X = 4.72 \text{ m}, z = 6.8 \text{ cm} \)

Mean velocity

Turbulence

II.24
Waterlevel, $H = 10 \text{ cm}$, $T = 1.50 \text{ s}$, $X = 4.78 \text{ m}$, $z = 6 \text{ cm}$

**Mean velocity**

**Turbulence**
Waterlevel, $H = 14 \text{ cm}$, $T = 1.34 \text{ s}$, $X = 4.20 \text{ s}$, $z = 0 \text{ cm}$

Mean velocity

Turbulence
Waterlevel, H = 14 cm, T = 1.34 s, X = 4.52 m, z = 8.8 cm

Mean velocity

Turbulence
Waterlevel, $H = 14$ cm, $T = 1.34$ s, $X = 4.54$ m, $z = 9$ cm

Mean velocity

Turbulence

II.38
Waterlevel, \( H = 14 \text{ cm}, T = 1.34 \text{ s}, X = 4.55 \text{ m}, z = 8.5 \text{ cm} \)

Mean velocity

Turbulence
Waterlevel, $H = 14$ cm, $T = 1.34$ s, $X = 4.57$ m, $z = 7.5$ cm

Mean velocity

Turbulence

SD $<u>$  -------- SD $<w>$
Appendix III

Sensitivity analysis of the models

Static model

To analyse the sensitivity of the β-d-equations with respect to the values of the coefficients, each time one of these values is changed compared to the basic situation. In Table III.1 these values are listed for the basic situation and 6 variants.

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</table>

Table III.1: Values of the coefficients for the sensitivity analysis

In Figure III.1 β-d/d_0-relations are plotted for the six variants with in each plot the basic situation. d_0 is the stone size following from horizontal equilibrium for the basic situation (where β=0°). The obvious kinks in the graphs arise because the relations consist of two equations. The kink indicates the transition between these equations. A description of the results of the sensitivity analysis is listed below.

1. Drag and down force increased by a factor 1.5. For the horizontal as well as for the moment equilibrium the required stone size increases for small β, because of the increased drag force. For larger β the influence of the down force increases, leading to a smaller stone size.

2. Lift force increased by factor 1.5. Both for the horizontal and the moment equilibrium the stone size increases for 0°<β<90°. The rise in stone size increases for large β, because the influence of the lift force increases for large angles of incidence.

3. a increased by factor 1.5. The stone size for horizontal equilibrium does not diverge from the basic situation (as expected). The stone size for moment equilibrium increases for all angles. The rise in stone size increases for large β, because the influence of the lift force increases for large angles of incidence.

4. Rotation point shifted upwards. This only influences the arm of moment for the drag force. The drag force decreases with increasing β, so the difference between the basic situation and variant 4 decreases with increasing β.
5. $\mu$ decreased. This coefficient is the only one which is known within a narrow margin -0.6-0.8. $\mu$ does not influence the moment equilibrium and has only little influence on the horizontal equilibrium.

6. $C_{rock}$ increased. The required stone size increases, because the jet directly hits a larger part of the stone.

Figure III.1: Variations on the basic situation
Simplified static model

To analyse the sensitivity of the β-d equations of the simplified model for the values of the coefficients, each time one of these values is changed with respect to the basic situation. In Table III.2 these values are listed for the basic situation and 6 variants.

<table>
<thead>
<tr>
<th>Coefficients</th>
<th>basic</th>
<th>variant 1</th>
<th>variant 2</th>
<th>variant 3</th>
<th>variant 4</th>
<th>variant 5</th>
<th>variant 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_d</td>
<td>1</td>
<td>1.5</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>C_l</td>
<td>2</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>a</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.75</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>b</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.75</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>μ</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
<td>0.7</td>
</tr>
<tr>
<td>C_rock</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table III.2: Values for the coefficients for the sensitivity analysis (simplified model).

In Figure III.2 β-d/d₀-relations are plotted for the six variants with in each plot the basic situation. d₀ is the stone size following from horizontal equilibrium for the basic situation (where β=0°).

A description of the results of the sensitivity analysis is listed below.

1. Drag and down force increased by a factor 1.5: For the horizontal as well as for the moment equilibrium the required stone size increases for small β (β<45°/50°), because of the increased drag force. For larger β, the influence of the down force increases, leading to a smaller stone size.

2. Lift force increased by factor 1.5: Both for the horizontal and the moment equilibrium the stone size increases for β>0°. The rise in stone size increases with increasing angle of incidence, because the influence of the lift force increases with increasing β.

3. a increased by factor 1.5. The stone size for horizontal equilibrium does not diverge from the basic situation (as expected). The stone size for moment equilibrium increases for β<0°.

4. b increased by factor 1.5. The stone size for the horizontal equilibrium does not diverge from the basic situation. The stone size calculated from the moment equilibrium increases for β<90°, and the rise first slightly increases and for larger β decreases. The term (sinβ+cosβ)-cosβ in the relation for F_D is responsible for this development. Compared to variant 3, the increase in stone size is smaller for variant 4.

5. μ decreased. A decrease in μ only has minor influence on the horizontal equilibrium.

6. C_rock increased. The required stone size increases, because the intrusion of the jet in the stone layer increases.

III.3
Figure III.2: Variations on the basic situation (simplified model)
Appendix IV

Wave generation

Waves were generated by a hydraulically driven piston-type wave board. In Figure IV.1 a schematic representation is given of the connections between the control-PC and the wave board controller.

![Schematic diagram of wave generation](image)

**Figure IV.1:** Schematic representation of wave generation

The program STIR of the software package AUKE/pc was used to generate the control signal for the wave board. An example of a control file of STIR is given below:

```
** The control file
data-stir,r6s1d55
** -----------------------------
** The wave board
wave-board,tud
biessel,off
transform,yes
compensation,on
time-step,120,Hz
depth,0.55
** -----------------------------
** The wave spectrum
wavetype,regular
height,0.07
period,1.96
end:wavetype
** -----------------------------
sound,first
subharm,yes
superharm,yes
modulation,no
** -----------------------------
```

IV.1