Cobble Sea Defence: Hydraulic Interface Stability of Sand underlying a Single Filter Layer

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Preface

A cobble sea defence appears to be an easy constructible protection, with relatively low total costs of ownership. Flume tests show that sand under such single layered constructions is stable. This report describes a research to the hydraulic stability of sand underlying a single filter layer.

The thesis is written for the completion of the Hydraulic Engineering MSc program at the faculty of Civil Engineering and Geosciences of the Delft University of Technology. I would like to thank the engineering department of Boskalis, Hydronamic bv, which gave me the opportunity to do my graduation within the company and provided me with equipment and a place to work. Furthermore I would like to thank PUMA (a joint-venture of Boskalis and Van Oord). Without their input this research wouldn’t have been possible. Besides advice and helpful comments, they supplied test results and measurements of a large-scale flume test to the behaviour of cobble beaches. Also, I would like to thank the Braunschweig University of Technology which was willing to supply the measurements and results of a large-scale flume test to the performance of Elastocoast revetments. Of course I would like to thank the manufacturer of Elastocoast, BASF, as well.

Special thanks go to Gerard Loman (PUMA) and Jelle Olthof (Hydronamic bv) for their effort in starting up this MSc thesis topic and providing advice and comments along the research process. Finally, I would like to thank the committee members of the Delft University of Technology: Prof.dr.ir. Marcel Stive, Prof.dr.ir. Wim Uijtewaal and Ir. Henk-Jan Verhagen. Besides helpful critics, their enthusiasm for hydraulic engineering made it a pleasure to work with them.

Arthur Zoon,

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Papendrecht.
Hydraulic interface stability of sand underlying a single filter layer
Summary

The design of the ‘Maasvlakte 2’ includes a cobble sea defence as a transition between the hard sea defence and the soft sea defence. The key design feature is the use of a single thick layer of cobbles (broad graded quarry stones 20/135 mm) covering a sand core, and functioning as a dynamic sea defence. The cobbles function as a geometrically open filter.

Flume tests show that the sand underlying a thick single filter layer is stable. However, it is not exactly known how the hydraulic loading of a breaking wave is reducing in the filter. Two datasets containing pressure measurements in a revetment were available for identification of this reduction:

- A dataset of tests in the Delta Flume, for the verification and optimization of the cobble shore design of the 'Maasvlakte 2' (Loman, 2009d).
- A dataset of a test performed in the Großer Wellenkanal (Hannover), for improvement of the understanding of all relevant processes in Elastocast revetments (obtained from the Braunschweig University of Technology).

Analysis of these datasets may result in an explanation for the stable interface of sand underlying a single filter layer and furthermore may provide information about the influence of properties (thickness, grading) on the performance of such constructions.

Turbulence describes the fluctuations of pressure and velocity. Literature assigns turbulence a significant role in particle transport under a turbulent open-channel flow:

- Velocity fluctuations can have the same order as the average velocity.
- Vertical fluctuations are involved in the entrainment process.

Fluctuations resulting from turbulent open-channel flow can be reduced by filters. A decrease of velocity and pressure fluctuations is achieved particularly in the upper part of the filter layer \((< 5D_{f/50})\). Whether fluctuations decrease so efficiently under the turbulent character of wave loading and breaking waves as well is uncertain. Because the turbulence under a breaking wave is much heavier than under a turbulent open-channel flow it is expected that turbulence generated by breaking waves has an important role in the hydraulic loading that eventually reaches the interface between cobbles and sand.

The dataset of the Großer Wellenkanal model is used to get a good understanding of the reduction of pressures, pressure fluctuations and gradients. The revetment of the model exists out of a bounded and an unbounded layer of similar material and can therefore be seen as a fixed single layered revetment.

Hydraulic loading at the top of a revetment exist out of two types of loads; impact and non-impact loads. The impact load can be distinguished in an impact part and a quasi-static part:
The bounded layer reduces the impact part by 40%. The unbounded layer reduces the remaining 60%. The quasi-static part of the impact load is not reduced by the bounded layer, but reduces by 35% by the unbounded layer. A similar reduction was observed for the non-impact load. The non-impact load and quasi-static part of the impact load are therefore described as penetrating loads. By increasing the filter thickness, an increasing reduction of penetrating loads is achieved.

Instead of the large influence that was expected for high frequent fluctuations (turbulence), they are only a fraction of the total occurring pressure. The fluctuations decrease by the influence of the filter and are not able to play a significant role in the hydraulic loading below the unbounded layer of the Großer Wellenkanal. Spectral analysis of the pressure measurements confirms the reduction of pressures by the bounded and unbounded layer. The impact peaks and high frequent fluctuations are damped by the porous material; the resulting spectral estimates are shifted to lower values of power and frequency respectively.

The parallel gradients at the interface are only the result of penetrating pressures. Maximum upward (‘destabilizing’) perpendicular gradients occur during run-down. The upward perpendicular gradients often appear simultaneously along the revetment. In almost all deviating cases, the perpendicular upward gradient first occurs on the lower part of the revetment, followed shortly by the middle part of the revetment and finally the higher part of the revetment. At every location, the maximal destabilizing perpendicular gradients are frequently accompanied by a maximal parallel gradient. The main loading mechanism at the interface results from the run-down; during run-down the largest parallel and perpendicular gradients are generated.

The revetments of the Delta Flume models exist out of a single unbounded filter layer of comparable material as the revetment of the Großer Wellenkanal model. A similar behaviour for pressure reduction could therefore be expected, but it appears that the revetment in the Großer Wellenkanal is able to reach an efficient damping relatively fast in comparison with the revetments of the Delta Flume models. This can partly be attributed to the fixation of the revetment in the Großer Wellenkanal model; the bounded layer prevents the unbounded material to move around. This results in a larger resistance against the wave action and thus a more efficient reduction of pressures than in a gravel revetment that has the freedom to find its dynamic equilibrium.

Predicted gradients acting on the interface between gravel and sand in the Delta Flume models are higher than traditional stability criteria. Therefore, an erosion process of the sandy embankment would be expected for the tested Delta Flume models. With observance of the direction and duration of the gradients from the Großer
Wellenkanal tests, an erosion pattern as was documented by Uelman (2006) and Ockeloen (2007) would be likely:

Such an erosion profile was not observed. The stable interface in the Delta Flume tests can thus not be explained from the pressure reduction resulting from the analysis of the measurements. The damping observed in the Großer Wellenkanal and the non-distorted interface in the Delta Flume models give strong indications that the reduction observed in the gravel material of the Delta flume is not representative for the performance of the gravel layer, resulting in a conservative prediction of gradients at the interface of cobbles and sand.

Several factors could have been influencing the pressures in the revetments of the Delta Flume models and the reduction found in the analysis of the measurements:

- An irregular wave climate has been used; two types of load subjected to the models.
- Hydraulic conditions differ for every test step.
- The low frequency of the pressure transducers makes discrimination between impact and non-impact loads not possible, which may result in wrong impression of reduction values.
- Model configurations of series are different and thereby also the location of the pressure transducers.
- Varying filter layer thickness including varying amounts of material on top of the upper transducer for each test.
- Wall effects; pressure transducers are mounted to the wall of the flume.

Extrapolation of the pressure reduction that is achieved by the revetment of the Großer Wellenkanal shows that a filter layer thickness of $20D_{50}$ should result in almost a complete reduction of pressures. Such a reduction will result in a decrease of the size of gradients as well. Comparing this to the filter layer thickness of the Delta Flume model configurations ($y/D_{50} > 40$), explains the stable interface in these models. An increasing reduction of penetrating pressures results in a reduction of gradients and explains the stable interface of sand underlying a single thick cobble layer.

The influence of grading could not be identified from the results of the tests that were available for analysis. The literature study reveals that the average velocity and the standard deviation decrease in a less accessible pore. In wide graded material pores are generally less accessible than in small graded materials, because smaller particles fill up the larger pores. Further research is necessary for a better identification of the influence of grading.
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<th>Unity</th>
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<tbody>
<tr>
<td>( A )</td>
<td>cm(^2)</td>
<td>Exposed surface area</td>
</tr>
<tr>
<td>( A_e )</td>
<td>cm(^2)</td>
<td>Erosion area</td>
</tr>
<tr>
<td>( c )</td>
<td>m/s</td>
<td>Wave velocity</td>
</tr>
<tr>
<td>( C )</td>
<td>-</td>
<td>Coefficient of proportionality</td>
</tr>
<tr>
<td>( C_u )</td>
<td>-</td>
<td>Coefficient of uniformity</td>
</tr>
<tr>
<td>( D )</td>
<td>m</td>
<td>Grain size</td>
</tr>
<tr>
<td>( D_{bx} )</td>
<td>m</td>
<td>Grain size base material of which X% of the mass of the grains has a smaller diameter</td>
</tr>
<tr>
<td>( d_f )</td>
<td>m</td>
<td>Thickness of the filter</td>
</tr>
<tr>
<td>( D_{fx} )</td>
<td>m</td>
<td>Grain size filter material of which X% of the mass of the grains has a smaller diameter</td>
</tr>
<tr>
<td>( e )</td>
<td>m</td>
<td>Exposure (stone height compared to local upstream bed level)</td>
</tr>
<tr>
<td>( f )</td>
<td>Hz</td>
<td>Frequency</td>
</tr>
<tr>
<td>( F )</td>
<td>N</td>
<td>Force</td>
</tr>
<tr>
<td>( g )</td>
<td>m/s(^2)</td>
<td>Gravitational acceleration (= 9.81 m/s(^2))</td>
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<tr>
<td>( h )</td>
<td>m</td>
<td>Water depth</td>
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<tr>
<td>( H )</td>
<td>m</td>
<td>Wave height</td>
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<td>( H_b )</td>
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<td>Wave height at which the waves start to break</td>
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<td>( H_{m0} )</td>
<td>m</td>
<td>Significant wave height (frequency domain)</td>
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<td>( H_s )</td>
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<td>Significant wave height (time domain)</td>
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<td>( p' )</td>
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<td>Fluctuating part of pressure</td>
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<td>kPa</td>
<td>Maximum pressure between peak and trough</td>
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<td>Pressure transducer</td>
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<td>( Q )</td>
<td>m(^3)/s</td>
<td>Discharge</td>
</tr>
<tr>
<td>( R )</td>
<td>m</td>
<td>Hydraulic radius</td>
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<td>( Re )</td>
<td>-</td>
<td>Reynolds number</td>
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<td>Relative depth-averaged turbulence intensity in channel flow</td>
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<td>( r_{0,f} )</td>
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<td>Mean relative turbulence intensity in granular filter</td>
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<td>Wave run-down height exceeded by 2% of the incoming waves</td>
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<td>m</td>
<td>Wave run-up height exceeded by 2% of the incoming waves</td>
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<td>Still water level</td>
</tr>
<tr>
<td>( t )</td>
<td>s</td>
<td>Time</td>
</tr>
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<td>( T_{m-1.0} )</td>
<td>s</td>
<td>Spectral period</td>
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<td>( T_p )</td>
<td>s</td>
<td>Peak period</td>
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<td>( u )</td>
<td>m/s</td>
<td>Flow velocity in horizontal direction</td>
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<td>( \bar{u} )</td>
<td>m/s</td>
<td>Mean part of velocity</td>
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<td>( v )</td>
<td>m/s</td>
<td>Flow velocity in vertical direction</td>
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<td>Variation coefficient representing the non-uniformity of the bed material</td>
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<tr>
<td>( V_{GF} )</td>
<td>-</td>
<td>Variation coefficient representing the non-uniformity of the filter material</td>
</tr>
<tr>
<td>Symbol</td>
<td>Unit</td>
<td>Description</td>
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<td>------------</td>
<td>-----------------------------------------------------------------------------</td>
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<td>$z_{p_{\text{max}}}$</td>
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<td>Predicted point of impact</td>
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<td>m</td>
<td>Vertical distance to profile from reference point</td>
</tr>
<tr>
<td>$z_{\text{SWL}}$</td>
<td>m</td>
<td>Vertical distance to location SWL from reference point</td>
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<tr>
<td>$\alpha$</td>
<td>-</td>
<td>Slope steepness</td>
</tr>
<tr>
<td>$\alpha_k$</td>
<td>-</td>
<td>Coefficient</td>
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<tr>
<td>$\gamma_b$</td>
<td>-</td>
<td>Influence factor for a berm</td>
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<tr>
<td>$\gamma_f$</td>
<td>-</td>
<td>Influence factor for roughness elements on a slope</td>
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<td>$\gamma_\beta$</td>
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<td>Influence factor for oblique wave attack</td>
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<td>Under water relative density</td>
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<td>$\xi$</td>
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<td>Surf similarity parameter</td>
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<td>Breaker parameter related to the spectral period ($T_{m-1.0}$)</td>
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<td>$\Pi_\Pi$</td>
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<td>Protrusion of particle above mean bed level</td>
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<td>$\rho$</td>
<td>kg/m$^3$</td>
<td>Density</td>
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<tr>
<td>$\sigma_p$</td>
<td>Pa</td>
<td>Standard deviation of pressure signal</td>
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<td>N/m$^2$</td>
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<td>$\tau_0$</td>
<td>N/m$^2$</td>
<td>Bed shear stress</td>
</tr>
<tr>
<td>$\tau_{bf}$</td>
<td>N/m$^2$</td>
<td>Shear stress at the interface of the filter and base material</td>
</tr>
<tr>
<td>$\nu$</td>
<td>m$^2$/s</td>
<td>Kinematic viscosity of water</td>
</tr>
<tr>
<td>$\chi$</td>
<td>-</td>
<td>Damping parameter</td>
</tr>
<tr>
<td>$\Psi$</td>
<td>-</td>
<td>Shields parameter</td>
</tr>
<tr>
<td>$\nabla$</td>
<td>1/m</td>
<td>Gradient</td>
</tr>
</tbody>
</table>

**Subscripts**

- **3D**: Three dimensional
- **b**: Base
- **c**: Critical
- **d**: Drag
- **f**: Filter
- **i**: Impact
- **l**: Lift
- **n**: Non-impact
- **p**: Pore
- **q**: Quasi-static
- **s**: Sediment
- **t**: Time
- **k**: Characteristic
- **w**: Water
- **x**: Parallel direction
- **y**: Perpendicular direction
1 Introduction

The design, construct and maintenance (DCM) contract of the ‘Maasvlakte 2’ project, awarded by the Port of Rotterdam in 2008 to PUMA (a joint-venture of Boskalis and Van Oord), includes an innovative design of a cobble sea defence section as a transition between the wide-footed southern sand sea defence and the small-footed northern hard sea defence (Figure 1.1). The key design feature is the use of a single thick layer of cobbles (broad graded quarry stones 20/135 mm) covering a sand core, and functioning as a dynamic sea defence (Loman 2009a). Because of the oblique wave conditions along this ‘Maasvlakte 2’ sea defence section, this cobble shore has to be recharged annually.

The above cobble shore design has been verified by PUMA making use of 2D and 3D physical scale modelling. The integral design has demonstrated that man-made cobble shores can be selected as a sea defence with the lowest total cost of ownership. With a view to the application else, Boskalis has initiated the underlying research study to develop calculation rules for predicting the dynamic interface stability between the cobbles and sand.

The cobbles function as a geometrically open filter. Information about interface stability of sand underlying geometrically open filters is limited, especially for situations with cyclic loading or even more turbulent situations due to (breaking) waves. There is some information about the erosion development under a geometrically open filter and the influence of parameters (like wave height, wave period and filter layer thickness) on this process. However, a good description of what happens in the filter with relevant parameters like velocity/pressure fluctuations (turbulence) and the influence of filter properties on the stability of the interface under wave loading conditions is missing. This will be the focus of this MSc graduate study.

The main interest goes to the stability of the interface below a thick single layered filter on sand in a very turbulent loading situation. Datasets of two large scale physical models tests are available for analysis; Delta Flume tests and a test performed in the Großer Wellenkanal (Hannover). The first were carried out for PUMA by Deltares | Delft Hydraulics for the verification and optimization of the cobble shore design of the ‘Maasvlakte 2’ (Loman, 2009d). The later was executed by the Braunschweig University of Technology for improvement of the understanding of all relevant processes and the creation of reliable and practical formula/diagrams, which may be applied for design purposes of...
Elastocoast revetments. Although both models were build with a different purpose than for this MSc thesis research, they provide very useful information.

The problem description, problem definition and approach are located in chapter 2. In chapter 3 the Delta Flume models and the Großer Wellenkanal model are described. Chapter 4 includes the literature study. In chapter 5 and 6 the analysis of available datasets is documented, after which in chapter 7 conclusions and recommendations are given.
2 Problem Analysis and objective

The results of the Delta Flume tests carried out for PUMA proved that the current available knowledge regarding particle stability under a geometrically open filter layer is not fully understood.

2.1 Problem description

The Delta Flume tests show that accordingly the verification criterion the intrusion of sand from the core into the cobble layer is negligible during design storm conditions.

Remarkably, this measured hydraulic interface stability is not fully in line with the 'instable interface' predicted by currently available calculation rules applied to the measured high pressure gradients (Loman, 2009b).

In their desk study Deltares| Delft Hydraulics expects that sand will move under an open filter subjected to storm conditions. This is concluded after verification with critical gradients found in literature, mostly related to permanent current attack, which have been exceeded in the tests. Possibly, the high gradients in the Delta Flume are only present during a very short period of time. In this respect gradients can better be described as fluctuations and may be not even capable of influencing the stability of particles at the interface in such a short time.

The use of literature about permanent current attack maybe is allowed in situations where a clear run-up and run-down take place when these flows have enough time to develop for a fair comparison with situations under permanent current attack. It is questionable if this is true for the Delta Flume tests. The loading mechanism at the interface (between the filter and core) of a breakwater is probably also influenced by turbulence generated by breaking waves and the impact of plunging waves. This makes the comparison between critical gradients under permanent current attack with the derived gradient fluctuations by Deltares somewhat doubtful. On the other hand, it is not understood to what extend these processes penetrate through a filter and how they can affect the interface stability.

2.2 Problem definition

There is a lack of knowledge in the loading process at the interface of a sandy core and a single filter layer which is geometrically open as a result of uncertainty in the reduction of highly turbulent loading by a filter layer and the influence of filter properties on the reduction, which makes an optimization of single layered constructions impossible.

2.3 Objective

The problem description, problem definition and the availability of datasets make it possible to formulate the following objective:

Identify the reduction of hydraulic loading under a breaking wave by a filter, get insight in the influence of different filter properties on this reduction and explain the stable interface of sand underlying a cobble layer.

In the Delta Flume models, the cobbles have been scaled to gravel. In this report the interface stability of sand underlying a gravel layer will therefore actually be investigated. In the objective, ‘a cobble layer’ could also have been replaced by ‘a gravel layer’.
2.4 Approach

Two datasets containing pressure measurements in a revetment are available for identification of reduction of hydraulic loading:

- A dataset of tests in the Delta Flume, for the verification and optimization of the cobble shore design of the ‘Maasvlakte 2’ (Loman, 2009d).
- A dataset of a test performed in the Großer Wellenkanal (Hannover), for improvement of the understanding of all relevant processes in Elastocoast revetments (obtained from the Braunschweig University of Technology).

The dataset of the Delta Flume pressure measurements is not likely to be an appropriate and reliable basis to encounter the problem that has been described. The pressure transducers are simply located to arbitrarily within the different profiles with the result that the breaking impact of the waves could be underestimated and significant processes can be overlooked. A more suitable dataset, in which breaking impact is measured guaranteed, can be found in the large scale model test to the behaviour of Elastocoast that was performed in the Großer Wellenkanal in Hannover (Germany) by the Leichtweiß-Institut für Wasserbau (LWI) of the Braunschweig University of Technology. Conclusions from the analysis of these measurements can help in understanding the results of the analysis of the measurements of the Delta Flume models.

In summary, the approach will be as follows:

1. Literature study.
2. Analysis of pressure measurements performed in the Großer Wellenkanal model test.
3. Analysis of the pressure measurements performed in the Delta Flume model tests.
4. Conclusions about performance of a geometrically open filter revetment under wave loading conditions.
5. Explain the stable interface in the Delta Flume tests, using the results of the analyses.

In the next chapter the models that have been tested in the Delta Flume and the Großer Wellenkanal will be elaborated.
3 Description of models

In this chapter the physical model tests that are available for analysis are described. First, the Delta Flume models will be treated. Thereafter, the model of the Großer Wellenkanal model is elaborated. Model properties may have a large influence on the performance of the revetments. Because the results of the models will be used for the same purpose it is important that the differences in model properties are clear. The last paragraph of this chapter is dedicated to the identification of these differences.

3.1 Delta Flume model description

The Delta Flume is a test facility of Deltares| Delft Hydraulics. The flume has a length of 240 m, a width of 5 m and a depth of 7 m. These dimensions make it possible to test close to prototype scale, which reduces the occurrence of scale effects. Both regular and irregular waves can be generated by the wave generator which effectively eliminates re-reflections of waves from the wave board.

Because the transition between the wide-footed southern sand sea defence and the small-footed northern hard sea defence requires a changing profile of the cobble sea defence, several model configurations were tested by Deltares| Delft Hydraulics (Figure 3.1).

![Figure 3.1 - Top view of the transition between the southern sand sea defence and the northern hard sea defence (Loman, 2009a).](image)

The numbers in Figure 3.1 indicate the following:

1. The northern foreshore and backshore adjacent to the steep-sloped hard sea defence.
2. The southern backshore adjacent to the sand dune.
3. The southern foreshore adjacent to the flat-sloped sand coast profile.

The purpose of the physical model tests carried out was to verify the design of the configurations on the behaviour of cross-shore deformation and wave overtopping. All model variants existed out of a sandy embankment on which a filter layer is placed. The tested models differ in slope steepness and filter thickness. The cross sections of the models are given in Appendix A.2.

Within the layer thickness of the models the extra material for the loss of filter material by 3D effects (like longshore transport) was not taken into account. The cobbles are scaled on basis of the stability number (3.1) which is kept constant for both model and prototype.
The used filter material was broken gravel. The lower half of the filter was washed in with sand, but the amount of washed in sand is varied over the models. Measurements to waves, wave overtopping and pressures have been carried out for all tested models.

3.1.1 Pressure measurements
Within each tested configuration one set of three pressure transducers was located at the lower part of the revetment (PT4, PT5 and PT6). It was tried to position this set such that it was situated at the location where the most erosion of the filter layer would occur. Another set of three transducers was positioned somewhat higher at the slope (PT1, PT2 and PT3). For both sets one transducer was located in the upper half of the filter, one transducer was located in the lower half of the filter and one transducer was located 0.1 m within the sandy embankment. For every tested model configuration the exact position of the transducers was different. The transducers were mounted to the wall of the flume.

The pressure transducers were set to \( p = 0 \text{ kPa} \) before the start of every test. In that way it was possible to measure the pressures induced by wave motion only, excluding hydrostatic pressure resulting from water depth at still water level (SWL).

In total 6 series of 5 test steps were carried out, from which 4 series (Series 2-5) are appropriate for analysis is this thesis. In every series a different model configuration has been tested, in every test step a different hydraulic condition. The 5 test steps together simulate the hydraulic conditions of a design storm (1/10,000 year). The specification of the steps can be found in Appendix A.1.

A brief summary of material properties and configurations is presented in Table 3.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base material</td>
<td>( D_{b50} = 0.213 \text{ mm} )</td>
</tr>
<tr>
<td>Filter material</td>
<td>( D_{f50} = 14.6 \text{ mm} )</td>
</tr>
<tr>
<td></td>
<td>( D_{f50} / D_{b50} = 4 )</td>
</tr>
<tr>
<td></td>
<td>( \rho = 2640 \text{ kg/m}^3 )</td>
</tr>
<tr>
<td>( D_{f50} / D_{b50} )</td>
<td>68.54</td>
</tr>
<tr>
<td>Relative filter thickness</td>
<td>37.9 – 87</td>
</tr>
<tr>
<td>Wave height</td>
<td>0.55 m – 1.45 m</td>
</tr>
<tr>
<td>Period</td>
<td>3.88 s – 4.95 s</td>
</tr>
<tr>
<td>Slope steepness</td>
<td>1:4 – 1:10</td>
</tr>
</tbody>
</table>

From this table the large relative layer thickness and the very open particle size ratio \( (D_{f50}/D_{b50}) \) do stand out.
3.2 Großer Wellenkanal model description

A large scale model test to the behaviour of Elastocoast was performed in the Großer Wellenkanal in Hannover (Germany) by the Leichtweiß-Institut für Wasserbau (LWI) of the Braunschweig University of Technology.

3.2.1 Elastocoast material description

Elastocoast is a polyurethane which is able to bind crushed stones such that a strong and porous composite is created. It is developed by Elastogran GmbH. Elastocoast revetments are a new type of coastal revetment systems. More than 75 large-scale model tests were performed in the Großer Wellenkanal to improve the understanding of the physical processes involved in the wave-structure foundation interaction and to develop prediction formula for both hydraulic performance and wave loading. Results are documented in LWI Report no. 988 (2010).

3.2.2 Großer Wellenkanal model description

The Großer Wellenkanal in Hanover is about 300 m long and 5 m wide. The depth is 7 m and the wave generator (with an active absorption control) can create regular waves up to 2 m and wave spectra with significant wave heights up to 1.3 m. The cross section of the flume is given in Figure 3.3.

![Figure 3.3 - Cross-section of the Großer Wellenkanal (LWI Report no. 988, 2010).](image)

The Elastocoast revetment configuration is presented in Figure 3.4. The embankment was built out of sand. The foreshore consists of a sand bed with a slope of 1:20.

![Figure 3.4 - Layout of the profile of the test to Elastocoast revetments (LWI Report no. 988, 2010).](image)

Three model alternatives (A, B and C) were tested of which only two (B and C, Figure 3.5) will be used for the analysis in this thesis.
The filter layers below the Elastocoast layers exist out of the same crushed rock material as the Elastocoast revetment. ‘Filter’ is therefore not a good description of the unbounded material below the Elastocoast layer. Although the bounded top layer may have somewhat smaller pores, the total revetment can be treated as a single layered revetment. For both models a geotextile has been placed between the unbounded layer and the sandy embankment. Properties of models B and C are given in Table 3.2.

**Table 3.2 - Properties of Model B and Model C.**

<table>
<thead>
<tr>
<th></th>
<th>Model B</th>
<th>Model C</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Material type</strong></td>
<td>Crushed limestone (20/40 mm)</td>
<td>Crushed granite stones (16/36 mm)</td>
</tr>
<tr>
<td><strong>Bounded layer (Elastocoast)</strong></td>
<td>0.15 m</td>
<td>0.15 m</td>
</tr>
<tr>
<td><strong>Unbounded layer</strong></td>
<td>0.10 m</td>
<td>0.20 m</td>
</tr>
<tr>
<td><strong>Geotextile</strong></td>
<td>Terrafix 609</td>
<td>Terrafix 609</td>
</tr>
<tr>
<td><strong>Sand foundation</strong></td>
<td>( D_{50} = 0.34 ) mm</td>
<td>( D_{50} = 0.34 ) mm</td>
</tr>
</tbody>
</table>

Models B and C are built side by side, each covering half of the width of the flume (2 x 2.5m) and tested simultaneously using the same incident wave conditions. Figure 3.6 shows models A and B,
which were built side by side like the models B and C. Although several wave climates have been tested, only one will be used for the analysis in this report; an irregular wave climate with a significant wave height of $0.67 \text{ m}$ and a spectral period of $3.61 \text{ s}$.

![Image](image.png)

**Figure 3.6 - Front view on revetment A and B, with separation wall (LWI Report no. 988, 2010).**

### 3.2.3 Pressure Transducers

Several pressure transducers are installed in both model alternatives to measure the wave load on the surface of the slope revetment, underneath the bounded layer, at the bottom of the unbounded layer, the wave-induced pore pressure inside the embankment and the fluctuation of the internal water level.

In Model B, the pressure transducers on the surface of the revetment are placed such that the distribution of the wave load over the surface is measured. Seven transducers, with a distance of $\Delta x = 0.25 \text{ m}$ cover the impact area. One further transducer is located near the still water level and a last transducer is located $3 \text{ m}$ seaward (in $x$-direction) of the impact point for the measurement of the quasi-hydrostatic wave pressure. The number of transducers beneath the bounded revetment has been reduced to three.

In model alternative B a total of 23 pressure transducers were installed. One transducer (PT40) had been added just above the geotextile in order to measure the pressure damping in the bounded layer. Unfortunately, after the test it appeared that this transducer did not work correctly. Still, pressure measurements of this test are suitable for the analysis of the performance of the bounded layer.

In model alternative C a total of 23 pressure transducers were installed, analogous to the placement of Model B. This time the pressure transducers were mounted at the interface of the bounded and unbounded layer and between the unbounded layer and the geotextile. Model C is suitable for the analysis of the performance of the unbounded layer.
3.3 Similarities and differences between models

Looking at the models of the Elastocoast tests and the test to the behaviour of the cobble beach in the Delta Flume there are quite some differences. In Table 3.3 an overview is given.

<table>
<thead>
<tr>
<th></th>
<th>Delta Flume tests</th>
<th>Großer Wellenkanal test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size base material</td>
<td>$D_{50} = 0.214$ mm</td>
<td>$D_{50} = 0.340$ mm</td>
</tr>
<tr>
<td>Filter material type</td>
<td>Crushed river gravel</td>
<td>B: Crushed limestone</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C: Crushed granite stones</td>
</tr>
<tr>
<td>$\rho_f$</td>
<td>2640 kg/m$^3$</td>
<td>2600 kg/m$^3$</td>
</tr>
<tr>
<td>Filter grading</td>
<td>$D_{10} = 5.0$ mm</td>
<td>B: 20/40 mm</td>
</tr>
<tr>
<td></td>
<td>$D_{50} = 14.6$ mm</td>
<td>C: 16/36 mm</td>
</tr>
<tr>
<td></td>
<td>$D_{95} = 20$ mm</td>
<td></td>
</tr>
<tr>
<td>Filter thickness</td>
<td>0.6 m – 1.0 m</td>
<td>B: 0.15 m Elastocoast</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+0.10 m filter</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C: 0.15 m Elastocoast</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+0.20 m filter</td>
</tr>
<tr>
<td>Relative filter thickness</td>
<td>41 – 68</td>
<td>B: 8.3</td>
</tr>
<tr>
<td>$\left(\frac{d_f}{D_{50}}\right)$</td>
<td></td>
<td>C: 13.5</td>
</tr>
<tr>
<td>Slope</td>
<td>1.4 – 1.75</td>
<td>1:3</td>
</tr>
<tr>
<td>$H_s$</td>
<td>0.71 m – 1.44 m</td>
<td>0.67 m</td>
</tr>
<tr>
<td>$T_{m-10}$</td>
<td>3.88 s – 4.95 s</td>
<td>3.61 s</td>
</tr>
<tr>
<td>$H/\Delta D_{50}$</td>
<td>30 – 60</td>
<td>14 – 16.1</td>
</tr>
<tr>
<td>Frequency pressure</td>
<td>5 Hz</td>
<td>500 Hz</td>
</tr>
<tr>
<td>measurements</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The used filter material differs for all treated models in size and in grading. This probably will have influence on the performance of the filters. The different materials can possibly help in identifying the influence of filter properties on the reduction of pressures. There is a difference in filter layer thickness, which is emphasized by the relative value. This is expected to have a large influence on the pressure reduction. Also, the slope steepness can have a big influence on the behaviour of the waves on the revetment.

According to Table 3.3, the slope steepness in the Elastocoast models deviate from those in the Delta Flume models. The Elastocoast models are built on a uniform 1:3 slope. All models tested in the Delta Flume are milder. Only Series 3 is completely uniform, but because the filter material has the freedom to move under the influence of hydraulic loading this can change in the process of approaching its dynamically stable shape (Figure 3.7). Theoretically, the thickness of the filter can change with every wave that hits the model, introducing varying filter thickness along the slope.

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1 Not specifically mentioned in report, 2600 kg/m$^3$ is assumed.
This extra variable in the Delta Flume models makes the analysis of results somewhat more difficult. The Elastocoast models are statically stable; the filter thickness is constant, also during testing. The pore structure cannot change, in contrast with the pore structure in the models of the Delta Flume. This ‘self healing effect’ of the Delta Flume models might be an important variable for the performance of a filter.

Pressure measurements were performed in both model tests. However, the frequency of the data acquisition differs significantly (a factor 100). Also the locations of the pressure transducers are different. In the Großer Wellenkanal the transducers are situated on top of the revetment, between the bounded and the unbounded layer and below the unbounded layer. Whereas the transducers in the Delta Flume models are located within the material; one in the upper part and one in the lower part of the filter. In the Großer Wellenkanal the water level is adjusted such that the wave impact will always be at the location where measuring equipment is installed and as a result the impact of the breaking waves is always captured. The pressure transducers in the Delta Flume models are situated differently for all series. In addition, the water level has been adjusted for all tests to simulate the raising water level in a storm without taking the consequences for the impact location into account.

Regarding the pressure measurements, the Großer Wellenkanal model test has the following advantages with respect to the Delta Flume model tests when taking the objective into account:

- The model contains fewer variables.
- More pressure measurements are available.
- Data acquisition obtained with higher frequency.

There is a geotextile between the embankment and the filter of the Großer Wellenkanal model. Intrusion of sand in the filter was not possible. Nevertheless, the dataset of pressure measurements in the extensive model in the Großer Wellenkanal can help in the process of understanding the relative stability of the interface of sand underlying a filter layer which is geometrically open. Probably, the analysis of the results of this model will not directly be applicable to the Delta Flume models, but it can increase the knowledge of hydraulic processes in the filter and on the interface between a filter and a sandy embankment. Processes will be analyzed at model scale. No calculations to prototype scale will be part of this thesis.

Before the results of the analysis of the dataset of the Großer Wellenkanal model is given, first a literature study is carried out to physical processes that occur when a wave is approaching a breakwater. Additionally, a closer look is taken at the stability of sand and the effect of filters on pressures and velocities resulting from hydraulic loading.
4 Literature study

This chapter describes literature of several topics. First a closer look is taken at large scale physical processes, introduced by the hydraulic loading of waves at a breakwater. Thereafter the erosion process under a geometrically open filter of a breakwater is described on basis of MSc Theses of Uelman (2006) and Ockeloen (2007).

Small scale processes, like the forces that play a role in particle entrainment and incipient motion are described followed by the elaboration of the role of turbulence. Wave attack results in a turbulent behaviour of the hydraulic loading. The fast fluctuations of velocity and pressure make turbulence a chaotic and complex process. Turbulence is expected to play a significant role in particle transport.

A filter is able to reduce the turbulence. The last part of this chapter is dedicated to the quantification of reduction of turbulence, thereby discriminating between velocity fluctuations and pressure fluctuations. Also a closer look is taken at the influence of filter properties.

4.1 Large scale physical processes

In this paragraph the physical processes resulting from wave attack on a breakwater are described. The purpose of this is to get a comprehension of the large scale physical processes that occur.

A revetment subjected to wave attack can be distinguished in zones. In Figure 4.1 a distinction between three zones has been made; A, B and C. The arrows in the figure indicate that, depending on the wave loading conditions, the borders between the zones can shift.

![Figure 4.1 - Definition of zones.](image)

**Zone A**

The motion of the water particles in deep water is circular and will turn into a more elliptical water motion when the water depth reduces. In very shallow water the ellipses become flatter towards the bottom. In Figure 4.2 this is schematized.

![Figure 4.2 - The orbital motion in deep water, intermediate-depth and very shallow water (Holthuijsen, 2007).](image)

At the bottom the ellipse degenerates into a straight horizontal line. The motion of the water particles in zone A of the breakwater shows a lot of resemblance with the above described water motion above a horizontal bottom. The difference is that the water depth is decreasing relatively fast above a...
breakwater, which makes the water motion more complex. In addition, the shoaling effect causes increasing wave height and wave steepness.

Zone A starts at the toe of the breakwater and continues until the location where the waves start to break. When the waves become too steep, breaking will begin. The limits of both steepness and shallow water are described with the breaking criterion by Miche:

$$H_b = 0.142L \tan(h) \left( \frac{2\pi}{L} \right)^{-1}$$  \hspace{1cm} (4.1)

If $H \geq H_b$, breaking will occur. From this point on, Zone B has been reached. The location of zone B will therefore depend on the wave climate.

Zone B

In zone B waves are breaking. These breaking waves show a lot of chaotic, turbulent motion including entrapped air. The surf similarity parameter describes the breaker type. It is determined by the angle of the slope at the front face of the structure and the wave steepness:

$$\xi = \frac{\tan \alpha}{\left( \frac{H_s}{L_0} \right)}$$ \hspace{1cm} (4.2)

For different values of the surf similarity parameter waves break in a different way, as can be seen in Figure 4.3. When the breaking is of the plunging type ($\xi \approx 0.5 - 2.5$), an asymmetric crest develops which attacks the slope like a water jet. An impact load is the result which causes velocity and pressure fluctuations that will penetrate into the filter. How the impact load will be absorbed and what will happen with pore pressures and pore velocities going deeper in the filter is unknown. The transition between breaking and non-breaking is around $\xi \approx 2.5 - 3$, where a collapsing type of breaking appears. For $\xi > 3$ a surging type of wave is present on the slope and a non-impact load will be the result.
Zone C
Zone C represents that part of the slope where only run-up and run-down take place. Run-up \((R_u)\) and run-down \((R_d)\) describe the difference in height with the SWL (Figure 4.4).

![Figure 4.4 – Run-up and run-down.](image)

In Figure 4.5 the relative wave run-down \((R_{d2\%}/H_{m0})\) is plotted against the surf similarity parameter \((\xi_m-1)\). \(R_{d2\%}\) describes the wave run-down height exceeded by 2% of the incoming waves, which is generally used to describe wave run-down. This figure shows that the transition from Zone B to C is not really clear, because wave run-down reaches the area below SWL where the waves break as well (Zone B). Figure 4.5 is obtained from the LWI Report which describes results of the experiment in the Großer Wellenkanal. Only measurements of Model B and Model C (see Figure 3.5) are involved in the figure. It is clear that an increasing surf similarity parameter results in a larger relative wave run-down.

![Figure 4.5 - Wave run-down (adjusted from LWI Report no. 988, 2010).](image)

Wave run-up can be described in a similar way as wave run-down. \(R_{u2\%}\) describes the wave run-up height exceeded by 2% of the incoming waves. Instead of a cloud of data points like was presented for the relative wave run-down, three relations between the relative wave run-up \((R_{u2\%}/H_{m0})\) and the surf similarity parameter are plotted in Figure 4.6. For more information about these relations reference is made to EurOtop (2007). The role of roughness of the surface and the permeability of the core should not be underestimated. A larger relative wave run-up can be expected when the surf similarity parameter is growing. For waves of the surging type, which have the characteristics of a standing wave, the wave run-up will be the highest.
The three zones (A, B and C) are clearly divided by a different type of process (non-breaking, breaking, run-up and run-down), although run-up and run-down both affect zone B and zone C. In the next section the erosion process under a geometrically open filter of a breakwater is described, whereby an important role is assigned to run-down.

4.2 Erosion process under a geometrically open filter

Uelman (2006) and Ockeloen (2007) both carried out flume experiments on a geometrically open filter of a breakwater. These experiments give information of the variables that determine the transportation of sandy material out of the core into a very open granular filter under influence of wave load. Before going deeper into the erosion process under a filter, the difference between a geometrically open and a geometrically closed filter is described.

4.2.1 Geometrically open and closed filters

Filters can be either geometrically closed or geometrically open. In geometrically closed filters the largest grains in the base layer get stuck in the pores of the filter layer and block the passage of all other grains of the base layer (Schiereck, 2001):

\[
\frac{D_{f15}}{D_{b85}} < 5 \quad (4.3)
\]

The range of grain diameters in the base layer should be such that the larger grains can block the smaller ones (internal stability) (Schiereck, 2001):

\[
\frac{D_{b60}}{D_{b10}} < 10 \quad (4.4)
\]

To prevent instability due to uplifting, the filter layer should by sufficiently permeable (Schiereck, 2001):

\[
\frac{D_{f15}}{D_{b15}} > 5 \quad (4.5)
\]

In geometrically open filters (or hydraulic sand tight filters) base particles are able to travel through the filter provided that the hydraulic loading \(V_{f,act}\) is larger than some critical value \(V_{f,c}\). In Figure
4.7 A graph is presented in which the transition between a geometrically open and closed filter is given.

![Graph showing transition between geometrically open and closed filter]

**Figure 4.7 - Relation between different filter types (adjusted from CUR report 161).**

When the ratio between \( D_{f15} \) and \( D_{b50} \) gets larger the base layer can act like there is not filter at all. Then a lower limit of the critical gradient has been reached, which can result in fluidization (piping).

### 4.2.2 Erosion process

The research of Uelman (2006) gives a good description of the erosion process under a geometrically open filter. Ockeloen (2007) focused his research in trying to find relations for the influence of variations of hydraulic loading, slope steepness and grading of filter material on the stability and erosion patterns of core material. The research of Ockeloen (2007) was a continuation of the research of Uelman (2006). The model properties of both experiments are described in Table 4.1.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Base material</td>
<td>( D_{b50} = 0.160 \text{ mm} )</td>
<td>( D_{b50} = 0.180 \text{ mm} )</td>
</tr>
<tr>
<td>Filter material</td>
<td>( D_{f50} = 1.80 \text{ cm} ) ( -4.40 \text{ cm} )</td>
<td>( D_{f50} = 2.60 \text{ cm} )</td>
</tr>
<tr>
<td></td>
<td>( D_{f50} / D_{f15} = 4 )</td>
<td>( D_{f50} / D_{f15} = 1.3 - 3.5 )</td>
</tr>
<tr>
<td>Filter thickness</td>
<td>10 cm – 20 cm</td>
<td>15 cm</td>
</tr>
<tr>
<td>Relative filter thickness</td>
<td>2.3 – 11.1</td>
<td>5.77</td>
</tr>
<tr>
<td>Wave loading</td>
<td>Regular:</td>
<td>Irregular:</td>
</tr>
<tr>
<td></td>
<td>0.10 m</td>
<td>0.08 m – 0.14 m</td>
</tr>
<tr>
<td>Wave period</td>
<td>1.2 $s$</td>
<td>1 $s$ – 2 $s$</td>
</tr>
<tr>
<td>Slope steepness</td>
<td>1:3</td>
<td>1:2 – 1:4</td>
</tr>
</tbody>
</table>
Both the researches contain a description of the development of erosion. Relevant conclusions regarding the erosion process made by Uelman (2006), which have a quantitative character are:

- “A curved profile develops in the sand which looks like a bar profile of a sandy beach.”
- The transport of the core material is governed by sheet flow (bed) transport and suspension transport. As the filter gets thicker, a larger part of the total transport is bottom transport. As the grain size gets smaller, a larger part of the total transport is bottom transport.
- At the location where net erosion takes place, the sand is mainly transported by bottom transport. Only with large grain size of the filter material or a thin filter layer a small part of the transport is suspension transport. During wave run-up there is less transport of sand then during wave run-down, also a larger part of the transport is suspension transport.
- The total erosion, erosion depth\(^2\), and the erosion length\(^3\) after 2400 waves are dependent on \((d_f/D_{50})\). With an increasing \((d_f/D_{50})\), the total erosion, erosion depth, and the erosion length are decreasing.
- For all experiments the erosion rate is decreasing in time. But after 2400 waves the erosion has not become zero.”

A relevant conclusion regarding the erosion process made by Ockeloen (2007):

- “The amount of erosion grows with the square-root of the number of waves. An equilibrium state has not been found during the relatively long tests, but a gradual decrease of the erosion rate is evident.”

Representative, schematic ‘before’ and ‘after’ profiles are presented in Figure 4.8.

The quoted conclusions of Uelman (2006) show how the erosion develops and the influence of the relative filter thickness on the type of transport below the filter. Both theses notice that the erosion rate is decreasing in time, but without finding an equilibrium state.

---

\(^2\) The erosion depth is defined as the depth of the erosion hole.

\(^3\) The erosion length is the length of the erosion hole.
When looking at the movement of water occurring in the model, Uelman (2006) gives the following description:

“The main wave is moving up and down the breakwater. Water is mainly moving up the slope on the outside of the filter layer and the water is mainly moving down the slope through the filter layer.”

Ockeloen (2007) describes the same water movement in downward direction, but as a function of the hydraulic gradient:

“Parallel downward porous flow is the dominant loading process rather than turbulence by the breaking waves. The turbulence has been observed to die out in the upper part of the filter layer. Porous flow has been observed to develop during the wave run-down, picking up sand when the velocity is high enough.

The parallel downward porous flow is driven by the hydraulic gradient that sets in from the internal setup- or run-up level to the external run-down level. The hydraulic gradient is a complex function of wave parameters (height, period, run-up, run-down, steepness) and structure parameters (filter grain size, thickness, porosity and slope steepness).”

In the description of zone C (section 4.1) it became clear that run-up and run-down show a relation with the surf similarity parameter, which in turn is a function of the wave height, period and the slope steepness. By keeping the wave climate (period and height) constant, the influence of the slope steepness is analyzed by Ockeloen (2007). The experiments show that milder slopes show less erosion under the same hydraulic loading conditions. Although only based on three points, Ockeloen (2007) gives a linear relation between slope steepness and erosion area. The erosion area is defined as the area between the original sand slope and the eroded sand slope in the part of the construction where erosion takes place. The linear relation can be extrapolated to the slope where hypothetically no erosion would occur (indicated by the green line in Figure 4.9). The result is a slope steepness of 1:8.5.

![Erosion area vs tan(α) variation slope steepness](image)

Figure 4.9 - Relation between erosion area and slope steepness (Ockeloen, 2007).

Ockeloen (2007) tried to give some explanations for the increasing erosion with increasing slope steepness:

- “The energy of a breaking wave is dissipated over a shorter distance on a steeper slope.”
• The horizontal component of the filter layer thickness is much smaller for a 1:2 slope, than for a 1:4 slope (Figure 4.10).

Figure 4.10 - Horizontal component of filter layer thickness (B > A).

• A steep slope has less resistance against sliding down than sand on a milder slope."

In addition to the listed explanations of Ockeloen it is known from run-up and run-down relations that both reduce with decreasing slope steepness under the same hydraulic wave conditions. Reduced slope steepness results in a lower surf similarity parameter under constant hydraulic conditions (Figure 4.5 and Figure 4.6).

The results from Ockeloen show that at a milder slope, the wave action does not seem to reach high enough to cause a sufficiently strong porous flow running down. Wave run-down was determined as an important cause of the transport of material from the upper slope to the lower slope, creating a step berm profile.

The erosion under a geometrically open filter at a steep slope of a breakwater is mainly governed by the porous flow running down the slope. Porous flow has been observed to develop during the wave run-down, picking up sand when the velocity is high enough. A milder slope results in a smaller hydraulic gradient and thus a weaker run-down flow.

The slopes of the Delta Flume models are milder compared to the slopes that were tested in the described experiments of Uelman (2006) and Ockeloen (2007). Extrapolation of measurements found by Ockeloen (2007) in Figure 4.9 resulted in a hypothetical point of no erosion when a slope is 1:8.5 or milder. The existence of such a point is questionable, but the reduction of erosion by milder slopes is obvious. Reduction of run-up and run-down height causes a smaller hydraulic gradient, which results in a reduced parallel downward porous flow.

After the Delta Flume tests it was noticed that the sandy embankment was not changed in volume and shape. Apparently, for the Delta Flume model tests porous run-down flow is not able to initiate such an erosion process as was occurring in the small scale flume tests of Uelman (2006) and Ockeloen (2007). Although breaking was observed to die out for the hydraulic conditions used in the described small scale flume tests, it is not likely that turbulence dies out so rapidly for the hydraulic conditions subjected to the model in the Delta Flume.

In the next section a closer look is taken at small scale processes; particle stability and the entrainment process. Thereby also taking the role of fluctuations of pressure and velocity into account, which are the result of the breaking waves.
4.3 Small scale physical processes

Particles are being transported when the load (shear stress) is larger than the strength. The bed can be considered stable when the load is less than some critical value. When the critical value is being exceeded incipient motion will occur. Small scale processes describe the forces that work on one single particle.

4.3.1 The drag and lift force

A turbulent flow on a horizontal bed causes different forces on a particle. In Figure 4.11 these forces are presented graphically.

![Figure 4.11 - Forces on a stone (Hofland, 2005).](image)

The forces on the grain can be expressed as follows:

Drag force:

\[ F_d = \frac{1}{2} C_d \rho_w u^2 A_d \]  

(4.6)

Lift force:

\[ F_l = \frac{1}{2} C_l \rho_w u^2 A_l \]  

(4.7)

The lift force is mainly caused by the curvature of the flow around a stone and is balanced by the submerged weight of the particle \( F_g \). Drag force is balanced by the stabilizing moment around the pivot point of the relevant stone. \( C_d \) and \( C_l \) are coefficients and dependent on the flow type (laminar/turbulent).

4.3.2 Incipient motion

Shields performed experiments for incipient motion of uniform granular material on a flat bed. Thereby not relating movement to the forces indicated in Figure 4.11, but to the shear stress as the active force. The result is presented in Figure 4.12.
Hydraulic interface stability of sand underlying a single filter layer

Figure 4.12 - Shields diagram; $\Psi$ as function of $D$, $\left( = \frac{\Delta g}{\bar{u}^{2}}^{0.33} \right)$ (Hoffmans et al., 2008).

The well known Shields formula is as follows:

$$\Psi_c = \frac{\tau_c}{(\rho_s - \rho_w)gD} = \frac{u_c^2}{\Delta gD} \quad (4.8)$$

Equation (4.8) considers uniform flow. Unsteady processes on the bed due to turbulence are not included. For a more detailed stability analysis the influence of fluctuating forces, which are introduced by turbulence, is important. For example; in open-channel flow turbulence is generated close to the bed and in non-uniform flow turbulence is also caused by the geometry of hydraulic structures. Turbulence is a complex process and is described in section 4.4.

4.3.3 The entrainment mechanism

The Reynolds decomposition separates the mean velocity $(\bar{u})$ from the fluctuating part $(u')$:

$$u = \bar{u} + u' \quad (4.9)$$

It appears that fluctuations play an important role in the entrainment of particles.

The described forces in Figure 4.11 only give a brief description of the process that causes instability. How grains can be entrained by a flow is described in Hofland (2005). He executed two experiments in order to get more insight in the entrainment mechanism of granular material. A short summary of his work is given below:

Fluctuating forces on bed material are generated by the same mechanisms as the mean forces. These fluctuations are called quasi-steady forces. These forces can account for the drag forces on exposed stones under uniform flow. For shielded particles and flow behind a backward-facing step the quasi-steady mechanism cannot explain all fluctuations anymore. The largest quasi-steady forces have a long duration, long enough to make the stone roll away.

Quasi steady forces are generated by pressure differences due to the streamline curvature that is caused by the presence of a stone protruding (II, Figure 4.11) in the flow. In a turbulent flow, acceleration of water parcels and streamline curvature are always present (induced by turbulence/vortices) even without the presence of a stone that forces this curvature. Therefore, turbulence near a wall creates fluctuating pressures on the bed, even when it is smooth. These turbulence wall pressure fluctuations (TWP) will result in net forces on the stone and therefore
contribute to the fluctuating forces on a stone. This already was stated by Booij (1998). Flow structures causing TWP are small, so these forces have a short duration. These forces will only lead to a rocking motion of the stone.

TWP give a significant contribution to the force fluctuations for lower exposures and higher turbulence levels. Therefore, the TWP are of importance for the entrainment of granular material, especially for stones that are shielded behind other stones, as they have generally small exposures. For higher exposures the relative influence of the TWP on the lift force seems larger than it is on the drag force.

Stones are entrained during the presence of large-scale areas with increased stream wise and downward velocity. This coincides with the presence of an increased quasi-steady force. Often the stones get an initial lift (or rotation) by an intense, small-scale fluctuation of the vertical velocity. This fluctuation of the vertical velocity is usually connected to the presence of a span wise vortex, rotating in the direction of the mean shear, which probably is a cross-section of a hairpin vortex (fluid motions swirling rapidly around a centre).

![Figure 4.13 - A span wise vortex, rotating in the direction of the mean shear (Hofland, 2005).](image)

The initial lift of short duration increases the exposed area and angle of repose such that the stone is moved more easily by the increased stream wise velocity reaching the stone after the fluctuation in vertical velocity. The small-scale force is related to the TWP. One can state that in general the stone is moved by an exceptional combination of two normal structures, at least one of which has an exceptionally high magnitude. If both flow structures work together they can make a stone rock and roll. The occurrence of a fluctuation in vertical direction results in a lower necessary strength (or shorter duration) of the quasi-steady part which would be required for entrainment.

Although the work of Hofland (2005) was focused on the stability of non-cohesive grains, it gives an idea of the importance of fluctuations. At the interface between filter and base these fluctuations will probably be reduced, but the particles of the base are smaller than the filter particles as well. The research of Hofland gives an indication that the fluctuations cause instability of particles at the interface, but when a flow is absent, no transport of base particles will occur. Breaking waves result in fluctuations of velocity and pressure and are therefore expected to have a significant influence on the entrainment of particles at the interface between filter and base.
4.4 Turbulence
Turbulent flows show irregular fluctuations in velocity and pressure. After giving a definition of these fluctuations a closer look is taken at spectral analysis, which makes it possible to identify the power that is carried per frequency (of the fluctuations). Spectral analysis is very useful in showing the reduction of power by filters.

4.4.1 Fluctuations of loading
The fluctuating part is a measure for the turbulence. The Reynolds decomposition (equation (4.9)) separates the fluctuations from the mean velocity. Turbulence can also be expressed as the square root of the average of the squares of the velocity fluctuations, which is the standard deviation (or the \( \text{rms of } u \)):

\[
\text{rms}(u) = \sqrt{\langle u'^2 \rangle} \tag{4.10}
\]

The same kind of fluctuations can be found in pressure. The term ‘turbulence’ indicates fluctuations in both velocity and pressure.

Fluctuations can have the same order of magnitude as the average values of the flow velocities. This means that the fluctuating part of the velocity can cause a force that can be of the same order of magnitude as the force caused by the time-averaged velocity. When examining stone/particle stability, this fluctuating part should be taken into account as well. The duration of the fluctuations is important too; a force should be working long enough to displace a grain. However, as was explained by the theory of Hofland (2005), a fluctuation can also initiate the entrainment process in which the duration is of less significance.

The forces that cause the fluctuations can have several origins. They can be caused by vortices that are shed from stones upstream, by turbulence that originates from the outer flow, or by vortices shedding from the stone under consideration itself.

Fluctuations appear with different frequencies. A usual tool for presenting the power along the frequencies is a spectral analysis.

4.4.2 Spectral analysis of turbulence
Turbulent flows generate eddies. Eddies with large length scale will form first and are called first-order disturbances. These eddies can be driven by external disturbances to the flow or non-linear internal fluid interactions within it. The length scales are in the same order of magnitude as the length scale of the external geometric features that shape the flow. The energies of the first-order eddies are mainly from mean flow. These eddies are also called the energy-containing eddies. Eddies with large length scale break down to second-order eddies of smaller length scale, because of their unstable character. The second-order eddies themselves are unstable as well and thus break down to form third-order eddies of smaller length scales and so forth. With each succeeding order, the length scales of eddies decrease as well as the times scales. The smaller time scales are in the former sections described as fluctuations. In the end, scales will be reached that are stable and do not break down anymore. Dissipative frictional forces act prominently to convert the energies of these eddies into heat.

In the sub-range where frictional dissipation is largely absent, the flux of energy is driven primarily by inertial forces responsible for the eddy formation and breakup. There is negligible dissipation and it turns out that this range is highly insensitive to external conditions. This range is called the inertial range and is conducive to the manifestation of a universal law of turbulence (Kolmogorov’s law).
Beyond the inertial range, the viscous forces become increasingly prominent. The range of scales in this region is known as the dissipation range.

Kolmogorov found that the inertial range in an energy density spectrum based on velocity measurements (under conditions of quasi-equilibrium, isotropy, homogeneity, and negligible dissipation) shows a constant -5/3 slope when plotting the energy along a logarithmic scale. This law applies to all turbulent flows (Chen, 2009).

In a similar manner as Kolmogorov, George et al. (1984) derived a spectrum function for pressure for the inertial range in a free shear flow. A distinction was made between the generation of pressure fluctuations by the interaction of the turbulence with the mean shear and the interaction of the turbulence with itself. It follows that the energy density spectrum of turbulent pressure fluctuations in the inertial range is the sum of three parts: a $f^{-11/3}$ component from the second-moment shear interaction, a $f^{-3}$ component of the third-moment shear interaction (which is zero when isotropy is assumed) and a $f^{-7/3}$ component from the turbulence-turbulence interaction.

The net energy density spectrum (Figure 4.15) is the sum of the contributions due to the turbulence-mean-shear interaction and the turbulence-turbulence interaction.
Regarding the fact that the breaking waves show a very turbulent character, a $-7/3$ slope is expected to be visible in the energy density spectra of the Großer Wellenkanal pressure signals. However, the validity of a $-7/3$ law for pressure is not generally accepted. Gotoh & Rogallo (1999) proposed a second range at $f^{-5/3}$ and Tsuji et al. (2007) even found exponents of $\lesssim -5/3$ (according to Detert, 2008).

### 4.5 Processes in filters

In the previous sections a distinction was made between the average flow velocity and the fluctuating part. This fluctuating part of velocity (and pressure) is described as turbulence and is considered important in the process that leads to transport of particles:

- Velocity fluctuations can have the same order of magnitude as the average velocity.
- Hofland (2005) explains that vertical fluctuations are involved in the entrainment process.

Filters are able to reduce both the average part of the velocity as the fluctuating part. The reduction of fluctuations of velocity and pressure has been described in several researches. A summary will be given in this chapter, thereby discriminating between velocity fluctuations and pressure fluctuations.

#### 4.5.1 Description of flows

Flow above a filter is different from flow within a filter. The flow in a base material again differs from the flow in a filter.

**Above the filter**

During uniform flow conditions the flow will not change in direction or velocity. The Reynolds number ($Re$) indicates whether a flow is laminar or not. The Reynolds number is a dimensionless number that gives a measure of the ratio of inertial forces to viscous forces and consequently quantifies the relative importance of these two types of forces for given flow conditions.

\[
Re = \frac{ul}{v} \quad (4.11)
\]

In practice, flow above a filter is never laminar ($Re \geq 2000$).

**In the filter**

Flow above the filter causes flow through the filter; porous flow. In the filter, a flow can have both laminar and turbulent properties and can either be perpendicular or parallel. The critical velocity and gradient for cyclic parallel flow with periods longer than two seconds is the same as for uniform parallel flow (De Graauw et al., 1983\textsuperscript{4}). The critical gradient for cyclic perpendicular loading appeared to be lower, explained by the inability of building arches during cyclic loading (De Graauw et al., 1983\textsuperscript{5}).

\textsuperscript{4} According to Schiereck (2001)

\textsuperscript{5} According to Schiereck (2001)
Hoffmans et al. (2000) discussed velocity of flow in a horizontal one-layer filter in open-channel flow. Based on theoretical considerations Hoffmans et al. (2000) came up with a relation describing the filter velocity as function of the vertical coordinate. The result is schematically given in Figure 4.17; the filter velocity decreases with increasing filter thickness.

The velocity of flow through a filter is sometimes expressed in pore velocity, which is the real velocity in the pores.

\[ u_f = n \cdot u_p \quad (4.12) \]

In the base

The remaining shear stress between the filter and the base (indicated by \( \tau_{bf} \) in Figure 4.17) will result in a flow through the base material. Flow trough fine material like sand is laminar; viscous forces will be dominant.

### 4.5.2 Relation between pressure and velocity

The reduction of the average velocity in a filter will accompanied by a reduction in pressure; velocity and pressure are related. This can be shown with the Navier-Stokes equations. These are equations governing the motion of Newtonian fluids. They are derived from the balance of forces of a fluid element, using Newton’s laws of motion:
When the Reynolds number is very large, which is the case under the hydraulic conditions in this research, the viscous term can be neglected. The resulting flow is called Euler flow:

\[
\frac{\partial \hat{u}}{\partial t} + (\hat{u} \cdot \nabla) \hat{u} = -\frac{1}{\rho} \nabla p + \nu \nabla^2 \hat{u} + \frac{1}{\rho} \hat{F} \tag{4.13}
\]

\[
\nabla \cdot \hat{u} = 0 \tag{4.14}
\]

The Navier-Stokes equations make clear that velocity and pressure are related and that a change in pressure will result in a change in velocity. Pressure gradients drive velocity changes.

### 4.5.3 Turbulence in a filter

In section 4.4 a first introduction was given about turbulence. The term turbulence describes fluctuations in velocity and fluctuations in pressure. Literature treating turbulence in filter layers is mostly dedicated to research in open-channel flow on a horizontal filter layer, without wave action. Although not directly applicable to the situation in this research it gives a good indication of the pressure and velocity processes in case of currents and an impression of what might happen in a wave loading situation. In this paragraph change of turbulence in a filter will be described. Although fluctuations in pressure and velocity always appear together they will be treated separately in the next paragraphs.

In the case of a horizontal filter under permanent flow, the cause of filter velocities and fluctuations of pressure and velocity is twofold. For one thing, there is the gradient that is caused by the difference in water level in front of and behind the construction. This introduces a more or less constant filter/pore velocity over the depth of the filter, with probably a more or less constant \(v_{rms}\) value of the fluctuations over the filter. For another thing, there is the exchange of momentum between the turbulent boundary layer just above the filter and the upper part of the filter layer. In the case of wave loading, only the latter is present.

**Hauer (1997)**

Hauer (1997) gives a description of the turbulence development, including average velocities and velocity fluctuations, based on both calculations and measurements. The maximal pressure gradients at the interface of the flow and the filter layer are a lot bigger than the average pressure gradient that drives the flow. These gradients are only active during a very short time period. Due to this short active period these large pressure gradients only result in a relative small velocity fluctuation. In the lower part of the filter layer the calculated velocity fluctuations appear to be a factor 10 to 20 bigger than the average filter velocity. This suggests that the pressure gradients are of longer duration and/or the average velocity is decreased. The latter is certainly true, as was explained in the former section. According to Hauer, the velocity fluctuations in a filter layer caused by pressure fluctuations do have a big influence on the erosion of the base layer.

**Bezuijen and Köhler (1998)**

Bezuijen and Köhler (1998) examined the stability of revetment structures, which is governed by the interaction between pore water on the one hand and the top layer, filter layer and base layer on the other hand. Based on theoretical consideration they derived an exponential function for reduction of the relative turbulence intensity:
Bezuijen and Köhler conclude that there must be a reduction in relative turbulence (the fluctuating part of the hydraulic loading). Hauer’s research doesn’t deny a decrease in the fluctuations, but states that the average velocity reduces more than the fluctuating part. A further literature search to the development of velocities within a filter should give more clearance.

4.5.4 Velocity fluctuations

In this section, a closer look is taken at the reduction of the velocity fluctuations in a filter.

*Van Os (1998)*

Van Os wrote a master thesis about the intrusion of hydraulic loading in horizontal geometrically open filter constructions and the influence of this on the erosion of the underlying layer. For that purpose physical model testing was performed in a small flume. Velocity measurements were done to demonstrate how the flow velocity and the turbulence above the filter penetrate in the filter. Only permanent current loading was studied.

Until \(1.5 \frac{D_{f15}}{\phi}\) a strong reduction of the average pore velocity and standard deviation of the velocity is observed. Deeper in the filter layer the pore velocity and the velocity fluctuations remain constant. However, an increase of filter thickness still shows a reduction in loss of base material. This is explained by the fact that the distance between base and flow is longer for a thicker layer.

*Jansens (2000)*

Jansens performed velocity measurements in a physical model test in and above a horizontal filter layer subjected to wave attack.

The chosen wave conditions are critical for the stability of the base material, observed by Halter (1999). The latter has determined a number of critical wave loads, that initiated incipient motion of base material under a falling apron. The same filter properties have been used in the model of Jansens, where the base material was replaced by cement.

Velocity measurements \(2\, \text{cm}\) above the filter bed were done with an electromagnetic velocity meter. For the measurements of the velocities in a pore of the filter layer a Laser-Doppler Flow Meter (LFDM) was used. Results show that the values for both the horizontal as the vertical pore velocity over the thickness of the filter layer are constant for every loading condition, when the orientation of the measured pores at different heights is constant. The explanation given by Jansens is that the wave length of the considered wave conditions are a lot bigger than the thickness of the filter layer, which results in an evenly distributed gradient over the whole filter depth. No relation between the orientation and the fluctuations was found.

Figure 4.18 summarizes the results of the observations of Jansens (2000). Both the average pore velocity and the fluctuations of the pore velocity remain constant going deeper in the filter.
The notion should be made that Jansens suspects that the measured fluctuations are influenced by the measuring equipment. Also, he suggest that during a critical wave loading there could be an average pore velocity, that combined with the fluctuations of the pore velocity is significant (critical, seems a better description) for a certain filter layer.

*Klar et al. (2004a/b), Klar (2005) and Detert et al. (2007)*
Among others, the researchers M. Klar, M. Detert, H.J. Köhler and T. Wenka all describe the same experiment each with their own focus, in which simultaneous pressure- and velocity measurements in horizontal gravel beds under a turbulent open-channel flow have been executed (2003-2007). The papers and dissertation describe the velocity fluctuations separately from the pressure fluctuations.

Velocity is measured by 3D-Particle Tracking Velocimetry (3D-PTV). Three endoscopic stereo probes recorded image sequences of flow inside specially prepared artificial gravel pores of the filter layer. These images are analysed with a special algorithm afterwards.

Pressures are measured by up to ten Miniaturized Piezoelectric Pressure Sensors (MPPS), which are located arbitrarily within the gravel layer. Three of the sensors were attached to the artificial gravel pores.

Measurements of the flow were done by applying 1D-Acoustic Doppler Current Profiler (ADCP). For a complete overview of the model set-up, see Figure 4.19.
Klar et al. (2004a/b) made a contribution to the research by investigating the velocity- and pressure-fields above and within the gravel bed. Goal was to gain new insight into the physical flow processes inside the gravel layer and to quantify the influence of turbulent fluctuations on the stability of both the gravel and the sand grains. To investigate the influence of accessibility of the pores, velocity measurements were done in an easy accessible pore ('open') and a pore which was less accessible ('closed') for the flow. In spite of the fact that both pores are located in the same vertical position from the gravel-water interface, results show that flow velocities and velocity fluctuations in a 'closed' pore are significantly lower than in an 'open' pore (see Figure 4.20). The standard deviation in the 'closed' pore is approximately 50% smaller than in the 'open' pore. Contrary, Jansens (2000) only found influence of the accessibility on the average pore velocity and not on the fluctuations. This was, however, under a wave loading situation. The conclusion of Klar et al. (2004a/b) gives an indication that both average pore velocities as velocity fluctuations are less in the pores of a wide graded filter; the porosity is generally smaller in a filter that has a wider grading, which makes the pores less 'accessible'.

Figure 4.19 - Model lay-out of Klar/Detert/Köhler/Wenka (Detert et al., 2007).

Figure 4.20 - Velocity measurements in an open (left) and a closed pore (right) (Klar et al., 2004a).
A power spectrum of the pore velocity shows that the whole frequency range of velocity fluctuations are dampened with increasing depth in the gravel layer, which is another proof that also fluctuations of high frequencies are reduced:

![Power spectrum of velocity data obtained from different vertical positions in the gravel layer (Klar et al., 2004a).](image)

Although not treated that extensively in the dissertation of Klar (2005), also measurements under surface waves with amplitudes of $\approx 8 \, \text{cm}$ and a period of $\approx 2 \, \text{s}$ were obtained. Figure 4.22 shows the result. Simultaneous time series of free surface flow and pore flow are visible. The free surface flow is shown in $\text{cm/s}$ and the pore flow in $\text{mm/s}$. The plot clearly indicates the penetration of the wave-induced pressure field into the gravel layer, which in turn drives the flow fields within the pores. Unfortunately, no velocities were measured deeper in the filter. What can be observed is the reduction of velocity within the pores compared to the velocity above the filter. To what extend the reduction will continue going deeper in the filter stays unknown.

![Simultaneous time series of the free surface flow and pore flow, obtained under non-stationary flow conditions due to surface waves (Klar, 2005).](image)
Detert et al. (2007) examined the distribution of the 3D-PTV results in more detail. Figure 4.23 shows the damping of the local 3D velocity and the fluctuations in a pore. The strong scatter in the velocity as well as the fluctuation part is explained by imperfections in the measurements:

- "The gravel layer represents a highly stochastic geometric system of channels, in directions and dead-end pores. Depending on the flow conditions and the current gravel geometry, in some experimental runs a rather homogeneous density of tracer particles was observed in the artificial pores, while in others this was not the case. As a result, the number of velocity vectors per frame is not constant.
- It was observed that during some experimental runs dirt particles were temporarily deposited in the artificial pores. These can reduce the intensity of the illuminations. The last effect is related to the limitations of the image processing.
- Especially under low mobility conditions, the turbulence intensity in the upper grain layers becomes very large. In these cases, the maximum pore flow velocities may reach values beyond the limits of the current endoscopic 3D-PTV. Again, the number of recovered velocity vectors drops and the velocity statistics get biased towards lower velocities."

In Figure 4.23 the local intrinsic 3D velocities \( u_{3D} \) within an artificial pore and the fluctuations \( \text{rms}(u_{3D}) \) are presented. From this figure, Detert et al. (2007) concluded that velocity fluctuations are reduced after \( 2D_{s90} \). The average pore velocity decreases somewhat slower. For the decrease of average pore velocity a logarithmic function and for the decrease of velocity fluctuations an exponential function has been chosen by Detert et al. (2007).

**Summary**

All above mentioned researches describe the development of velocity and velocity fluctuations in a narrow graded horizontal open filter layer. A short summary of observation of processes within the filter is given here.
In the filter, under current loading situations:

- Van Os (1998) concluded that both the average velocity as the velocity fluctuations reduce in the first $1.5D_{50}$, but are constant going further into the filter. The reduction in erosion of base material is declared by the longer path the particles have to travel before they reach the flow above the filter, and not the reduction of the velocities or fluctuations in general.
- Detert et al. (2007) gives a reduction of the pore velocity fluctuations within two times the nominal grain size and a somewhat slower reduction of the pore velocity. Klar et al. (2004a/b) only observed that all frequencies within the pore velocity signals are dampened by increasing filter thickness.
- Klar (2004a/b) concludes that both the velocity as the velocity fluctuations depend of the orientation and accessibility of the pore structure.

In the filter, under wave loading situations:

- Klar (2005) shows that the flow velocity under a wave loading situation is lower in the filter, than above the filter.
- The average pore velocity is dependent of the orientation and accessibility of the pore structure (Jansens, 2000).
- The fluctuation of the pore velocity is independent of the orientation and accessibility of the pore structure (Jansens, 2000).

A decrease in velocity and velocity fluctuations is shown by all researches, particularly in the upper part of the filter layer ($< 3D_{50}$). Bezuijen and Köhler’s theoretical consideration, which predicts an exponential decay of relative turbulence within a filter, supports those observations. The same process is expected for the development of pressure and pressure fluctuations. The next paragraph will treat this aspect of turbulence.

4.5.5 Pressure fluctuations

Pressure fluctuations result in velocity fluctuations. In the former section it was concluded that the velocity fluctuations decrease. Therefore, it seems logical that reduction in pressure fluctuations can be found as well.

Detert et al. (2007)

The previous section already treated the velocity analysis of this research. The model set-up is indicated in Figure 4.19. Now, the pressure analysis of the research will be dealt with. Figure 4.24 shows an example of the $rms$-values of the pressure fluctuations for increasing shear stress $\tau_0/\tau_{0,c}$. Pressure sensors were located at vertical positions from $(y/D_{50}) = 1.0$ above to $-7.5$ within the gravel layer. Only the transducers until $(y/D_{50}) = -2.0$ are plotted, because deeper in the gravel no essential difference in $rms$-values between the vertical positions could be determined. The transducer above the filter is horizontally aligned, the transducers in the filter vertically. The reason for the difference in placement is not clear.
Table 4.2 - Location of transducers above (+) and in the filter (-) ($D_f = 1.0$ cm) for Detert et al. (2007).

<table>
<thead>
<tr>
<th>Sensor</th>
<th>y/D_f</th>
<th>[cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{+10}$</td>
<td>1.0</td>
<td>+1.0</td>
</tr>
<tr>
<td>$P_{0.0a}$</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$P_{0.0b}$</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>$P_{-10}$</td>
<td>-1.0</td>
<td>-1.0</td>
</tr>
<tr>
<td>$P_{-20}$</td>
<td>-2.0</td>
<td>-2.0</td>
</tr>
<tr>
<td>$P_{-75}$</td>
<td>-7.5</td>
<td>-7.5</td>
</tr>
</tbody>
</table>

Figure 4.24 - Pressure fluctuations $rms(p)$ for increasing $\tau_0$, both normalized by $\tau_0=8.8$ Pa (Detert et al., 2007).

The damping by the gravel becomes obvious. This supports the observation of Van Os (1998) and Klar (2004a) regarding the decrease in velocity fluctuations.

Figure 4.25 shows a detailed view at the pressure fluctuations, recorded at low mobility-conditions. This figure only shows the fluctuations, so the average pressure is subtracted from the total pressure. This is another confirmation of the damping of the higher frequencies, which continues with increasing gravel depth.
Figure 4.25 - Simultaneous time traces of $p'$ ($u = 0.86 \, m/s, \tau_d/\tau_{0c} = 0.59, h = 0.249 \, m$) (Detert et al., 2007).

Figure 4.26 shows power spectral densities of the time series of all pressure transducers. In section 4.4.2 an introduction to spectral analysis has been given. The $f^{-5/3}$, $f^{-7/3}$ and $f^{-11/3}$ were explained as typical in the inertial sub-range. This range is highly insensitive to external conditions.

Figure 4.26 - Power spectral densities for measured pressures (Detert et al., 2007).

Within the gravel layer an essential damping between 1 to 3 Hz can be recognized. Below $\approx 2$-3 times $D_{fs0}$ ($P_{20}$) within the gravel layer there is no identifiable difference in damping pressure fluctuations higher than 3 Hz. The power spectral densities of the oscillating water level has been added to illustrate that this coincides very well with the spectra calculated for the lower part of the gravel. The low frequent fluctuations penetrate deeper in the filter than high frequent fluctuations. It can be
supposed that these long wave fluctuations as a result of the oscillating water level play a minor role in the entrainment of single grains (Hofland, 2002).  

In Figure 4.27 the damping of fluctuations with increasing gravel depth becomes even more obvious. The signal is filtered with a high pass \( f > 1.5 \text{ Hz} \) to eliminate the long wave influence. Deeper than a dimensionless depth of \( \left( y / D_{50} \right) < -2.0 \), the detected fluctuations stay nearly constant.

![Figure 4.27](image)

Comparing this result with Figure 4.23 gives obvious similarities between the decrease of pressure fluctuations and velocity fluctuations within the filter.

**Vollmer et al. (2002)**

Vollmer et al. performed pressure measurement under turbulent open-channel flow over rough surfaces and relatively large submergence. It was shown that an oscillating fluid motion is driven by highly unsteady pressure gradients and contributes to inflow and outflow at the interface between filter surface and flow, but its intensity decays rapidly with increasing filter depth. Vollmer et al. observed that about 90% of the high frequency pressure fluctuations measured at the surface of the filter layer is damped at a filter depth of about 4-5 times the average gravel size of surface bed. In Figure 4.28 the impact of pressure fluctuations on the pore flow is shown as a function of the dimensionless filter depth \( \left( y / D_{50} \right) \). The pressure fluctuations are defined in the same way as the velocity fluctuations (4.10); the square root of the variance, which is the standard deviation.

\[
\text{rms}(p) = \sqrt{\langle \sigma_p^2 \rangle} \quad (4.17)
\]

Vollmer et al. eliminated the low frequency fluctuations caused by water level fluctuations in Figure 4.28.

---

\( ^6 \) According to Detert et al. (2007)
Summary

It is clear that pressure fluctuations do decrease, just as velocity fluctuations. Vollmer et al. (2002) observed that about 90% of the high frequency pressure fluctuations measured at the surface of the filter layer is damped at a filter depth of about 4-5 times the average gravel size of surface bed.

Detert et al. (2007) also analyzed the frequencies of the pressure fluctuations. Below $2 - 3D_{f50}$ within the gravel layer there is no identifiable difference in damping pressure fluctuations higher than 3 Hz. Lower frequencies of pressure do not show any reduction. The long wave oscillations of the outer flow dominate the fluctuations within the gravel layer. These long wave fluctuations play a minor role in the entrainment process of single grains (Hofland, 2002).\(^7\)

\(^7\) According to Detert et al. (2007)
4.5.6 Overview of conclusions

In the former sections the development in fluctuations of pressure and velocity are treated separately. A clear overview of all results is made in Table 4.3.

Table 4.3 - Overview of conclusions about reduction of velocity and pressure fluctuations in filters.

<table>
<thead>
<tr>
<th>Source</th>
<th>Loading type</th>
<th>Reduction of average pore velocities ($\bar{u}$)</th>
<th>Reduction of velocity fluctuations ($\text{rms}(u)$)</th>
<th>Reduction of pressure fluctuations ($\text{rms}(p)$)</th>
<th>Reduction of turbulence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hauer (1997)</td>
<td>Open-channel flow (calculation)</td>
<td>Yes</td>
<td>Not clear</td>
<td>Not clear</td>
<td>-</td>
</tr>
<tr>
<td>van Os (1998)</td>
<td>Open-channel flow</td>
<td>Yes, until 1.5$D_{50}$</td>
<td>Yes, until 1.5$D_{50}$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bezuijen and Köhler (1998)</td>
<td>Open-channel flow</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Yes</td>
</tr>
<tr>
<td>Jansens (2000)</td>
<td>Surface waves</td>
<td>No</td>
<td>No</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vollmer et al. (2002)</td>
<td>Open-channel flow</td>
<td>-</td>
<td>-</td>
<td>Yes, 90% of fluctuations reduced within 4 – 5$D_{50}$</td>
<td>-</td>
</tr>
<tr>
<td>Klar et al. (2004a/b)</td>
<td>Open-channel flow</td>
<td>-</td>
<td>Yes, especially for $f &lt; 10$ Hz</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Klar (2005)</td>
<td>Surface waves</td>
<td>Yes</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Detert et al. (2007)</td>
<td>Open-channel flow</td>
<td>Yes, within 3 – 4$D_{50}$</td>
<td>Yes, within 2 – 3$D_{50}$</td>
<td>Yes, within 2 – 3$D_{50}$</td>
<td>-</td>
</tr>
</tbody>
</table>

Results show that only the conclusions of Jansens (2000) deviate from the trend that is given by the other researches. The studies of Detert et al. (2007) and Klar et al. (2004a/b) are the most reliable, because high-tech measuring devices were used. For simplicity it is assumed that the term ‘turbulence’, used by Bezuijen and Köhler (1998), can be compared with fluctuations in velocity and pressure.

For open-channel flow it can be concluded that fluctuations of both pressure and velocity do decrease with increasing filter thickness, with a limit around 2 – 3$D_{50}$ for the velocity fluctuations and a limit around 4 – 5$D_{50}$ for the pressure fluctuations.

For reduction of velocity under surface waves less information was available. Klar (2005) includes some information, but the result of that research is only treated briefly in literature. It indicates the reduction of pore velocities by the filter. For even more turbulent situations, central in this thesis, it is unknown whether a decrease in average values and fluctuations will appear. However, the results in Table 4.3 make it probable that a reduction will be found. The filter stones presumably absorb the turbulent energy, but to what extend is unknown.

In Appendix B all the absolute material properties of filter and base of the experiments are compared. The material properties appear to be all in the same range. That isn’t the case for the loading conditions, which is best indicated by the ratio:
The filter material in the Delta Flume tests is under a relatively strong wave attack. Another obvious difference between the experiments is indicated by the following ratio (Table B.2):

\[
\frac{D_{f50}}{D_{b50}}
\]

This ratio is the lowest for the Delta Flume tests. This may be one of the reasons why in the experiments of Uelman (2006) and Ockeloen (2007) severe erosion appeared and no erosion was observed in the Delta Flume experiment.

The ratio \( n_f D_{f15}/D_{b50} \), which includes the porosity in the filter layer, can be used to describe the critical gradient for perpendicular flow. Most experiments were under horizontal flow. For others, the values in Table B.2 are much larger than plotted in Figure 4.29. This results in a small critical gradient for erosion of base particles as a result of perpendicular flow. When \( n_f D_{f15}/D_{b50} \) is large, this may result in a behaviour of the base material like there is no filter at all; fluidization. Waves acting on an unprotected slope cause a step profile. This is the natural equilibrium profile for all slopes composed of loose material (Vellinga, 1986)\(^8\). Uelman (2006) and Ockeloen (2007) found such profiles in their experiments.

---

8 According to Schiereck (2001)
4.6 The Influence of filter properties

This section examines the consequence of the filter thickness and the grading on filter behaviour.

4.6.1 Filter thickness

An important research to the influence of filter thickness is done by Uelman (2006), which also was discussed in section 4.2. He varied the filter thickness of a geometrically open filter at a breakwater subjected to wave attack in a small scale physical model test in order to get more insight in the influence on the erosion of the core material.

The variation of the filter thickness by Uelman (2006) resulted in the following: “The total erosion, erosion depth \(d_s\), and the erosion length 2 \(L_{r2}\) after 2400 waves are dependent on the relative filter thickness \(d_f/D_{50}\). With an increasing \(d_f/D_{50}\), the total erosion, erosion depth \(d_s\), and the erosion length 2 \(L_{r2}\) are decreasing”, for definitions see Figure 4.30.

The erosion area is defined as the area between the original sand slope and the eroded sand slope in the part of the construction where erosion takes place.

In Figure 4.31 the values found by Uelman (2006) are extrapolated to values where hypothetically no erosion would occur. The left part of the figure only shows the erosion under a constant wave condition and a constant filter material type with varying relative filter thickness. The right part of the figure shows varying relative filter thickness, where all tested wave conditions and filter configurations are present. Both extrapolations suggest that the relative filter layer thickness of the Delta Flume models \(m > 0.37\) should result in a stable interface under the same conditions as tested by Uelman (2006). A stable interface under increased loading conditions is even possible, given the margin in Figure 4.31 to the relative filter layer thickness in the Delta Flume model.
The relation between decreasing erosion area with increasing relative layer thickness seems interesting, regarding the fact that in section 4.5.6 the large relative filter layer thickness in the tested Delta Flume models was noticed.

Wörman (1989) investigated the use of one thick single layer of riprap within a filter layer as an alternative to multi-layered riprap protection and the result for the interface stability around a pile under a highly turbulent flow. The following relationship resulted from his research in which the filter thickness is related to the ratio of the size of filter particles and base particles:

\[
\frac{d_f}{D_{f15}} = 0.16 \frac{\Delta_f}{\Delta_b} \frac{n_f}{1 - n_f} \frac{D_{fb5}}{D_{bbs}}
\] (4.18)

This formula endorses on the reduction of velocities and fluctuations, which was observed in the former sections. A larger ratio of \(D_{fb5}/D_{bbs}\), thus a more open filter, results in a larger filter thickness which is straightforward; a larger reduction of the hydraulic loading is necessary.

However, in section 4.5.6 it was shown that a reduction of average velocity, velocity fluctuations and pressure fluctuations under turbulent open-channel flow is limited to maximum five times the nominal diameter. Therefore, it would be most economical to limit the filter layer thickness to five times the nominal filter diameter. Nevertheless, like van Os (1998) mentioned, the transport of base particles is hindered by the length of the path to the (cyclic/permanent) flow above the filter. The thicker the filter layer, the longer this path will be. So, a thicker filter does have a function in reducing the erosion of base particles, but probably not by unlimited reduction of the flow velocities.

### 4.6.2 Grading

The Rock Manual (CIRIA, 2007) gives a definition of narrow, wide and very wide gradations as a function of the \(D_{fb5}/D_{f15}\) ratio (Table 4.4).

<table>
<thead>
<tr>
<th>Grading width</th>
<th>(D_{fb5}/D_{f15})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Narrow or single-sized gradation</td>
<td>Less than 1.5</td>
</tr>
<tr>
<td>Wide gradation</td>
<td>1.5 – 2.5</td>
</tr>
<tr>
<td>Very wide gradation or quarry run gradation</td>
<td>2.5 – 5.0</td>
</tr>
</tbody>
</table>

\(\text{gap graded - well graded}\)
For wide graded materials a distinction can be made between gap graded and well graded. The latter does not show any gap in material sizes over the total width of the grading.

In theory, the advantage of a well graded filter is that the larger particles can prevent the smaller particles from being washed out and that the pores in the filter are small. A wide graded filter layer with the same $D_{50}$ and the same filter thickness as a narrow graded filter should result in less erosion. This may be one of the reasons that the profile of the base material in the Delta Flume tests was stable, when viewing the results of Table B.1 (Appendix B).

The downside of a wide grading is the danger of loss of internal stability. Internal stability means that the range of grain diameters in the filter layer should not be too large, so that the larger grains can block the smaller ones. Instability may lead to internal erosion or segregation:

- **Internal erosion:**
  The process of washout of fine-grained particles through the voids associated with the coarse particles within the same layer is called internal erosion. Internal erosion is a consequence of an internally unstable filter. It results in compaction, but it is not likely to lead to a rapid failure.

- **Segregation:**
  Segregation of filters is another consequence of internal instability. Herewith the separation of the larger and smaller particles of a filter is described, which can be the result of (for instance) wave loading. In the test done for ‘Cobble beach Zuidwal MV1’, commissioned by Rijkswaterstaat (Verslag Modelonderzoek Waterloopkundig Laboratorium, 1973) there was a tendency that the coarse material is gathered around the breaker point of the waves, while the finer particles were gathering higher on the profile.

A material is described as having a stable gradation if it does not lose particles. Grading stability is dependent upon size and size distribution of particles, porosity or relative density and severity of the disturbing forces. Kenney and Lau (1985) made a proposal for evaluating the potential for grading instability based on the shape of a material’s grain size curve, in which the absolute sizes of the particles are of little importance in comparison with the shape of the grading curve. When the potential travel distances of mobile particles are short, most of the filter volume will retain its initial grain size characteristic. When the potential travel distances for certain sizes of loose particles are larger than the filter thickness, these particles can be lost from throughout the filter. The grains size characteristic can change, and the filter grading must be judged as being potentially unstable.

Heibaum (2004)⁹ presents a summary of a procedure originally developed by Cistin and Ziems, whereby the $D_{50}$-size of the filter is selected on the coefficients of uniformity ($D_{100}/D_{50}$) of both the base and the filter material. Figure 4.32 provides a chart based on the Cistin-Ziems approach.

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⁹ According to CUR Desk Study
So far, the grading is especially presented as a property that determines the stability of a filter. That the grading, filter thickness, filter material are related is shown in CUR Desk Study. A new formula for interface stability of granular filter structures in open-channel flow is presented. It combines the influence of grading and the filter layer thickness, showing that these parameters are related to the interface stability:

$$\frac{df}{D_{f15}} = 2 \ln \left( \alpha_r \frac{\Delta f}{D_{f50}} \psi_{ef} \left(1 - \gamma V_{ef}\right) \right)$$  \hspace{1cm} (4.19)

Ockeloen (2007) performed, among other physical model tests in the flume of the Delft University of Technology, an experiment to identify the influence of the grading of a filter on the erosion beneath a geometrically open filter layer of a breakwater construction. He found out that using the same initial slope and the same loading conditions, the wide graded filter results in the same amount of erosion and the same bar-profile. The only difference in the test with wide grading is that the erosion area above SWL is less pronounced. A comment that can be made is that a wider grading could have been chosen. This, and the fact that Ockeloen only tested one wide graded filter, makes the result questionable.

The Delta Flume filter has three advantages in comparison with the filter of the Großer Wellenkanal:

- The filter layer is thicker (absolute and relative).
- The median size of the filter particles is smaller.
- The grading of the filter is wider.

The last two points will result in smaller and thus less accessible pores. A higher reduction is therefore expected for the Delta Flume filter under the same hydraulic conditions. The thicker filter will be efficient due the longer path that particles have to travel through the filter.

---

30 According to CUR Desk Study
4.7 Overall summary literature study

Flume tests of Uelman (2006) and Ockeloen (2007) show that the parallel downward porous flow is the dominant loading process rather than the turbulence caused by breaking waves for the interface (filter-base) of reasonably steep single layered breakwater constructions. It is not likely that turbulence dies out so rapidly in the models in the Delta Flume and the Großer Wellenkanal, because hydraulic conditions are much heavier than in the tests of Uelman and Ockeloen.

Turbulence is a measure for the fluctuating part of velocities and pressures. The role of fluctuations in tests under open-channel flow is twofold:

- Velocity fluctuations can have the same order of magnitude as the average velocity. This means that the fluctuating part of the velocity can cause a force that can be of the same order of magnitude as force caused by the time-averaged velocity.
- Hofland (2005) explains that vertical fluctuations are involved in the entrainment process.

Fluctuations resulting from turbulent open-channel flow can be reduced by filters. A decrease in velocity and high frequent velocity fluctuations and pressure fluctuations is shown by all relevant researches, particularly in the upper part of the filter layer ($< 5D_{50}$). Spectral analysis of pressures show that lower frequencies of pressure do not show any reduction deeper in the filter (Detert et al., 2007), but a reduction of power of higher frequencies is evident.

Such a decrease of pore velocity and/or pressure (fluctuations) under cyclic loading is less obvious. Jansens (2000) did not find a reduction in pore velocity nor pore fluctuations. Klar (2005) only shows a reduction of velocity in the filter compared to the velocity of the flow above the filter. Whether velocity fluctuations decrease under the turbulent character of wave loading and breaking waves as well is therefore uncertain.

Overall, under the hydraulic loading of a turbulent open-channel flow, both filter thickness and filter grading have influence on the process that leads to erosion of base particles underlying a filter. The increasing filter thickness causes an increasing path length from the base to the channel flow, thereby reducing the chance that a base particle can travel through the whole filter. Filter grading is especially important regarding the stability of the filter itself, although formula (4.19) shows that filter material, grading and filter thickness all together have influence on the interface stability. Besides, Klar (2004a) shows that velocity and velocity fluctuations are lower in less accessible pores.

The analysis of the Großer Wellenkanal measurements and the Delta Flume measurements should provide enough information to obtain a presumption of the reduction of turbulence resulting from breaking waves. Additionally, it should be possible to identify the character of the hydraulic loading at the interface filter-sand of a single layered filter construction which may be helpful in explaining the stable interface in the Delta Flume models.
5 Analysis Großer Wellenkanal measurements

The literature study made clear how the fluctuating part of the pressure signals generated by a turbulent open-channel flow reduces in a filter. The pressure signal resulting from a turbulent open-channel flow is different from the signal obtained under a breaking wave. After demonstrating this difference, useful results of the LWI research to the performance of the Elastocoast revetment will be elaborated. Thereafter, spectral analysis of the pressure signals are presented which show the damping of pressures in a filter along the frequencies. Pressures measurements are made available by the Braunschweig University of Technology.

Pressure differences cause gradients. The model configuration of the Großer Wellenkanal model contains a lot of pressure transducers. A detailed analysis of occurring gradients is performed to identify the most critical loading situation regarding interface stability. Traditional stability criteria will be used to verify if transport would occur when no geotextile would be present.

The irregular wave climate that has been used during the measurements of the pressures can be described by a significant wave height of $H_s = 0.67 \text{ m}$ and a spectral period of $T_{m-1.0} = 3.61 \text{ s}$. The pressure signals have a length of 4200 s.

The Elastocoast revetment exists out of an Elastocoast layer and a filter layer. The term ‘filter’ is confusing. It suggests that the material is different from the top layer, which isn’t the case. Both layers exist out of the same material and the difference in porosity is negligible. Therefore the Elastocoast layer will be described as a bounded layer. The filter layer will be described as an unbounded layer (see Figure 5.1).

![Figure 5.1 - Bounded and unbounded layer.](image-url)
5.1 Pressure signals

The literature study is mainly limited to research under the hydraulic loading of turbulent open-channel flow. A pressure signal resulting from a (breaking) wave is significantly different from those caused by open-channel flow. In Figure 5.2 the typical pressure signals created under different conditions is sketched.

![Pressure signals](image)

Figure 5.2 - Pressure signal of a turbulent open-channel flow (left), a pressure signal as a result of a plunging wave resulting in an impact load (middle) and a pressure signal as a result of a non-impact load (right).

Where for a pressure signal resulting from turbulent open-channel flow a clear distinction can be made between an average and a fluctuating part, this is not as straightforward for the pressure signal of an impact load and a non-impact load. In the former sections of this report, turbulence has been described as the \( \text{rms} \)-value (root mean square value) of the fluctuating part of the pressure signal. Under an open-channel flow the average part of the signal is more or less constant, whereas the signal created under a (breaking) wave changes continuously and thus cannot be described by an average.

In Figure 5.3 a distinction between impact loads and non-impact loads has been made using the surf similarity parameter (LWI Report no. 988, 2010).

![Impact, transition and non-impact zone](image)

Figure 5.3 - Impact, transition and non-impact zone, indicated as a function of \( \xi \) (LWI Report no. 988, 2010).

The surf similarity parameter can be calculated by using equation (4.2). The pressure signals that have been analyzed in this chapter are the result of a wave climate with a surf similarity parameter of \( \xi = 1.84 \). Impact loads can thus be expected, but because a JONSWAP spectrum has been used also non-impact waves will be present in the pressure signals. Between \( 2.5 < \xi < 2.9 \) there is a transition zone. For \( \xi > 2.9 \) non-impact peaks and troughs appear in the pressure signals.

A closer look at the impact loads is necessary to distinguish two parts of that signal. The impact part should be treated separately from the rest of the signal, which is called the quasi-static part.
The signals in Figure 5.3 look rather smooth compared to an actual pressure signal as indicated in Figure 5.4. In the signal of PT30 a high frequent fluctuation can be noticed. These high frequent fluctuations are also present in the pressure signal of non-impact waves. The diagram in Figure 5.5 has been made to show all the definitions of the pressure signal that will be treated in this chapter.

When only pressure signals are known, like in the Delta Flume, it is impossible to recognize a separate section for the pressures resulting from waves with a surf similarity parameter that falls within the transition zone.

The pressure signal will change by the influence of the revetment. The identification of this change is the focus in the next paragraph.
5.1.1 Reduction of pressures

For a large part, the analysis of the pressures of the Großer Wellenkanal test has already been done by the Braunschweig University of Technology. In LWI Report no. 988 (2010) the reduction of the impact part and the quasi-static part of the impact load and the reduction of the non-impact load is already performed. A summary of the results will be given in this section.

In the mentioned report, a non-dimensional pressure (5.1) has been used to identify the reduction of pressures:

$$\hat{p}_{\text{max}} = \frac{p_{\text{max}}}{\rho g H_2}$$  \hspace{1cm} (5.1)

By comparing the non-dimensional pressures above and below the bounded layer and above and below the unbounded layer the identification of the reduction was possible.

**Impact load** ($\xi < 2.5$)

The impact load can be described by two maximum pressures.

One is resulting from impact part and ($p_{\text{max},i}$) one is resulting from the quasi-static part ($p_{\text{max},q}$), thereby defining the values as the absolute difference between the trough and the peak. By comparing the non-dimensional maximum pressure above and below the bounded layer and above and below the unbounded layer a reduction value can be found.

LWI analysed the regular wave tests and found out that reduction of 40% of the $\hat{p}_{\text{max},i}$ is achieved by the bounded layer. The quasi-static maximum pressure is not reducing.

In Figure 5.7 typical signals of the pressures which occur above and below the bounded layer as a result of irregular waves with $H_s = 0.67\ m$ and $T_m^{m-1.0} = 3.61\ s$ are plotted. It is clearly visible that the troughs of the pressure signals do not coincide.
This means that besides a peak reduction, also a trough reduction is achieved by the bounded layer. The quasi-static part is not as strongly damped as the impact part. This confirms the observations made by LWI.

The unbounded layer has a different effect on the pressures. The same time frame that was plotted for the bounded layer in Figure 5.7 has been plotted for the unbounded layer in Figure 5.8.

The transducers that were used to plot Figure 5.7 (Model B) and Figure 5.8 (Model C) were not located directly below each other. Model B and Model C have been tested side by side simultaneously. By plotting the same time frame for both transducers, the pressures as a result of the same wave are plotted. This does not necessarily mean that PT5 shows the same impact peak as...
PT37. The material of the bounded layer is different for the models and waves do not have to be entirely uniform over the width of the flume. Nevertheless, Figure 5.8 shows the removal of the impact part below the unbounded layer.

The quasi-static part of the signal is significantly less than the impact part. According to LWI Report no. 988 (2010), 35% of the quasi-static part is damped by the unbounded layer.

**Non-impact load (ξ > 2.9)**

In Figure 5.9 the maximum pressure value for a non-impact load ($p_{\text{max}, n}$) is defined.

LWI found no reduction of the non-impact pressures by the bounded layer. The unbounded layer reduces the maximum pressures from non-impact loads by 35%. A similar behaviour for the quasi-static component of the impact load was observed.

The resulting pressure signal at the interface of Model C is therefore only showing the remaining pressures of the non-impact loads and the quasi-static parts of the impact load. The geometrically open revetment is able to reduce both loads by 35%. Therefore these loads can be described by one single term; penetrating loads. In Figure 5.10 the reduction of the penetrating pressures is plotted against the dimensionless filter thickness ($y/D_{50}$).
5.1.2 The high frequent fluctuations in the pressure signal

When looking at the unmodified signals on a much smaller scale, high frequent fluctuations of pressure are visible. The high frequent fluctuations of the pressure signal under turbulent open-channel flow were described in several researches in section 4.5.5. Under a breaking wave, the pressure signals show the same fluctuations (Figure 5.11).

The \( \text{rms} \)-value of the difference between the average and the original signal of a pressure transducer is a usual value to indicate the strength of fluctuations or turbulence intensity. Using this approach for pressure signals resulting from waves requires the use of a moving average. The problem when taking
the moving average of impact loads is that the impact part of the pressure signals deforms and as a result does have a lot of influence on the final value of the turbulence intensity. This can be seen in Figure 5.12.

For that reason, the impact peaks have to be skipped when calculating the \( \text{rms} \)-value of a signal. Every signal has been checked visually on impact parts between \( t = 500 - 1000 \text{ s} \). In this time frame about 10% of the waves caused an impact load. These waves were deleted from the dataset. The remaining data was used to calculate the fluctuation intensity per signal. In that way a representative fluctuation value for about \( (\frac{500}{3.61}) \cdot 0.9 = 125 \) waves is calculated. For the influence of the bounded layer the transducers of Model B have been used (Table 5.1). For the evaluation of the influence of the unbounded the transducers of Model C have been used (Table 5.2).

![Figure 5.12 - The deformation of peaks and the influence for the \( \text{rms} \)-value.](image)

<table>
<thead>
<tr>
<th>Pressure [kPa]</th>
<th>PT5</th>
<th>PT12</th>
<th>Moving Average PT5</th>
<th>Moving Average PT12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pressure [kPa]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time [s]</td>
<td>929.4</td>
<td>929.6</td>
<td>929.8</td>
<td>930</td>
</tr>
</tbody>
</table>

\( \text{Figure 5.12 - The deformation of peaks and the influence for the } \text{rms} \text{-value.} \)
Table 5.1 - Fluctuation of pressure (rms-values) above and below bounded layer of Model B.

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Fluctuation above revetment [kPa]</th>
<th>Transducer</th>
<th>Fluctuation below revetment [kPa]</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT28</td>
<td>0.061</td>
<td>PT36</td>
<td>0.025</td>
<td>67%</td>
</tr>
<tr>
<td>PT30</td>
<td>0.067</td>
<td>PT37</td>
<td>0.037</td>
<td>45%</td>
</tr>
<tr>
<td>PT32</td>
<td>0.073</td>
<td>PT38</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.2 - Fluctuation of pressure (rms-values) above and below the unbounded layer of Model C.

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Fluctuation above revetment [kPa]</th>
<th>Transducer</th>
<th>Fluctuation below revetment [kPa]</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT3</td>
<td>0.050</td>
<td>PT11</td>
<td>0.042</td>
<td>16%</td>
</tr>
<tr>
<td>PT5</td>
<td>0.066</td>
<td>PT12</td>
<td>0.042</td>
<td>36%</td>
</tr>
<tr>
<td>PT7</td>
<td>0.074</td>
<td>PT13</td>
<td>0.031</td>
<td>58%</td>
</tr>
</tbody>
</table>

The absolute value of a high frequent fluctuation is just a fraction of the total pressure signal. In section 5.3 a closer look will be taken at the influence of the fluctuations on the occurring gradients. Fluctuations around the moving average are decreasing by the influence of the unbounded layer and by the influence of the bounded layer. The turbulence intensity is increasing in the wave direction (from PT28 to PT32, and from PT3 to PT5), which can be explained by the fact that the wave is breaking along the transducers resulting in a more turbulent water motion somewhat higher at the slope. This trend is not visible for transducers below the filter, where all fluctuation values are more or less the same.

A contradiction in the results is that the values of the fluctuations at the interface of the bounded and unbounded layer are larger for Model C than Model B. Smaller fluctuations than in Model B would be expected, because the material of the bounded layer is somewhat smaller.

High frequent pressure fluctuations are only a fraction of the total occurring pressure. These fluctuations decrease by the influence of the filter, which was also expected on basis of the results of the literature study. It is striking that the fluctuations below the unbounded layer are more or less constant along the revetment. This makes it probable that a complete reduction of the small-scale fluctuations is not possible. On the other hand it is also possible that the remaining fluctuations are the result of noise in the pressure signal.
5.2 Spectral analysis

Spectral analysis makes it possible to estimate how the total power is distributed over frequencies from a finite record of a stationary data sequence. It can help in identifying possible hidden periodicities in time series. The basics of spectral analysis and the method used to create the spectral estimates in this chapter are elaborated in Appendix D.

From section 4.4.2 it is known that within the inertial range of frequencies the power spectrum shows an energy-cascade. The time-series generated by the transducers of the Großer Wellenkanal test have been used to create spectral estimates above, within and below the revetment. Detert et al. (2007) showed (section 4.5.5) how the reducing effect of a filter is visible in a power spectrum, which was created using the pressure signals that were generated under a turbulent open-channel flow. This results in the expectation that the more sheltered (or deeper in the filter located) sensors of the Großer Wellenkanal show a spectral estimate below the sensors situated on top of the revetment.

Power spectral density estimates which show the largest power per location (above/layer 1, within/layer 2 and below/layer 3) in the revetment are assigned as representative spectral estimates. In that way the power of the impact part of pressures of impact loads are certainly included in the spectral estimates. Figure 5.13 shows the model definitions. For layer 1 the transducers of Model B have been used for the power spectral density estimate. For layer 2 and 3 the pressure signals of transducers of Model C have been used. For a detailed explanation of the creation of representative power spectral density estimates, reference is made to Appendix D.3.2.

Figure 5.14 shows the power spectral density estimates of the signals distributed over the revetment. Peaks in all estimates were found at $f = 0.24 \, \text{Hz}$, which more or less resembles the spectral period ($T = 3.61 \, \text{s}$) of the JONSWAP spectrum that was applied in the model test. A second peak was found in all estimates at $f = 0.48 \, \text{Hz}$, which is exactly twice the peak frequency. A third peak was found in most estimates at around $f = 0.72 \, \text{Hz}$, which is exactly three times the peak frequency. The second and third peak are called the higher harmonics and are resulting from the Fourier transform that is necessary for creating spectral estimates.
The representative power spectral density estimate above the revetment shows a $f^{-5/3}$-tail. The spectral estimate below the bounded layer shows a $f^{-7/3}$-tail and the spectral estimate below the unbounded layer shows a $f^{-11/3}$-tail. Instead of a spectral estimate that contains all three tail-types, described as the net energy density spectrum in Figure 4.15, each spectral estimate shows a clearly different behaviour indicating the change in turbulence character. With increasing filter depth the width of the spectral estimate decreases, indicating the damping of frequencies higher than $f = 1$ Hz. Frequencies higher than $f = 6$ Hz appear to be almost powerless at the interface. This confirms the reduction of the impact parts of the pressure signals (section 5.1.1) and the reduction of the high frequent fluctuations of the pressure signals (section 5.1.2).

The energy bump within frequencies of $f = 7 - 20$ Hz in the spectral estimate of the signals above the revetment indicates the peak pressures as a result if the impact of the breaking waves. This bump is reduced by the bounded layer and completely removed by the unbounded layer.

The noise in the spectral estimates at higher frequencies is probably the result of aliasing produced by high frequencies of which the signal was too small to absorb by the low-pass filter that was applied to the obtained data. The logarithmic scale gives a somewhat distorted view on the reduction of power. A relative damping of power gives a clearer overview. The relative damping is given in Figure 5.15.
Hydraulic interface stability of sand underlying a single filter layer

Reduction of power is visible for all frequencies, although it looks like that there is some increase in power at higher frequencies. This is caused by the increased noise in the spectral estimates at higher frequencies, which makes it hard to really identify the influence. Therefore the relative damping at frequencies $f > 20 \text{ Hz}$ should not be taken as reliable; the trend in Figure 5.14 shows that also these frequencies are damped by the revetment. The power of the high frequent fluctuations of the pressure signals, treated in section 5.1.2, should be visible in the frequencies $f > 25 \text{ Hz}$. This is exactly the range which shows the scatter. Nevertheless, these fluctuations seem to have less power below the bounded layer and even lower power below the unbounded layer.

The bounded layer is most effective in reducing frequencies $f = 2 - 11 \text{ Hz}$. The unbounded layer is the most efficient within the frequencies $f = 2 - 9 \text{ Hz}$. Below the unbounded layer, within the range $f = 2 - 11 \text{ Hz}$, only power in the order of 1% of the power on top of the revetment is left.

The expectation that the more sheltered (or deeper in the filter located) sensors show a spectral estimate below the sensors situated on top of the revetment appears to be valid. The impact peaks and high frequent fluctuations are damped by the porous material; the resulting curves are shifted to lower values of power and frequency respectively.

Figure 5.15 - Relative reduction of power along frequencies.
5.3 Gradient analysis

In the former paragraphs the influence of the revetment on the change of the pressures signals and the reduction of power that is carried by these signals has been described. A link of particle stability with the loading below the filter has not been made yet. Particles are being transported when the load is larger than the strength. The transport can either be parallel to the interface or perpendicular. A separate critical value for the parallel and the perpendicular direction can be distinguished. The bed can be considered stable when the load is less than some critical value for both directions. When the critical value is being exceeded incipient motion will occur.

The experiment in the Großer Wellenkanal makes it possible to calculate gradients in perpendicular and parallel direction. The impact parts of the pressures, which are a result of the impact load, are completely damped by the combination of the bounded layer and the unbounded layer below it (see section 5.1.1). However, when calculating perpendicular gradients these impact parts cannot be ignored. They have a large influence on the pressure differences between the transducers and thus the gradients. The parallel gradients at the interface are only the result of penetrating pressures.

The Großer Wellenkanal model was constructed with a geotextile between the unbounded layer and the bed material. Stability analysis will be done using the base material that was applied in the Delta Flume models.

5.3.1 Parallel gradients

The parallel gradient can be calculated as follows:

\[ i_x = \frac{1}{\rho g} \frac{\delta p}{\delta x} \] (5.2)

In Table 5.3 traditional methods of determining a critical parallel gradient are summarized. More information about these methods is available in Appendix E. In the original model in the Großer Wellenkanal a geotextile was present between the sand and the unbounded layer; particle transport is not possible. Therefore, critical gradients are calculated by using the material characteristics of the base material of the Delta Flume model tests.

<table>
<thead>
<tr>
<th>Method</th>
<th>Loading</th>
<th>( i_c )</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterloopkundig laboratorium (1970)</td>
<td>Cyclic</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Klein Breteler et al. (1992)</td>
<td>Cyclic</td>
<td>0.1</td>
<td>From test with smaller ( D_{150}/D_{50} )</td>
</tr>
<tr>
<td>Waterloopkundig laboratorium (1969)</td>
<td>Stationary</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>De Graauw et al. (1983)</td>
<td>Stationary</td>
<td>0.02</td>
<td>With slope correction</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.04</td>
<td>Without slope correction</td>
</tr>
<tr>
<td>Klein Breteler (1987)</td>
<td>Stationary</td>
<td>0.04</td>
<td></td>
</tr>
<tr>
<td>Klein Breteler et al. (1992)</td>
<td>Stationary</td>
<td>0.05-0.1</td>
<td></td>
</tr>
</tbody>
</table>
5.3.2 Perpendicular gradients
Both gasses and liquids move from high pressure to low pressure. In Figure 5.16 two transducers are indicated; one above (PT5) and one below the filter (PT12). When the pressure in PT5 is higher than PT12 the gradient is directed to PT12. This should result in increased stability; particles are pushed against the bed. A gradient directed to PT5 results in an upward force at the interface particles.

<table>
<thead>
<tr>
<th>$p_{PT5} &gt; p_{PT12}$</th>
<th>$p_{PT5} &lt; p_{PT12}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT5</td>
<td>PT5</td>
</tr>
<tr>
<td>Unbounded layer</td>
<td>Unbounded layer</td>
</tr>
<tr>
<td>PT12</td>
<td>PT12</td>
</tr>
</tbody>
</table>

Figure 5.16 - Resulting gradient pointed from higher pressure to lower pressure.

Looking at the pressure signals of PT5 and PT12 in Figure 5.17 a gradient directed towards PT12 is marked in green. A destabilizing gradient directed towards PT5 is indicated in yellow. Remarkably, an increased reduction of the troughs results in an increased ‘destabilizing’ perpendicular gradient. The upward directed gradients occur when pressures are negative, indicating that they are created during run-down.

![Direction of gradients](image)

Figure 5.17 - A positive and a negative pressure gradient.

In Figure 5.18 the forces acting on a particle during a purely upward perpendicular gradient when assuming a spherical shape are indicated. Again, the material characteristics of the base material of the Delta Flume models have been used as verification of the gradient.
When $F_L = F_S = F_G \cos \alpha$, there is equilibrium in perpendicular direction.

$$F_L = F_G \cos \alpha \quad (5.3)$$

$$i_y = \frac{1}{\rho g} \frac{\delta p}{\delta y} \quad (5.4)$$

$$\rho_w g i_y = (1 - n) (\rho_g - \rho_w) g \cos \alpha \quad (5.5)$$

$$\rho_w g \frac{1}{\rho_w g} \frac{\delta p}{\delta y} = (1 - n) (\rho_g - \rho_w) g \cos \alpha \quad (5.6)$$

$$\frac{\delta p}{\delta y} = 0.6 (\rho_g - \rho_w) g \cos \alpha \quad (5.7)$$

$$\frac{F_L}{F_S} = \frac{\delta p}{0.6 (\rho_g - \rho_w) g \cos \alpha} \quad (5.8)$$

When $\frac{F_L}{F_S} > 1$, the vertical gradient is able to lift the particles.

5.3.3 Results
The critical gradient in parallel direction and the ratio of forces in perpendicular direction can be used to verify the gradients that occur in the Großer Wellenkanal experiment. In Figure 5.20 the gradients in parallel direction and the relative lift forces in perpendicular direction are given. The definition of the subscripts is as follows:

- **Subscript (1/3):**
  - Parallel case: The mean of 1/3 of the highest gradients that occur.
  - Perpendicular case: The mean of 1/3 of the highest ratio of forces that occur.

- **Subscript (2%)**:
  - Parallel case: The mean of 2% of the highest gradients that occur.
  - Perpendicular case: The mean of 2% of the highest ratio of forces that occur.

- **Subscript (max):**
  - Parallel case: The maximum gradient that occurs.
  - Perpendicular case: The maximum ratio of forces that occurs.
The maximum perpendicular forces that occur are strong enough to lift the particles. According to $(F_L/F_S)_{1/3}$ and $(F_L/F_S)_{2\%}$ the largest upward forces can be found between transducers PT7 and PT13. The ratio $(F_L/F_S)_{max}$, however, is located somewhat lower at the slope. This value is only based on one observation and is therefore not representative for the occurring gradients.

The parallel gradients are able to transport particles as well. Comparing the parallel gradients at layer 3 with the parallel gradients at layer 2 it is noticeable that they are reduced (see Table 5.4).

<table>
<thead>
<tr>
<th>$i$</th>
<th>PT3-PT5</th>
<th>PT11-PT12</th>
<th>Reduction</th>
<th>PT5-PT7</th>
<th>PT12-PT13</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$i_{1/3}$</td>
<td>0.19</td>
<td>0.13</td>
<td>32%</td>
<td>0.19</td>
<td>0.17</td>
<td>11%</td>
</tr>
<tr>
<td>$i_{2%}$</td>
<td>0.37</td>
<td>0.21</td>
<td>43%</td>
<td>0.31</td>
<td>0.27</td>
<td>13%</td>
</tr>
<tr>
<td>$i_{max}$</td>
<td>1.62</td>
<td>0.83</td>
<td>49%</td>
<td>1.62</td>
<td>0.34</td>
<td>79%</td>
</tr>
</tbody>
</table>

The reduction of $i_{max}$ between PT5-PT7 and PT12-PT13 is higher than for the other transducers. For $i_{1/3}$ and $i_{2\%}$ the damping is significantly higher between PT3-PT5 and PT11-PT12. Because the maximum gradient is based on only one value, its reduction can be overestimated. Figure 5.21 clarifies why especially the $i_{max}$ shows a large reduction.
The impact part of impact loads are responsible for the largest pressure differences in parallel direction between PT5 and PT3 and thus create for the largest gradients. In Figure 5.21 it can be noticed that the highest pressure differences are of very short duration, which makes it questionable if they are really able to move the particles. Peak pressures are damped out by the revetment, resulting in a smaller pressure difference between PT11 and PT12 at the exact same moment as the large pressure difference between PT3 and PT5.

The parallel gradients at the interface are the result of differences of penetrating pressures (no impacts). It can be noticed that for $t > 2145.5$ the pressure differences PT3-PT5 and PT11-PT12 are almost equal, but that the difference between PT11 and PT12 is continuously smaller than the pressure difference between PT3 and PT5. The same mechanism was found in the measurement of PT5-PT7 and PT12-PT13.

A positive gradient in perpendicular direction will be in favour of stability; base particles will be ‘pushed’ into the embankment (see Figure 5.16 and Figure 5.17). Figure 5.22 shows the perpendicular gradients between the transducers PT3-PT11, PT5-PT12 and PT7-PT13. It appears that the negative perpendicular gradients are in phase remarkably often (in the order of >60% of the cases), especially when relative large gradients are occurring. A negative gradient between transducers means a lift force.

Figure 5.21 - Time series of pressures and pressure differences in parallel direction.

Analysis Großer Wellenkanal measurements
In other cases the maximum 'destabilizing' gradients occur in a particular order; first between PT3-PT11, followed by the maximum gradient between PT5-PT12 and finally a maximum gradient between PT7-PT13 (see Figure 5.23). Still, during these three moments the perpendicular gradients are all negative and thus pointed in the same direction.

Upward directed perpendicular gradients appear when pressures below the unbounded material are higher than pressure above the unbounded layer (Figure 5.16). It was already noticed that the upward gradients occur during wave run-down. The direction of the perpendicular gradients changes on
average every half wave period. The largest upward gradient can generally be found between transducers PT7 and PT13, as already was concluded from Figure 5.20.

Because the test has been carried out with an irregular wave climate, it is very difficult to recognize certain regularities in the pressure measurements and the occurring gradients. In Appendix E.8 it was tried to identify certain structures that are returning in the measurements. The results of the analysis of the lower, middle and higher part of the revetment are given in Figure 5.24.

In the figure above t₁, t₂ and t₃ are presented as separate moments in time:

At t₁, the maximal upward directed perpendicular gradient is often accompanied by a maximal upward directed parallel gradient between PT10 and PT11. The gradient between PT11 and PT12 is of negligible value at the same moment in time.

At t₂, the maximal upward directed perpendicular gradient is frequently accompanied by a maximal downward directed parallel gradient between PT13 and PT12. Again, the gradient between PT11 and PT12 is very small.

At t₃, the maximal upward directed perpendicular gradient is often accompanied by maximal downward directed parallel gradients between PT13 and PT12 and PT14 and PT13. The gradient between PT14 and PT13 appears to be there all the time, but is somewhat overestimated because wave set-up causes an increased average of pressure at that location of the revetment.
Hydraulic interface stability of sand underlying a single filter layer

It has to be noted that the situations as described above are not returning constantly, but are recognized to dominate the gradient signals. Independent of the location along the revetment, a maximum upward directed perpendicular gradient is frequently accompanied by a maximum parallel gradient.

High frequent fluctuations of the pressure signal around the moving average do not influence the actual gradients significantly. Generally the gradient fluctuations are only a fraction of the total gradients. Therefore, the high frequent fluctuations of pressure cannot be responsible for particle transport.

In more than 60% of the cases the ‘destabilizing’ perpendicular gradients along the revetment are occurring at the same time. This results in a situation where maximum gradients appear along a large part of the construction simultaneously (see Figure 5.25).

Figure 5.25 – Gradients when maximum lift forces along revetment appear simultaneously.

The gradient between transducers PT11 and PT12 is rarely of any value when the other parallel gradients are. When the gradient between transducers PT11 and PT12 is reaching its maximum downward directed gradient, there is often a small upward directed perpendicular gradient along the revetment. A summary is given in Figure 5.26. The small arrows indicate that there is still a gradient, but not at its maximum.
Besides the size, also the duration and direction of the gradients is important. A large gradient appearing for a short amount of time can have less influence than a small gradient that appears for a longer time. In Figure 5.27 the duration and direction of the gradients is presented.

From Figure 5.27 a downward transport of particles from the area between PT12 until PT14 would be expected when no geotextile would be present between the unbounded layer and the sandy embankment. The downward parallel gradient between PT11 and PT12 probably is not able to transport the same amount of material downwards, resulting in sedimentation of particles. The slope reduces the strength of particles in the bed. Therefore, downward transport of particles is expected to be somewhat higher than upward transport between PT10 and PT11. In Figure 5.28 the expected interface is plotted.
5.4 Summary of results

Relevant conclusions documented in LWI Report no. 988 (2010) are:

- Loads can be separated between impact loads ($\xi < 2.5$) and non-impact loads ($\xi > 2.9$).
- An impact load exists out of an impact part and a quasi-static part (see Figure 5.4).
- The bounded layer reduces the impact part of the impact load by 40% (crushed limestone 20/40 mm) of 0.15 m.
- The remaining 60% of the impact part is reduced by unbounded material of 0.20 m thickness (crushed granite 16/36 mm).
- Penetrating pressures (non-impact loads and quasi-static part of impact loads) are not reduced by the bounded layer.
- Penetrating pressures are reduced with 35% by the unbounded layer.

Pressure differences in the troughs of the signal are causing lift forces in perpendicular direction. Increasing reduction of pressures of the troughs will result in increasing upward gradients. Also, it was observed that high frequent fluctuations are only a fraction of the total pressure. Spectral analysis confirmed the reduction of power caused by impact peaks and high frequent fluctuations. The largest reduction of power occurs for frequencies between $f = 2 - 11$ Hz by the bounded layer and for $f = 2 - 9$ Hz in the unbounded layer.

Maximal parallel gradients at the interface of the bounded and unbounded layer are very large. This is the result if impact peaks of impact loads (Figure 5.21). Impact peaks are completely reduced by the bounded and unbounded layer together. Parallel gradients, which occur below the unbounded layer, are therefore only the result of the penetrating part of the pressures. This term describes the non-impact load and the quasi-static part of the impact load. These loads are reduced similarly; no reduction by the bounded layer and 35% reduction by the unbounded layer. A reduction of the penetrating loads will result in a decrease of the parallel gradients.

Impact causes a downward perpendicular gradient, which has a ‘stabilizing’ effect on the bed particles. Maximum upward (‘destabilizing’) perpendicular gradients occur during run-down. The upward perpendicular gradients often appear simultaneously along the revetment. In almost all deviating cases, the perpendicular upward gradient first occurs on the lower part of the revetment, followed shortly by the middle part of the revetment and finally the higher part of the revetment. At every location, the maximal ‘destabilizing’ perpendicular gradients are frequently accompanied by a maximal parallel gradient.
The duration of the direction of the parallel gradients is not the same along the revetment. Gradients between PT10-PT11 change direction every ½ wave period. Gradients between PT11-PT12 are ¾ of the wave period directed in downward direction. Gradients between PT12-PT13 and PT13-PT14 are continuously directed downwards, in- and decreasing with the wave period.

From the literature study it is known that high frequent fluctuations have an important role in the entrainment and transport of particles in a turbulent open-channel flow. Therefore, it was expected that the high frequent fluctuations caused by wave breaking would have an important role in the hydraulic loading within a filter and the interface between filter and sand. However, analysis of the high frequent fluctuations in the Großer Wellenkanal showed that they do not influence the occurring gradients significantly. Generally the fluctuations are only responsible for a fraction of the total occurring gradients. Therefore, the high frequent fluctuations of pressure cannot be responsible for particle transport. The main loading mechanism at the interface results from the run-down; during run-down the largest parallel and perpendicular gradients are generated.
6  Analysis Delta Flume measurements

The pressure measurements of the Delta Flume models (Loman, 2009d) have some disadvantages regarding the objective. Several models have been tested under different conditions, which make a fair comparison between the behaviour of the models difficult.

In this chapter, an analysis of the damping of pressure at the location of wave impact within the gravel revetment of the Delta Flume models is presented. By applying restrictions to the available model tests it is tried to come to a selection of comparable tests. Results of the former chapter will be used for verification. Thereafter, a formula for obtaining parallel gradients from the time derivative of a pressure signal is verified by using the Großer Wellenkanal test results. The chapter will be closed with an expectation of gradients at the interface between cobbles and sand.

6.1 Pressure signal

The pressure signals of transducers on top of the revetment in the Großer Wellenkanal show a lot of impact peaks. In the Delta Flume tests, the surf similarity parameters are of the plunging type ($\xi = 0.69 - 1.27$) and therefore imply that impact peaks should be present in these measurements as well. However, when plotting the pressure signals of the Delta Flume tests that were applied under comparable hydraulic conditions as the analyzed Großer Wellenkanal test, a relatively low amount of impact peaks is noticeable. Impacts that are present are of significant lower value (see Appendix F).

The low amount of these impacts is caused by the fact that the transducers in the Delta Flume models are situated in the top layer, rather than on top of this layer. The upper pressure transducer is covered with gravel; $\left(\frac{y}{D_{50}}\right)_{\text{TOP-PT3}}$ (see Figure 6.1). As was noticed for the revetment tested in the Großer Wellenkanal in section 5.1.1, the impact peaks are reduced very efficiently by a relatively thin layer of bounded granite stones. Additionally, impact peaks of impact loads have a very short duration. Because the frequency of the pressure measurements in the Delta Flume is only 5 Hz, discrimination between impact loads and penetrating loads is not possible.

In LWI Report no. 988 (2010) the reduction of the maximum pressures has been calculated making use of a non-dimensional pressure. By comparing the non-dimensional pressures above and below the bounded layer and above and below the unbounded layer the identification of the reduction was possible (described in section 5.1). It was not possible to use this approach in the Delta Flume analysis, a different method has been used to come to reduction values that are comparable.

Within the top layer of the Delta Flume models two transducers can be found; PT2 and PT3 (see Figure 6.1). The signal of PT2 is expected to be somewhat damped compared to the signal of PT3. $\left(\frac{y}{D_{50}}\right)_{\text{PT2-PT3}}$ is different for each model configuration and $\left(\frac{y}{D_{50}}\right)_{\text{TOP-PT3}}$ can change during each test; the hydraulic loading is free to transport the gravel particles. The thickness of the gravel layer on top of PT3 changes during every test. By incorporating the change in thickness during each test a reasonable detailed filter layer thickness is obtained. By doing this, a variety of dimensionless filter depths are resulting from one single series.
Because the amount of filter material is different for each test it can be seen whether the reduction of the pressure signals increases when a thicker layer of gravel is present between the transducers. The identification of the reduction has been obtained by using $\Delta p_{\text{mean}}$. This $\Delta p_{\text{mean}}$ describes the absolute difference between the mean peak and the mean trough of each signal. In Figure 6.2 an example of the signals of Series 4 Test 1 has been given. Although only a fraction of the total time series is plotted, the mean peaks and troughs have been calculated using the whole time series. By comparing $\Delta p_{\text{mean}}$ of PT2 by $\Delta p_{\text{mean}}$ of PT3 an impression of the reduction is gained.

From the tests in the Großer Wellenkanal it is known that a reduction of the penetrating pressures is not achieved in the upper layer of the revetment. The penetrating part of the pressures is only reducing in the unbounded layer and the impact is reducing in both layers. Because no transducers were situated on top of the revetment in the Delta Flume models such a reduction mechanism cannot be identified using the Delta Flume measurements. Because of the similarities between the material properties it is assumed that in the upper layer of the Delta Flume models no reduction of the penetrating pressures can be found either. The reduction between transducer PT3 and PT2 is therefore expected to show a similar reduction as the penetrating loads in the unbounded layer of the Großer Wellenkanal model.

Compared to the evaluated test conditions in the Großer Wellenkanal only two situations tested in the Delta Flume are such that wave height and period fall in an equivalent range. In Table 6.1 these tests including the damping results are given.
Table 6.1 – Reduction of pressures for equivalent Delta Flume tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>$H_s$ [m]</th>
<th>$T_{m-1.0}$ [s]</th>
<th>$\xi$ [-]</th>
<th>$(y/D_{f50})_{TOP-PT3}$</th>
<th>$(y/D_{f50})_{PT2-PT3}$</th>
<th>$(y/D_{f50})_{TOTAL}$</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>S4T1</td>
<td>0.71</td>
<td>3.88</td>
<td>1.14</td>
<td>15.57</td>
<td>27.65</td>
<td>43.22</td>
<td>16%</td>
</tr>
<tr>
<td>S5T1</td>
<td>0.87</td>
<td>3.88</td>
<td>0.70</td>
<td>13.73</td>
<td>25.19</td>
<td>38.92</td>
<td>18%</td>
</tr>
</tbody>
</table>

The revetment in the Großer Wellenkanal achieved a reduction of 35% of the penetrating pressures in a dimensionless filter layer thickness of $14D_{f50}$. A filter layer in the Delta Flume model which is almost three times as thick only achieves a reduction of 16%. Table 6.1 shows that the damping in S4T1 is less than in S5T1, while the filter layer thickness is larger. The filter layer thickness above transducer PT3 only differs slightly. The different breaker type cannot be the cause of the higher reduction in S5T1. According to the surf similarity parameter, the signal of PT3 in S4T1 shows higher impact peaks. Impacts are reduced more effectively than penetrating pressures; a higher reduction should be observed in S4T1.

Because the model configurations and hydraulic conditions differ it is complex to make a representative comparison between reduction values of the tests. By applying certain restrictions this imperfection is tried to overcome. Criteria that are used:

- The SWL must be above PT3, otherwise no reduction can be identified. This selection has been made in Appendix F, Table F.1.
- The distance between the impact location and PT3 must not be too large.
- The amount of filter material on top of PT3 must be similar for all tests, because this material has influence on the pressures.

The breaker type can be used to identify the location of the wave impact. Klein Breteler’s formula for wave impact location (6.1) predicts the location of peak pressure of maximum wave impact.

$$
\frac{z_{pmax}}{H_{m0}} = \min\left(\frac{0.45 \cdot \xi - 0.3}{1.7}\right)
$$

(6.1)

![Diagram showing the relationship between SWL (sea water level) and Diĕke (dike), with point of maximum wave impact marked.](image)

**Figure 6.3 - Definition of maximum $z_{pmax}$**

For a fair comparison with the Großer Wellenkanal tests, the transducers of PT3 should be located around the impact point of the plunging waves. The differences between the locations predicted by equation (6.1) and PT3 are given in Table 6.2.
Table 6.2 - Reduction and difference between point of impact and PT3.

<table>
<thead>
<tr>
<th>Test</th>
<th>$\xi$ [-]</th>
<th>$(y/D_{f50})_{TOP-PT3}$</th>
<th>$(y/D_{f50})_{PT2-PT3}$</th>
<th>$(y/D_{f50})_{TOTAL}$</th>
<th>Reduction</th>
<th>Difference $z_{p_{max}}$ - PT3 [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2T4</td>
<td>0.81</td>
<td>24.76</td>
<td>34.44</td>
<td>59.20</td>
<td>53%</td>
<td>0.19</td>
</tr>
<tr>
<td>S3bT3</td>
<td>1.26</td>
<td>5.04</td>
<td>20.88</td>
<td>25.92</td>
<td>16%</td>
<td>0.20</td>
</tr>
<tr>
<td>S4T1</td>
<td>1.14</td>
<td>15.57</td>
<td>27.65</td>
<td>43.22</td>
<td>16%</td>
<td>0.15</td>
</tr>
<tr>
<td>S4T2</td>
<td>1.14</td>
<td>17.49</td>
<td>27.65</td>
<td>45.14</td>
<td>14%</td>
<td>0.12</td>
</tr>
<tr>
<td>S4T3</td>
<td>1.23</td>
<td>19.85</td>
<td>27.65</td>
<td>47.50</td>
<td>12%</td>
<td>0.30</td>
</tr>
<tr>
<td>S4T4</td>
<td>1.13</td>
<td>26.37</td>
<td>27.65</td>
<td>54.02</td>
<td>8%</td>
<td>0.54</td>
</tr>
<tr>
<td>S4T5</td>
<td>1.19</td>
<td>27.25</td>
<td>27.65</td>
<td>54.90</td>
<td>11%</td>
<td>0.39</td>
</tr>
<tr>
<td>S5T1</td>
<td>0.70</td>
<td>13.73</td>
<td>25.19</td>
<td>38.92</td>
<td>18%</td>
<td>0.36</td>
</tr>
<tr>
<td>S5T2</td>
<td>0.70</td>
<td>15.01</td>
<td>25.19</td>
<td>40.20</td>
<td>15%</td>
<td>0.42</td>
</tr>
<tr>
<td>S5T3</td>
<td>0.69</td>
<td>16.08</td>
<td>25.19</td>
<td>41.27</td>
<td>16%</td>
<td>0.58</td>
</tr>
<tr>
<td>S5T4</td>
<td>0.70</td>
<td>22.18</td>
<td>25.19</td>
<td>47.37</td>
<td>17%</td>
<td>0.84</td>
</tr>
<tr>
<td>S5T5</td>
<td>0.74</td>
<td>24.47</td>
<td>25.19</td>
<td>49.66</td>
<td>19%</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Looking at the difference between the $z_{p_{max}}$ calculated with equation (6.1) and the actual position of PT3 it can be seen that for some tests this difference is quite large. Therefore a selection can be made. For S2T4, S3bT3, S4T1 and S4T2 the difference $z_{p_{max}}$ - PT3 is reasonably small. Looking at the model configurations in Appendix A.2 shows that for S4T3, S4T4, S4T5 and Series 5 the distance between SWL and PT3 is indeed higher than for other series. The surf similarity parameter, which determines the expected location of impact, is calculated with an average slope between the toe of the revetment and the crossing between SWL and the revetment. Klein Breteler’s formula can be seen as a line through a cloud of data and is therefore not able to precisely discriminate between the small range of surf similarity parameters in Table 6.2. Thus, predicted points of impact can be somewhat different from the points that have been calculated with the formula of Klein Breteler. For that reason no reduction value will completely be ignored.

It can be noticed that the amount of material on top of PT3 during S3bT3 is low. This results in more impacts in the signal of this pressure transducer. Because impacts are reduced efficiently the resulting reduction value is expected to be higher and thus not giving a representative value for the reduction of penetrating pressures.

It has been assumed that the penetrating pressures do not reduce by the influence of the material on top of PT3. This results in a reduction of 0% at the location of PT3 for all tests. In Figure 6.4 the reduction in percentages have been replaced by the ratio:

$$\frac{\Delta P_{2, mean}}{\Delta P_{3, mean}}$$

This approach should result in reduction values comparable with the approach of LWI. The ratio is plotted against the dimensionless filter depth, including a trend line (no mathematical background). Transducers at wave impact are indicated by blue dots.
The results of Table 6.1 already showed the difference in the damping of the pressure signals between both model tests. Figure 6.4 emphasizes this.

### 6.2 Formula verification

As can be seen in Appendix A.2 the transducers in the Delta Flume models are situated below each other. It is therefore not possible to directly find parallel gradients from the pressure measurements. Deltares used the following formula for the calculation of parallel gradients (Loman, 2009b):

\[ i = \frac{1}{\rho g} \frac{\delta p}{\delta x} = \frac{1}{\rho g} \frac{\delta p}{\delta t} \frac{1}{c} \quad (6.2) \]

\[ c = \sqrt{gh'} \quad (6.3) \]

\[ h' = \max \left( h + \frac{\rho g}{\rho \rho g} \right) \quad (6.4) \]

\[ h = z_{SWL} - z_{profile} \quad (6.5) \]

In Appendix G this formula is validated using the measurements that were available from the Großer Wellenkanal. These measurements make it possible to compare results of formula (6.2) with actual parallel gradients. It appears that the formula overestimates the parallel gradients significantly, using the original signals. This is caused by impact peaks and small scale fluctuations (described in section 5.1.2), which introduce large time derivatives of the pressure signal. Reasonable results are only obtained when the moving average of the signals of transducers below the unbounded layer is used. In these modified signals no impact peaks and no small fluctuations are present.

Therefore, applying formula (6.2) to the pressure signals of transducers deeper in the filter (like PT2) will result in reliable predictions of gradients. The character of the pressure signal is more quasi-static / non-impact like. Since no transducer was situated directly at the interface between gravel and sand, Deltares used two methods to estimate gradients at the interface between cobbles and sand of the Delta Flume models:
• Linear extrapolation of the pressure measurement from transducer PT3 and PT2 to the interface. An indication of this approach is plotted in Figure 6.5.
• Linear interpolation of the pressure measurements between PT2 and PT1.

![Figure 6.5 - Linear extrapolation method Deltasres.](image)

The higher of the two resulting gradients has been chosen as a ‘best estimate’. Whether this is indeed a good approximation is questionable; it seems that pressures decrease different than the linear assumption made by Deltasres. Figure 6.5 indicates the upper and lower bound of the linear approach by Deltasres. In the next paragraph a prediction of gradients at the interface will be given on basis of the results of Figure 6.4.

The extrapolation method assumes that time derivatives reduce with the same factor as the absolute pressures. This means that the parallel gradient reduces with the same factor as the absolute pressures. It is questionable if this is true. A gradient is nothing more than a pressure difference over a certain distance. When absolute pressures decrease, this does not necessarily mean that the pressure difference between two points also decreases by the same factor. A further research to the 1:1 relation that is given by formula (6.2) is therefore recommended.

### 6.3 Prediction of parallel gradients at interface

Because the reduction of pressures at the location of a breaking wave is known and validation of equation (6.2) is positive, gradients occurring at the interface between gravel and base material can be estimated:

\[
i = \frac{1}{\rho g \delta x} \frac{\delta p}{\delta t} = \frac{1}{\rho g \delta t} \frac{1}{c}
\]

(6.6)

\[
\delta p = p_{t1} - p_{t2}
\]

(6.7)

When the pressure reduces with factor \(k\) (reduction value gained from the red line in Figure 6.4), it can be assumed that the time derivative also reduces with a factor \(k\) :
\[ k \cdot p_{t1} - k \cdot p_{t2} = k(p_{t1} - p_{t2}) = k \cdot \delta p \] \hfill (6.8)

\[ i = \frac{k \cdot \delta p}{\rho g} \frac{1}{c} \] \hfill (6.9)

The gradients can be calculated using the pressure signal of PT3 or PT2. Because formula (6.9) is sensitive to the influence of impact peaks, using the pressure signal of PT2 will lead to more reliable estimates of gradients at the interface. However, some reduction values are deviating significantly from the trend, indicated with the red line in Figure 6.4. For those tests the signal of PT3 has been used. Table 6.3 indicates the critical gradients which have been used for determination of particle motion. These values are a summary of critical gradients from traditional stability methods, indicated in Appendix E. In Table 6.4 all results are given, including the gradients resulting from the approach of Deltares.

### Table 6.3 - Critical gradients.

<table>
<thead>
<tr>
<th>Expected behaviour</th>
<th>( \iota_z % [-] )</th>
<th>Indicated in Table 6.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>No movement</td>
<td>&lt; 0.05</td>
<td></td>
</tr>
<tr>
<td>Movement/No movement</td>
<td>0.05 - 0.15</td>
<td></td>
</tr>
<tr>
<td>Movement</td>
<td>&gt; 0.15</td>
<td></td>
</tr>
</tbody>
</table>

### Table 6.4 - Prediction of parallel gradients at interface.

<table>
<thead>
<tr>
<th>Test</th>
<th>( i_{z1} % [-] )</th>
<th>( i_{z2} % [-] )</th>
<th>Best estimate</th>
<th>( i_{z2} % [-] ) PT2</th>
<th>( i_{z2} % [-] ) PT3</th>
<th>( (y/D_{f50})_{TOTAL} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2T4</td>
<td>-0.08</td>
<td>0.02</td>
<td>0.02</td>
<td>0.01</td>
<td>0.03</td>
<td>59.20</td>
</tr>
<tr>
<td>S3bT3</td>
<td>0.13</td>
<td>0.16</td>
<td>0.13</td>
<td>0.01</td>
<td>0.01</td>
<td>25.92</td>
</tr>
<tr>
<td>S4T1</td>
<td>0.07</td>
<td>0.06</td>
<td>0.07</td>
<td>0.10</td>
<td>0.13</td>
<td>43.22</td>
</tr>
<tr>
<td>S4T2</td>
<td>0.10</td>
<td>0.07</td>
<td>0.10</td>
<td>0.13</td>
<td>0.14</td>
<td>45.14</td>
</tr>
<tr>
<td>S4T3</td>
<td>0.07</td>
<td>0.07</td>
<td>0.07</td>
<td>0.10</td>
<td>0.15</td>
<td>47.50</td>
</tr>
<tr>
<td>S4T4</td>
<td>0.25</td>
<td>0.12</td>
<td>0.25</td>
<td>0.10</td>
<td>0.08</td>
<td>54.02</td>
</tr>
<tr>
<td>S4T5</td>
<td>0.17</td>
<td>0.10</td>
<td>0.17</td>
<td>0.07</td>
<td>0.08</td>
<td>54.90</td>
</tr>
<tr>
<td>S5T1</td>
<td>0.10</td>
<td>0.07</td>
<td>0.10</td>
<td>0.07</td>
<td>0.11</td>
<td>38.92</td>
</tr>
<tr>
<td>S5T2</td>
<td>0.16</td>
<td>0.10</td>
<td>0.16</td>
<td>0.17</td>
<td>0.06</td>
<td>40.20</td>
</tr>
<tr>
<td>S5T3</td>
<td>0.22</td>
<td>0.12</td>
<td>0.22</td>
<td>0.17</td>
<td>0.06</td>
<td>41.27</td>
</tr>
<tr>
<td>S5T4</td>
<td>0.23</td>
<td>0.13</td>
<td>0.23</td>
<td>0.21</td>
<td>0.06</td>
<td>47.37</td>
</tr>
<tr>
<td>S5T5</td>
<td>0.18</td>
<td>0.10</td>
<td>0.18</td>
<td>0.09</td>
<td>0.11</td>
<td>49.66</td>
</tr>
</tbody>
</table>

Because the reductions of S3bT3, S4T4, S4T5 and S5T1 are deviating significantly from the trend in Figure 6.4, only PT3 has been used for estimations of the gradients at the interface. From Table 6.4 it can be concluded that using a different approach of pressure reduction does not lead to gradients below the critical gradient of particle motion. The Deltares approach even results in slightly lower gradients for almost all tests. Only a very thick gravel layer \((y/D_{f50})_{TOTAL} > 50\) will lead to smaller gradients using the red line of Figure 6.4.
6.4 Hydraulic stability of sand underlying cobble layer

Gradients at the interface between sand and gravel of the Delta Flume models are higher than critical gradients for particles motion. An erosion process of the sandy embankment is therefore expected. With observance of the direction and duration of the gradients of the Großer Wellenkanal tests, an erosion pattern as was documented by Uelman (2006) and Ockeloen (2007) would be likely (see Figure 6.6).

Such an erosion profile was not observed. The stable interface in the Delta Flume tests can thus not be explained.

The fact that the revetment in the Großer Wellenkanal is able to reach an efficient damping relatively fast in comparison with the revetment of the Delta Flume is rather strange when taking the material of the revetments into account. The material of the Delta Flume revetment is somewhat smaller en the grading somewhat wider. In chapter 4 it was concluded that this should be an advantage, because smaller (more sheltered) pores result in smaller velocities and smaller fluctuations.

Nevertheless, Figure 6.4 shows that the performance of the revetment in the Großer Wellenkanal is much better. Partly, this can be attributed to the fact that the revetment in the model is fixed; the bounded layer prevents the unbounded material to move around. This results in a larger resistance against the wave action and thus a more efficient reduction of pressures than in a gravel revetment that has the freedom to find its dynamic equilibrium. On the other hand, the movement of the particles of the latter can dissipate the energy they got from the hydraulic loading by moving around. It is not believed that the fixation completely explains the large differences between the damping of the different revetments.

The damping observed in the Großer Wellenkanal and the non-distorted interface in the Delta Flume models give strong indications that the measurements in the gravel material of the Delta Flume are not representative for the performance of the gravel layer, resulting in a conservative prediction of gradients at the interface of cobbles and sand.

Several factors could have been influencing the pressures in the revetments of the Delta Flume and the reduction found in the analysis of the measurements:

- An irregular wave climate has been used; two types of load subjected to the models.
- Hydraulic conditions differ for every test step.
• The low frequency of the pressure transducers makes discrimination between impact and non-impact loads not possible, which may result in wrong impression of reduction values.
• Model configurations of series are different and thereby also the location of the pressure transducers.
• Varying filter layer thickness including varying amounts of material on top of the upper transducer for each test.
• Wall effects; pressure transducers are mounted to the wall of the flume. Pressure transducers in the Großer Wellenkanal model are mounted closer to the centre of the flume.

Figure 6.7 – Pressure transducer mounted to the wall of the Delta Flume (Loman, 2009c).

Figure 6.8 – Location of pressures transducer in the Großer Wellenkanal model (LWI Report no. 988, 2010).
7 Conclusions and recommendations

The objective of this research was as follows:

*Identify the reduction of hydraulic loading under a breaking wave by a filter, get insight in the influence of different filter properties on this reduction and explain the stable interface of sand underlying a cobble layer.*

This objective exists out of three parts, which will be treated in the first paragraph:

- Identify the reduction of hydraulic loading under a breaking wave by a filter.
- Get insight in the influence of different filter properties.
- Explain the stable interface of sand underlying a cobble layer.

This chapter will end with some recommendations.

7.1 Conclusions

*Identify the reduction of hydraulic loading under a breaking wave by a filter*

Using the pressure measurements obtained from the Großer Wellenkanal model and the results of LWI Report no. 988 (2010) the reduction of hydraulic loading in a revetment could be identified.

- The hydraulic loading of waves results in two types of loads; impact and non-impact loads. The impact load can be distinguished in an impact part and a quasi-static part.

![Figure 7.1 - Impact part and quasi-static part of pressure signal (left) and non-impact load (right).](image)

- Impact parts are causing very large parallel pressure differences in the upper part of the gravel layer, lasting for only a fraction of time (see Figure 7.2).
Impact peaks are effectively reduced by the bounded (40%) and unbounded (60%) material. Non-impact pressures and quasi-static pressures are only reduced by 35% by the unbounded layer. Parallel gradients at the interface are the result of pressure differences of the quasi-static pressure part caused by impact waves and the non-impact waves (penetrating pressures). The penetrating pressures are reduced by the revetment, but a reasonable filter thickness is needed before efficient reduction starts.

High frequent fluctuations are assigned an important role in the entrainment and transport process of particles under a turbulent open-channel flow. From the literature study it became clear that a filter is able to cause a reduction of average velocity, velocity fluctuations and pressure fluctuations under turbulent open-channel flow. The high frequent fluctuating part appears to be only a fraction of the total pressure. Additionally, these high frequent fluctuations are reduced by the revetment. As a result, the influence on the gradients below the revetment of those fluctuations is negligible.

The remaining pressures at the interface are reduced penetrating pressures including very small high frequent fluctuations. These fluctuations have negligible influence on the hydraulic loading at the interface. Parallel gradients, caused by these penetrating pressures, can be calculated using formula (7.1) when high frequent fluctuations are removed by taking the moving average of the penetrating loads.
Conclusions and recommendations

\[ i = \frac{1}{\rho g} \frac{\delta p}{\delta x} = \frac{1}{\rho g} \frac{1}{c} \]  
(7.1)

\[ c = \sqrt{gh'} \]  
(7.2)

\[ h' = \max \left\{ h + \frac{\bar{p}}{\rho g}, \bar{p} \right\} \]  
(7.3)

\[ h = z_{SWL} - z_{profile} \]  
(7.4)

- Resulting parallel gradients are mainly directed downwards for the higher part of the revetment, thereby in- and decreasing with the wave period. Somewhat lower at the revetment, also upward directed gradients will be present. These gradients in- and decrease with at least \( \frac{1}{4} \) wave period. A thick layer of gravel type material is able to reduce penetrating pressures and thereby pressure gradients.

- Upward directed perpendicular gradients appear simultaneously along the revetment remarkably often. Frequently these maximal perpendicular ‘destabilizing’ gradients are accompanied by a maximal parallel gradient. Upward perpendicular gradients are generated during the run-down. This means that under thick single layered revetments the breaking wave is not responsible for the most severe loading at the interface, but the run-down flow.

Get insight in the influence of different filter properties

From two properties it has been tried to identify the influence: Thickness and grading.

Because high frequent pressure fluctuations only have a small influence on the occurring gradients, it is not necessary to increase the filter thickness for the reduction of these fluctuations.

Although the analysis of two available large scale model tests show a tremendous difference in damping of pressures, both indicate an increasing damping of pressures with increasing filter layer thickness. Increasing filter thickness keeps decreasing the penetrating part of pressures. These penetrating parts are responsible for the gradients at the interface of sand underlying a single filter. The influence of filter thickness is thus evident.

The influence of grading could not be identified from the results of the tests that were available for analysis. The literature study brought up the research of Heibaum (2004)\textsuperscript{11}. This research presents a summary of a procedure originally developed by Cistin and Ziem, whereby the \( D_{f50} \)-size of the filter is selected on the coefficients of uniformity \( (D_{x60}/D_{x10}) \) of both the base soil and the filter material. By a constant coefficient of uniformity for the base material, an increasing coefficient of uniformity of the filter material makes the use of a larger \( D_{f50} \) possible. This gives the filter a more open character.

Another result that supports the use of a wide graded filter is the observation of Klar et al. (2004a) that the average velocity and the standard deviation decrease in a less accessible pore. In wide graded material pores are generally less accessible than in small graded materials, because smaller particles fill up the larger pores.

Further research is necessary for a better identification of the influence of grading.

\textsuperscript{11} According to CUR Desk Study
Explain the stable interface of sand underlying a cobble layer.

The difference between the pressure damping of the revetments in the Großer Wellenkanal and the Delta Flume is large. The revetment in the Großer Wellenkanal is fixed; the bounded layer prevents the unbounded material to move around. This results in a larger resistance against the wave action and thus a more efficient reduction of pressures than in a gravel revetment that has the freedom to find its dynamic equilibrium. However, it is believed that the large difference in performance cannot be explained completely by this difference in model configuration.

Calculated gradients at the interface between gravel and sand of the Delta Flume models are higher than traditional stability criteria. With observance of the direction and duration of the gradients from the Großer Wellenkanal tests, an erosion pattern as was documented by Uelman (2006) and Ockeloen (2007) would be expected (see Figure 7.3) in the models.

Such an erosion profile was not observed; there was no movement of sand in the Delta Flume models. Therefore, the gradients at the interface must be lower than was calculated. From the analysis of the model in the Großer Wellenkanal it seems probable that the damping of pressures is significantly better in a gravel layer than the pressure measurements of the Delta Flume models indicate.

The damping observed in the Großer Wellenkanal and the non-distorted interface in the Delta Flume models give strong indications that the measurements in the gravel material of the Delta Flume models are not representative for the performance of the gravel layer, resulting in a conservative prediction of gradients at the interface of cobbles and sand.

Several factors could have been influencing the pressures in the revetments of the Delta Flume and the reduction found in the analysis of the measurements:

- An irregular wave climate has been used; two types of load subjected to the models.
- Hydraulic conditions differ for every test step.
- The low frequency of the pressure transducers makes discrimination between impact and non-impact loads not possible, which may result in wrong impression of reduction values.
- Model configurations of series are different and thereby also the location of the pressure transducers.
- Varying filter layer thickness including varying amounts of material on top of the upper transducer for each test.
- Wall effects; pressure transducers are mounted to the wall of the flume.
Extrapolation of the pressure reduction that is achieved by the revetment of the Großer Wellenkanal shows that a filter layer thickness of $20D_{75}$ should result in almost a complete reduction of pressures. Such a reduction will result in a decrease of the size of gradients as well. Leaving scale effects and the different hydraulic conditions aside, the same layer thickness resulted from extrapolation of the measurements of Uelman (Figure 4.31). Comparing this to the filter layer thickness of the Delta Flume model configurations ($y/D_{75} > 40$), explains the stable interface in these models. An increasing reduction of penetrating pressures results in a reduction of gradients and explains the stable interface of sand underlying a single thick cobble layer.

### 7.2 Recommendations for further research

The relation between a hydraulic gradient (resulting from run-up) and a slope seems interesting for further research. Nevertheless, slopes of cobble beaches are relatively flat in general. Further research to such a relation will therefore not be in favour of the understanding of the hydraulic stability of sand underlying a cobble layer.

**Recommendations:**

- A growing reduction of pressures is expected to occur with increasing filter layer thickness. A similar test as was performed for the evaluation of Elastocoast revetments should be done, including more transducers over the vertical of the filter layer. These transducers should be able to process pressure data with $f \gg 5 H_2$ to be able to confirm the efficient reduction of impact peaks.
- Only one wave loading situation of the Elastocoast experiment has been analyzed using spectral analysis. Applying spectral analysis to several wave heights and periods may result in more insight in the pressure reduction within the filter. It may be even possible to create a formula with the variables like filter layer thickness, porosity and stone diameter which describes the reduction of the power over the frequencies.
- Formula (6.2) predicts parallel gradients from the time derivative of the pressure signal. A 1:1 relation between the pressure decrease, time derivative and thus gradients is assumed by Deltares. Further research to the validity of this assumption is necessary.
- Instead of identifying the absolute pressure reduction, it is also possible to focus the attention on the change of the time derivative of pressures. Thereby only taking into account the penetrating pressures. This can be done by removing impact peaks from signals by a despiking operation.
- The description of the influence of different grading of filter material was only possible theoretically. Tests are necessary to really identify the influence of grading on pressure and velocity.
- The use of a wide graded filter is supported by Cistin and Ziems (CUR Desk Study). However, segregation of filter material is likely to occur in wide graded materials. The analysis of samples that were taken over the profile of the Delta Flume could confirm the occurrence of segregation of the cobble material.
- Identify the influence of wave groups. Damage to coastal defences may be caused by the occurrence of runs of successive high waves.
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Appendix
Hydraulic interface stability of sand underlying a single filter layer
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Hydraulic interface stability of sand underlying a single filter layer
A. Model configurations Delta Flume tests

A.1. Hydraulic conditions of Delta Flume tests.
Hydraulic conditions of Series 2 and 5 are given in Table A.1. Hydraulic conditions of Series 3 and 4 are given in Table A.2.

<table>
<thead>
<tr>
<th>Table A.1 - The conditions for Series 2 and 5.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
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<tr>
<td>------</td>
</tr>
<tr>
<td>1</td>
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<tr>
<td>2</td>
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<tr>
<td>3</td>
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<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
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</table>

<table>
<thead>
<tr>
<th>Table A.2 - The conditions for Series 3 and 4.</th>
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<tbody>
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<td>------</td>
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<tr>
<td>1</td>
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<tr>
<td>2</td>
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<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
</tbody>
</table>

A.2. Model configurations
The next pages show model configurations of Series 2, Series 3, Series 4 and Series 5 of the models that have been tested in the Delta Flume. PT4 and PT1 are situated 10 cm in the sandy embankment. The thickness of the gravel layer is different for every configuration. The layer thickness for the evaluated configurations is presented in Table A.3.

<table>
<thead>
<tr>
<th>Table A.3 - Layer thickness of top layer.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>Series 2</td>
</tr>
<tr>
<td>Series 3</td>
</tr>
<tr>
<td>Series 4</td>
</tr>
<tr>
<td>Series 5</td>
</tr>
</tbody>
</table>

BCL stands for Basis Contour Line. This is the reference line from which the horizontal distance was measured in the Delta Flume models.
Model configuration of Series 3a is equal to Series 3b.
B. Overview of experiments

In this appendix, treated experiments in the literature study (chapter 4) will be compared by means of material and hydraulic parameters. The overviews in Table B.1 and Table B.2 have the purpose to identify contradictories that need further attention in the continuation of the study. In addition they give information to what extend the analysis of the results of the Großer Wellenkanal gives insight in the situation of the Delta Flume.

Parameter explanation

Hydraulic loading conditions can be compared by the following ratio (Table B.2):

\[ \frac{H}{\Delta D_{f50}} \]

This ratio is often used in literature to make a comparison between load and strength.

To what extend the filter can be described as geometrically open or closed is described by the following ratio (Table B.2):

\[ \frac{D_{f50}}{D_{b50}} \]

This ratio is relatively low for the Delta Flume experiment.

The following ratio can be used to describe the critical gradient for perpendicular flow:

\[ n_f \cdot \frac{D_{f15}}{D_{b50}} \]

It includes the porosity in the filter layer.
Table B.1 - Overview of parameters of the experiments (1/2).

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<tbody>
<tr>
<td><strong>Base material</strong></td>
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<tr>
<td></td>
<td>$D_{10} = 141-148 , \mu m$</td>
<td>$D_{10} = 340 , \mu m$</td>
<td>$D_{10} = 160 , \mu m$</td>
<td>$D_{10} = 140 , \mu m$</td>
<td>$D_{10} = 70 , \mu m$</td>
<td>Cement</td>
<td>$D_{50} = 1000 , \mu m$</td>
<td>Fine sand</td>
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<td>$D_{25} = 213-214 , \mu m$</td>
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<tr>
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<td>$D_{50} = 302-317 , \mu m$</td>
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<td>$D_{10} = 16 , mm$</td>
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<td>$D_{10} = 23.5 , mm$</td>
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<td><strong>Model C</strong></td>
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<td>$D_{10} = 14 , mm$</td>
<td>$D_{10} = 16 , mm$</td>
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<td>$D_{10} = 16 , mm$</td>
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## Table B.2 - Overview of parameters of the experiments (2/2).

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<tbody>
<tr>
<td>$\rho$ [kg/m$^3$]</td>
<td>2638</td>
<td>2600$^{12}$</td>
<td>2650</td>
<td>2650</td>
<td>2650</td>
<td>2540</td>
<td>2464</td>
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<td>$D_{50}/D_{95}$</td>
<td>68.5</td>
<td>76-88$^{13}$</td>
<td>112.5</td>
<td>144.4</td>
<td>209</td>
<td>-</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>m</td>
<td>37.9 - 87</td>
<td>8.3-13.5$^{14}$</td>
<td>2.3 - 11.1</td>
<td>5.77</td>
<td>3 - 5</td>
<td>1 - 5</td>
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<td>10</td>
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<tr>
<td>$n_f^*D_{50}$</td>
<td>9.3 - 9.4$^{15}$</td>
<td>18.8 - 23.5</td>
<td>38.8 - 99.5</td>
<td>35.6 - 48.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Wave height</td>
<td>$H_s = 0.55$ - 1.45 m</td>
<td>$H_{m0} = 0.67$ m</td>
<td>$H = 0.1$ m</td>
<td>$H = 0.08$ m</td>
<td>$H = 0.12$ m</td>
<td>$H = 0.14$ m</td>
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<td>Wave period</td>
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<td>$T_{m,10} = 3.61$ s</td>
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<td>$T = 1$ s</td>
<td>$T = 1.5$ s</td>
<td>$T = 2$ s</td>
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<td>-</td>
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<tr>
<td>$H/\Delta D_{50}$</td>
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<td>16.1 - 20.9</td>
<td>1.41 - 3.37</td>
<td>1.86 - 3.26</td>
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<td>0.66 - 5.30</td>
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</tr>
<tr>
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<td>1:3</td>
<td>1:2, 1:3, 1:4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

---

$^{12}$ Not specifically mentioned in report, 2600 kg/m$^3$ is assumed.

$^{13}$ A geotextile was present between the filter and the base.

$^{14}$ Bounded layer and unbounded layer.

$^{15}$ Probably this value should be smaller, because a wider grading usually results in a smaller porosity than 0.4
C. Signal of PT38

Figure C.1 shows why PT38 will not be used in the Delta Flume analysis. The time series of transducer PT38 is very chaotic. Measurements made by transducer PT38 will not be taken into account.
Hydraulic interface stability of sand underlying a single filter layer
Appendix D

In this appendix the spectral analysis of the pressure signals of the Großer Wellenkanal model is under consideration. A brief introduction is given about the subject. Thereafter the used windowing method for the plots of spectral estimators in this thesis is given. The pressure transducers have a sampling frequency of 500 Hz and time series have a length of 4200 s.

D. Spectral analysis

A spectral estimation makes it possible to estimate how the total power is distributed over frequency from a finite record of stationary data sequence. It can help in identifying possible hidden periodicities in time series.

The spectral estimation problem can be described as follows:

If \( y(t) \) is a deterministic discrete-time data sequence \( \{ y(t); t = 0, \pm 1, \pm 2, \ldots \} \) that has finite energy, then it possesses a discrete-time Fourier transform (DTFT):

\[
Y(\omega) = \sum_{t=-\infty}^{\infty} y(t)e^{-i\omega t} \tag{D.1}
\]

The samples are transformed to the frequency domain.

Let the Energy Spectral Density be the result of the squared outcome of (D.1):

\[
S(\omega) = |Y(\omega)|^2 \tag{D.2}
\]

From a finite length record \( \{ y(1), \ldots, y(N) \} \) of a second-order stationary random process, one could determine an estimate \( \hat{\Phi}_p(\omega) \) of its power spectral density \( \Phi(\omega) \). It is desirable that the estimate is as close to the power spectral density.

A definition of power spectral density is:

\[
\Phi(\omega) = \lim_{N \to \infty} E\left\{ \frac{1}{N} \sum_{t=1}^{N} y(t)e^{-i\omega t} \right\} \tag{D.3}
\]

By neglecting the expectation and the limit operation, which cannot be performed when the only available information on the signal consists of the samples, we get the estimate in the form of a periodogram:

\[
\hat{\Phi}_p(\omega) = \frac{1}{N} \left| \sum_{t=1}^{N} y(t)e^{-i\omega t} \right|^2 \tag{D.4}
\]

A periodogram has the property to fluctuate around the true power spectral density, with a nonzero variance even if the length of the processed sample increases without bound. The total squared error of the estimate is the sum of the bias squared and the variance (\( \hat{\alpha} \) being an estimate of \( \alpha \)):

\[
MSE = var \{ \hat{\alpha} \} + |bias \{ \hat{\alpha} \}|^2 \tag{D.5}
\]

It is possible to modify the periodogram to obtain better spectral estimators. These ‘improved’ methods decrease the variance of the estimated spectrum at the expense of increasing its bias (or resolution).

\[\text{Stoica et al., 1997}\]
D.2. Windowing

By introducing windowing techniques the squared error can be reduced significantly:

$$\hat{\varphi}_W(\omega) = \hat{\varphi}_p \cdot W(\omega) \quad (D.6)$$

$W(\omega)$ acts as a window (or weighting) in the frequency domain, and is called the spectral window. A window has a shape and a size $(M)$. The choice of a window’s size should be based on a trade-off between spectral resolution and statistical variance. As a rule of thumb use $M \leq N/10$ in order to reduce the standard deviation of the estimated spectrum at least three times, compared with the periodogram.

The choice of a windows’ shape (i.e. rectangular, hamming etc.) should be based on a trade-off between smearing and leakage lengths. A number of windows have been developed to address this trade-off. Each of these windows can be seen as a design at a specific point in the resolution/leakage trade-off curve.

An example of smearing is when, for instance, two peaks separated by the frequency $f$ by less than $1/N$. These peaks appear as a single broader peak in the estimated spectrum.

The principle effect of the side lobes on the estimated spectrum consists of transferring power from the frequency bands that concentrate most of the power (main lobe) in the signal to bands that contain less or no power. This effect is called leakage.

![Figure D.1 - Main lobe and side lobes (adjusted from Stoica et al., 1997).](image)

Well-known modified periodogram methods are the Bartlett- and Welch-methods. They seek to reduce the variance of the periodogram by smoothing or averaging the periodogram estimates in some way.

D.2.1. Bartlett-method

The Bartlett method reduces the large fluctuations of the periodogram. It splits up the available sample of $N$ observations into $L = N/M$ subsamples of $M$ observations each, and then average the periodograms obtained from the subsamples for each of $\omega$. A rectangular window is used. The resolution afforded should be on the order of $1/M$. So, the spectral resolution is reduced by a factor $L$, compared to the original periodogram method. In return, a reduced variance can be expected. In fact it can be shown that the Bartlett method reduces the variance of the periodogram by the same factor $L$. The compromise between resolution and variance when selecting $M$ (or $L$) is thus evident.
D.2.2. Welch-method

The Welch method is obtained by refining the Bartlett method in two respects. First, the data segments in the Welch method are allowed to overlap. Second, each data segment is windowed prior to computing the periodogram. When no overlap is used in combination with a rectangular window, you simply approach the Bartlett method. The overlap recommended in the Welch method is 50%. The standard window used by Matlab is the Hamming window. Plots in this research will be generated by the Welch-method, because this gives spectral estimates with a lower bias and a higher resolution then the periodogram which results from the Bartlett-method.
D.3. **Analysis of power spectral density estimates**

In this section an elaboration of the chosen method for obtaining the power spectral density estimates will be given. Thereafter the results of the pressure measurements of the Großer Wellenkanal will be analyzed. This may give a view at what happens within the filter.

**D.3.1. Elaboration of chosen method**

In section D.2 two well-known windowing methods were introduced; the Bartlett and the Welch-method. The latter results in the spectral estimate with the least variance in combination with the lowest loss in resolution. That is why the Welch-method has been chosen to create the power spectral density estimates.

Although the process of creating a spectral estimate is quite complex, Matlab (v7.7 R2008b) has a function which makes the generation of an estimate rather quick (pwelch). This basic Matlab code uses a Hamming window (Figure D.2).

![The Hamming window](image)

**Figure D.2 – The Hamming window**

The additional parameters that Matlab needs to perform calculations are the window size and the overlap percentage. The lower bound of the frequencies is restricted by the size of the window. When a window of 200 seconds is chosen, only periodicities of at least 0.005 Hz can be found. Thus, a larger window seems interesting. However, as was explained in section D.2, a larger window also results in a higher variance. This is illustrated in Figure D.3.
The decrease in variance results in a lower resolution and less information about the power in lower frequencies. Looking at the right plot in Figure D.3 (window size of \(2000 \text{s}\)), the high power for lower frequencies is striking and not a good impression of the real power of those lower frequencies. Aliasing effects and a slow increase of the average pressure over a time series causes such high powers. Moreover, for particle instability the higher frequencies are much more interesting. A choice has to be made between the loss in information at lower frequencies (at the favour of a reduction in variance) and a loss in resolution.

For the time series of the transducers in the Großer Wellenkanal a window of 10000 points (or \(10000 \cdot \left(\frac{1}{500}\right) = 20\text{s}\)) appears to be appropriate in combination with 50% overlap. For the same signal as has been used in Figure D.3, this results in the power spectral density estimate as can be seen in Figure D.4.
At around 50 Hz there is a sudden drop in power. This is caused by a low-pass filter, which removes frequencies higher than 50 Hz from the signal. In further plots, the horizontal axis will be plotted from $10^{-1}$ Hz until $5 \cdot 10^4$ Hz, which includes the inertial range of turbulent eddies (see section 4.4.2). The vertical axis will be plotted from $10^{-8} \text{kPa}^2 / \text{Hz}$ until $10^2 \text{kPa}^2 / \text{Hz}$.

The peak of the power lies at $f_1 = 0.24 \text{ Hz} = 4.17 \text{ s}$, which is slightly longer than the $T_{\text{nom}} = 4.0 \text{ s}$ and the measured $T_{m-1,0} = 3.61 \text{ s}$. Minor peaks are visible at higher orders; $f_2 = 0.48 \text{ Hz}$ and $f_3 = 0.72 \text{ Hz}$. They appear in all spectra, which is straightforward because the measurements were done simultaneously using the same incident waves. They are called the higher harmonics and are resulting from the Fourier transform that is necessary for creating spectral estimates.

As a check for validity of the spectrum the area below the spectrum should be equal to the variance of the signal. The variance of signal PT5 appears to be equal to $1.63 \text{ kPa}$. The surface area below the power spectral density is equal to $1.66 \text{ kPa}$, which is only 1.8% more than the actual variance.
D.3.2. Reduction of power by the revetment

The change in power spectral density estimates of the pressure measurements above, within and below the revetment makes it possible to identify the influence of the revetment on the turbulence created by breaking waves. Therefore representative spectra of layer 1, layer 2 and layer 3 need to be found. Model B may give a representative result for the spectra of layer 1 and layer 2. Model C may give a representative result for the spectra of layer 2 and layer 3.

![Model B and Model C](image-url)

*Figure D.5 - Model B and Model C (LWI Report no. 988, 2010).*

First the estimates of the spectra of the signals of layer 1 will be plotted. These are not all the same, because the character of the wave changes along these transducers from a non-broken wave to a run-up tongue. In Figure D.6 the spectra of all transducers on top of the bounded revetment can be found.
Hydraulic interface stability of sand underlying a single filter layer

Figure D.6 – Power spectral density estimates of layer 1 (Model B).

A representative spectrum can be formed with the signals that are the most powerful. In Figure D.6 it can be seen that spectra of transducers PT30 and PT31 imply that the impact of the breaking waves is located around these points. For that reason the representative spectrum of layer 1 is generated by averaging the spectral estimates of the signals of transducers PT30 and PT31.

For layer 2, Model B and Model C both have appropriate sensors. First a closer look is taken at the spectral estimates of the pressure signals of Model B. They can be found in Figure D.7.
Figure D.7 shows that the estimates of the spectra of PT36 and PT37 look very different between frequencies $f = 0.8 \text{ to } 2\, \text{Hz}$. As can be seen in Appendix C, PT38 unfortunately didn’t deliver a proper signal. For that reason it cannot be used in the spectral analysis. From the transducers that were mounted above the revetment it is known that the impact was located between PT30 and PT31. Therefore it seems logic to continue with the spectra of PT37. Because of the large difference between the spectra of the transducers, there is doubt about the validity of the measurements. Therefore, the transducers in Model C are checked first.
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**Figure D.8 -** Power spectral density estimates of transducers in Model C just below the bounded layer.

The spectra of PT4, PT5 and PT6 are the most powerful ones and show a $f^{-7/3}$-tail. These are selected in forming a representative spectrum at the location between the unbounded and the bounded layer by taking the average. Plotting this representative spectrum in the spectral estimates of layer 2 of Model B confirms the strange results of signals of the transducer in Model B (Figure D.9).
The clear difference in the spectra of Model B and Model C can not only be declared by the different type of material used in the Elastocoast composite. Transducer PT38 already was skipped in the analysis, because the transducer didn’t deliver a proper signal. The representative spectrum of Model C will be used in further analysis as representative for layer 2.

Next, the transducers at the interface between geotextile and unbounded layer are analyzed.
It should be clear from Figure D.10 that transducers PT11 and PT12 will be used in the continuation of the analysis. By averaging the spectra of PT11 and PT12 a representative result is obtained for layer 3.

By plotting all representative spectra in one figure it should be clear how the filter reduces turbulence intensities of different frequencies.
E. Critical parallel gradients

E.1. Cyclic: Waterloopkundig laboratorium (1970)\textsuperscript{17}
On basis of measurements in WL (1969)\textsuperscript{18} a critical gradient of $i_c = 0.2$ was chosen.

E.2. Cyclic: Klein Breteler et al. (1992)\textsuperscript{19}
This report contains results of tests with cyclic currents in a wave tunnel. The moment of incipient motion was defined as the situation in which a layer of a thickness of a particle is in movement everywhere and was determined visually. Klein Breteler concluded that currents regarding incipient motion can be taken as quasi-stationary when the wave period is longer than 2 seconds. The normative gradient is than equal to the maximal occurring gradient in a wave period. Critical gradients $i_c = 0.1 - 0.15$.

In WL (1969) results of experiments for stationary currents are described. For different ratio’s of $D_{bs5}/D_{bs0}$ the transport of sand was measured. The critical gradient was determined by extrapolating the measurements to transport = 0. For $D_{bs5}/D_{bs0} = 69$, $i_c = 0.2$.

E.4. Stationary: De Graauw (1983)\textsuperscript{21}
A parallel critical shear velocity can be determined with the theory of Shields (section 4.3.2):
\[
\Psi_c = \frac{\tau_c}{(\rho_s - \rho_w)gD} = \frac{u_{c2}^2}{\Delta gD} \tag{E.1}
\]
De Graauw (1983)\textsuperscript{22} stated that critical velocities and gradients for cyclic parallel flow with periods longer than two seconds are the same as for uniform parallel flow. A relation was formed by linking the Forchheimer equation (E.4) with the threshold of movement given by Shields. This resulted in an empirical relation:
\[
i_c = \left[ 0.06 \frac{n_f^{5/3} D_{f15}^{1/3}}{n_f^2 D_{f15}^{5/3} + 1000 D_{bs0}^{5/3}} u_{c2}^2 \right] \tag{E.2}
\]
For $n_f = 0.4$, $D_{f15} = 0.02$ m, $\Psi_c = 0.05$ and $D_{bs0} = 213 \cdot 10^{-6}$ m (choosing the bed material of the Delta Flume experiment as reference) results in a critical gradient $i_c = 0.04$, without a correction for a 1:3 slope. Including this correction $i_c = 0.02$.

\textsuperscript{17} According to Loman (2009b)
\textsuperscript{18} Waterloopkundig laboratorium, 1969, Filteropbouw havendammen – deel III – kritieke verhangen, M905 Rapport modelonderzoek
\textsuperscript{19} According to Loman (2009b)
\textsuperscript{20} According to Loman (2009b)
\textsuperscript{21} According to Schiereck (2001)
\textsuperscript{22} According to Schiereck (2001)
E.5. **Stationary: Klein Breteler (1987)**\(^{23}\)

Klein Breteler (1987) found the following formula for the critical velocity:

\[
U_{f,c} = \left( \frac{n_{f}}{c} \left( \frac{D_{15}}{v} \right)^{m} (\Psi_{b} \Delta \beta \alpha D_{b50})^{0.5} \right)^{1/(1-m)}
\]  \hspace{1cm} (E.3)

\(c, \Psi_{b}\) and \(m\) are dependent of the base material. For \(D_{b50} = 0.213\ mm:\)

\[c = 0.71\]
\[\Psi_{b} = 0.055\]
\[m = 0.18\]

The gradient can be determined with the formula Forchheimer’s equation:

\[
i = au_{f,c} + bu_{f,c}^{2}
\]  \hspace{1cm} (E.4)

\[a = \frac{160v(1 - n_{f})^{2}}{gn_{f}^{2}D_{f15}^{2}}\]  \hspace{1cm} (E.5)

\[b = \frac{2.2}{gn_{f}^{2}D_{f15}}\]  \hspace{1cm} (E.6)

Which result in \(i_{c} = 0.04\).  

\(^{23}\) According to CUR 161

In Klein Breteler et al. (1992) a design graph is given.

For $D_{f35} = 20 \text{ mm}, D_{b50} = 0.213 \text{ mm}, n_f = 0.4$ and $\cot \alpha = 3; i_c = 0.05 - 0.1$.

E.7. Summary

<table>
<thead>
<tr>
<th>Method</th>
<th>Loading</th>
<th>$i_c$</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterloopkundig laboratorium (1970)</td>
<td>Cyclic</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Klein Breteler et al. (1992)</td>
<td>Cyclic</td>
<td>0.1</td>
<td>From test with smaller $D_{f50}/D_{b50}$</td>
</tr>
<tr>
<td>Waterloopkundig laboratorium (1969)</td>
<td>Stationary</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>De Graauw et al. (1983)</td>
<td>Stationary</td>
<td>0.02</td>
<td>With slope correction</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.04</td>
<td>Without slope correction</td>
</tr>
<tr>
<td>Klein Breteler (1987)</td>
<td>Stationary</td>
<td>0.04</td>
<td></td>
</tr>
<tr>
<td>Klein Breteler et al. (1992)</td>
<td>Stationary</td>
<td>0.05-0.1</td>
<td></td>
</tr>
<tr>
<td>Expected behaviour</td>
<td>$z_{25} [\text{-}]$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-------------------------</td>
<td>---------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No movement</td>
<td>&lt; 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Movement no movement</td>
<td>0.05 - 0.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Movement</td>
<td>&gt; 0.15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
E.8. Analysis of gradients

In this section the phase differences between the parallel and perpendicular gradients will be analyzed. A closer look will be taken at the gradients working at the lower part, the middle part and the higher part of the revetment separately.

![Figure E.2 - The lower part (A), middle part (B) and higher part of the revetment (C).](image)

**E.8.1. The lower part of the revetment**

![Figure E.3 - Gradients between transducers PT3, PT10, PT11 and PT12.](image)

In Figure E.3 the perpendicular and parallel gradients are plotted for the lower part of the revetment. A positive gradient for the parallel case is pointed in downward direction. A negative gradient for the perpendicular case is pointed upwards. Time series of these pressure transducers are plotted in Figure E.4 for
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the same time frame to be able to get a better view on the cause of the gradients. The vertical black lines in Figure E.3 highlight the same moment in time as the red lines in Figure E.4.

Parallel gradients between PT10-PT11 are changing direction after half the wave period. Parallel gradients between PT11-PT12 are about ¾ of the wave period directed in downward direction and ¼ of the period directed upward.

![Time-series of PT3 and PT11](image1.png)

![Time-series of PT3 and PT11](image2.png)

![Time-series of PT3 and PT11](image3.png)

Figure E.4 - Time series of pressure transducers PT3, PT10, PT11 and PT12.

When the upward perpendicular gradient is at its maximum, the parallel gradient between transducers PT10 and PT11 is mostly maximal as well. This is due to another incoming wave, which causes a high pressure at PT10. At that moment almost no difference in pressure is present between transducers PT11 and PT12. A maximum gradient in the direction of the wave between transducers PT10 and PT11 coincides with a negligible pressure difference which is pointed against the direction of the wave between transducers PT11 and PT12. This is indicated with the red lines in Figure E.4. However, a fraction earlier the gradient between PT11 and PT12 is maximal in downward direction accompanied by a significant upward perpendicular gradient.

The maximum upward perpendicular gradients between PT3 and PT11 are often in phase with the maximum upward directed parallel gradients between PT10 and PT11. A maximum ‘stabilizing’ perpendicular gradient
corresponds repeatedly with maximal upward gradient between transducers PT11 and PT12. At the same moment there is negligible gradient between PT10 and PT11. A maximum downward directed parallel gradient between PT11 and PT12 is accompanied by a lift force and a small downward directed gradient between PT10 and PT11.

**E.8.2. The middle part of the revetment**

![Diagram](image.png)

In Figure E.5 the perpendicular and parallel gradients are plotted for the middle part of the revetment. Time series of these pressure transducers are plotted in Figure E.6 for the same time frame. The vertical black lines in Figure E.5 highlight the same moment in time as the red lines in Figure E.6. The fluctuations in the signals look comparable with the high frequent fluctuations in the lower part of the revetment. From chapter 5.1.2 it is known that these fluctuations are only a fraction of the pressure and are reduced by the revetment. This results in an increased fluctuation of the perpendicular gradients and in smaller fluctuations in the parallel gradients.

Parallel gradients between PT11-PT12 are about \( \frac{3}{4} \) of the wave period directed in downward direction and \( \frac{1}{4} \) of the period directed upward. Parallel gradients between PT12-PT13 are almost continuously directed downwards, but still increase and decrease in size with the wave period.
During the maximum destabilizing perpendicular gradient, the parallel gradient between transducer PT11 and PT12 is close to zero. This already was noticed from Figure E.4. Between transducers PT12 and PT13 a gradient in downward direction is present (almost continuously), which is frequently at its maximum during the maximum upward gradient.

Just after the maximal upward perpendicular gradient, the gradient between transducers PT11 and PT12 is growing pointed in the direction of the wave (upward), just as was concluded from Figure E.3 and Figure E.4. This growing gradient goes along with a decreasing gradient between PT12 and PT13, which appears almost constantly larger than zero. A maximum downward directed gradient between transducer PT11 and PT12 is accompanied by a lift force.
E.8.3. The higher part of the revetment

In Figure E.7 the perpendicular and parallel gradients are plotted for the higher part of the revetment. Time series of these pressure transducers are plotted in Figure E.8 for the same time frame. The vertical black lines in Figure E.7 highlight the same moment in time as the red lines in Figure E.8.

The damping of fluctuations at this location of the revetment is the highest (Table 5.2), which causes the high frequent fluctuations of the perpendicular gradient. These fluctuations are not significantly larger than the fluctuations that were visible in Figure E.3 and Figure E.5. The range of the vertical axis is somewhat smaller in Figure E.7.
There continuously appears to be a parallel gradient between transducers PT13 and PT14, pointed in downward direction. Looking at the times series of PT14 it is concluded that this is caused by the wave set-up, which was not present during resetting the transducers. This results in an overestimation of the gradient between PT13 and PT14.

The maximum 'destabilizing' upward gradient in perpendicular direction is frequently in phase with the parallel downward gradient between PT12 and PT13, but at the same time the gradient between PT13 and PT14 also appears to be maximal in downward direction.

In Figure E.9 all parallel gradients are given in one overview. In Figure E.10 all perpendicular gradients are given and one parallel gradient for checking the phase difference.
Figure E.9 - All parallel gradients in one overview.

Figure E.10 - All vertical gradients and one parallel gradient.
E.9. Analysis of gradients (different time range)
In this section, the same analysis as in section E.8 is done for a different time range.

E.9.1. The lower part of the revetment

The gradients working at the lower part of the revetment are plotted in Figure E.11. The same time frame of the time series of the transducers is given in Figure E.12. The black lines in Figure E.11 correspond to the red lines in Figure E.12.
A maximum upward perpendicular gradient occurs simultaneously with a maximum gradient between transducers PT10 and PT11 in wave direction and a negligible parallel gradient between PT11 and PT12.

A maximum downward gradient between PT11 and PT12 is accompanied by a small lift force.
E.9.2. The middle part of the revetment

The gradients working at the middle part of the revetment are plotted in Figure E.13. The same time frame of the time series of the transducers is given in Figure E.14. The black lines in Figure E.13 correspond to the red lines in Figure E.14.
A maximum upward perpendicular gradient occurs simultaneously with a maximum parallel downward gradient, which is pointed in against the wave direction. The gradient between PT11 and PT12 is negligible, when the maximum upward gradient in perpendicular direction has been reached.

Between PT11 and PT12 a maximum downward parallel gradient is accompanied by a small lift force.
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E.9.3. The higher part of the revetment

The higher part of the revetment is shown in Figure E.15. The gradients working at the higher part of the revetment are plotted in Figure E.15. The same time frame of the time series of the transducers is given in Figure E.16. The black lines in Figure E.15 correspond to the red lines in Figure E.16.

**Figure E.15 - Gradients between transducers PT7, PT12, PT13 and PT14.**

The gradients working at the higher part of the revetment are plotted in Figure E.15. The same time frame of the time series of the transducers is given in Figure E.16. The black lines in Figure E.15 correspond to the red lines in Figure E.16.
The maximum upward gradient is accompanied by a maximum gradient between PT12 and PT13 in downward direction. The gradient between PT13-PT14 is continuously causing a downward parallel gradient.

Figure E.16 - Time series of transducers PT7, PT12, PT13 and PT14.
F. Signal analysis Delta Flume model

F.1. Impact peaks in signal

The transducers on top of the revetment of the Großer Wellenkanal model show impact loads with $p \gg 5 \, \text{kPa}$ (section 5.1.1). Looking at Figure F.1, where the pressure signal of a Delta Flume transducer close to the surface of the revetment is plotted, no peaks in the order of $> 5 \, \text{kPa}$ are present in the signal.

![Figure F.1 - Signal of pressure transducer PT3 in Delta Flume Series 4 Test 1.](image)

Peaks in the signal of the Großer Wellenkanal are caused by impact peaks of impact loads.

Hydraulic conditions of Series 4 Test 1:

\begin{align*}
H_s &= 0.71 \\
T_{m-1.0} &= 3.88 \\
\xi_{m-1.0} &= 1.14
\end{align*}

The surf similarity parameter indicates that plunging waves can be expected. Probably, the material on top of PT3 prevents the impacts to penetrate into the gravel layer. Figure F.2 zooms in at one of the maximum pressures of the signal plotted in Figure F.1.
Figure F.2 – Zoom in at impact part of pressure signal.

This looks like a reduced impact peak, but due to the low frequency of the pressure transducer the impact can also be missed. The latter would mean that the actual pressure signal looks rather different than recorded by the pressure transducers of the Delta Flume.
F.2. Overview of tests

In Table F.1 all available tests are indicated. Further it is indicated:

- if the SWL is above transducer PT3,
- where the impact can be expected using Klein Breteler’s formula (6.1),
- what the difference is between the location of PT3 and the location of impact,
- how much material is present on top of PT3.

<table>
<thead>
<tr>
<th>Test</th>
<th>PT3 under SWL</th>
<th>$z_{pmax}$ [m]</th>
<th>Location PT3 below SWL [m]</th>
<th>Difference</th>
<th>$y/D_{50}$ above PT3 [-]</th>
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<tbody>
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<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
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<td>-</td>
<td>-</td>
<td>-</td>
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</tr>
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<td>-</td>
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<td>-</td>
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<tr>
<td>S3bT1</td>
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<td>0.30</td>
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<tr>
<td>S4T2</td>
<td>Yes</td>
<td>0.24</td>
<td>0.36</td>
<td>0.12</td>
<td>17.49</td>
</tr>
<tr>
<td>S4T3</td>
<td>Yes</td>
<td>0.23</td>
<td>0.53</td>
<td>0.30</td>
<td>19.85</td>
</tr>
<tr>
<td>S4T4</td>
<td>Yes</td>
<td>0.27</td>
<td>0.81</td>
<td>0.54</td>
<td>26.37</td>
</tr>
<tr>
<td>S4T5</td>
<td>Yes</td>
<td>0.21</td>
<td>0.60</td>
<td>0.39</td>
<td>27.25</td>
</tr>
<tr>
<td>S5T1</td>
<td>Yes</td>
<td>0.01</td>
<td>0.37</td>
<td>0.36</td>
<td>13.73</td>
</tr>
<tr>
<td>S5T2</td>
<td>Yes</td>
<td>0.01</td>
<td>0.43</td>
<td>0.42</td>
<td>15.01</td>
</tr>
<tr>
<td>S5T3</td>
<td>Yes</td>
<td>0.02</td>
<td>0.60</td>
<td>0.58</td>
<td>16.08</td>
</tr>
<tr>
<td>S5T4</td>
<td>Yes</td>
<td>0.05</td>
<td>0.88</td>
<td>0.84</td>
<td>22.18</td>
</tr>
<tr>
<td>S5T5</td>
<td>Yes</td>
<td>0.01</td>
<td>0.67</td>
<td>0.66</td>
<td>24.47</td>
</tr>
</tbody>
</table>
### G. Formula verification Deltares

Deltares uses the formula \( (G.1) \) for calculating the parallel gradient by using the time series of only one pressure transducer. Hereby, the gradient \( i \) is derived from the time derivative of the pressure \( \frac{\delta p}{\delta t} \).

\[
i = \frac{1}{\rho g} \frac{\delta p}{\delta x} = \frac{1}{\rho g} \frac{\delta p}{\delta t} c \quad \text{(G.1)}
\]

\[
c = \sqrt{gh'} \quad \text{(G.2)}
\]

The pressure measurements of the transducers above and below the unbounded layer, performed in the Großer Wellenkanal, will be used to evaluate the formula. These transducers can also be used to calculate the parallel gradients that really occur. Because pressure signals on top of the unbounded layer still show (reduced) impact peaks and high frequent fluctuations, unlikely high gradients are expected (time derivatives are very high) using formula \( (G.1) \). Probably, using the moving average of the transducer below the unbounded layer will result in more realistic estimates of gradients.

The local wave velocity can be calculated using the local erosion profile from the beginning of the experiment, the nominal water table during the test and the average pressure during the experiment. When the water table is higher than the profile than the wave velocity is based on the average pressure height and the local water depth. In the case of negative local water depth the wave velocity is based on the average pressure height. The water depth and the resulting wave velocity on the position of the transducer are varying only during the experiment.

\[
h' = \max \left\{ h + \frac{\bar{p}}{\rho g}; \frac{\bar{p}}{\rho g} \right\} \quad \text{(G.3)}
\]

\[
h = z_{SWL} - z_{profile} \quad \text{(G.4)}
\]

In the Deltares report the following gradients per test and per pressure transducer have been calculated:

- \( i_{max} \) for the maximum positive and minimum negative gradient.
- The average of the 2% highest positive gradients and the average of the 2% smallest negative gradients.

Thereafter the maximum gradient \( i_{max} \) has been taken as the maximum of the absolute highest positive gradient and the absolute smallest negative gradient and \( i_{2\%} \) as the average of the absolute 2% highest positive gradients and the absolute 2% smallest negative gradients. The same gradients will be calculated using formula \( (G.1) \).

Results for the unmodified pressure signals are presented in Figure G.1.
Figure G.1 - Gradients calculated using formula (G.1) using the whole time series.

As expected, the gradients are unrealistically high. Table G.1 indicates the actual gradients, which have been presented in section 5.3.3. Especially $i_{\text{max}}$ in Figure G.1 has been overestimated. Gradients indicated in Table G.1 are located between the transducers, gradients obtained by applying formula (G.1) are an indication of the gradient at the location of the pressure transducer.

<table>
<thead>
<tr>
<th>Pressure transducers</th>
<th>$i_{2%}$</th>
<th>$i_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT3-PT5</td>
<td>0.37</td>
<td>1.62</td>
</tr>
<tr>
<td>PT5-PT7</td>
<td>0.31</td>
<td>1.62</td>
</tr>
<tr>
<td>PT10-PT11</td>
<td>0.18</td>
<td>0.25</td>
</tr>
<tr>
<td>PT11-PT12</td>
<td>0.21</td>
<td>0.83</td>
</tr>
<tr>
<td>PT12-PT13</td>
<td>0.27</td>
<td>0.34</td>
</tr>
<tr>
<td>PT13-PT14</td>
<td>0.18</td>
<td>0.20</td>
</tr>
</tbody>
</table>

As already was mentioned before, signals of the pressure transducers of the Delta Flume models and the Großer Wellenkanal model look somewhat different. The high frequent fluctuations are not present in the signals obtained from the Delta Flume tests, probably because the frequency of the transducers was rather low (see Figure G.2).
The small-scale fluctuations, which have been analyzed in section 5.3.3, result in very large time derivatives. Fluctuations do not appear in the Delta Flume pressure signals. For a fair evaluation of the formula, gradients are calculated again, now using the moving average of the transducers PT11, PT12 and PT13 (see Figure G.3), which do not show any impact peaks nor fluctuations.

Comparing the results presented in Figure G.3 to the results in Table G.1 it can be concluded that both the calculated $i_{2\%}$ and the $i_{\text{max}}$ are a reasonable representation of the gradients that occur in reality (see Table G.2 and Table G.3).
Table G.2 – Comparing gradients

<table>
<thead>
<tr>
<th>Location</th>
<th>$t_2$%</th>
<th>$t_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT11</td>
<td>0.20</td>
<td>0.79</td>
</tr>
<tr>
<td>PT12</td>
<td>0.24</td>
<td>1.05</td>
</tr>
<tr>
<td>Between PT11-PT12</td>
<td>0.21</td>
<td>0.83</td>
</tr>
</tbody>
</table>

Table G.3 – Comparing gradients

<table>
<thead>
<tr>
<th>Location</th>
<th>$t_2$%</th>
<th>$t_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT12</td>
<td>0.24</td>
<td>1.05</td>
</tr>
<tr>
<td>PT13</td>
<td>0.14</td>
<td>0.93</td>
</tr>
<tr>
<td>Between PT12-PT13</td>
<td>0.27</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table G.2 shows rather good results, but Table G.3 shows that the gradient of PT13 is underestimated. Nevertheless, gradients expected by formula (G.1) can be considered realistic.

Because the measurement in the Delta Flume do not show any impact peaks nor fluctuations, the use of formula (G.1) will result in reasonably good predication of the actual gradients and is therefore justified. Because of the large influence of time derivatives, the appearance of impact peaks should be minimal when using the formula. Therefore it is recommended that the formula is only applied to transducers which are situated deeper in the revetment (like PT2).