# CHAPTER 126

# WATER PARTICLE VELOCITIES ON A BERM BREAKWATER

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## ABSTRACT

Water particle velocities in waves running up and down a berm breakwater have been measured for several wave heights and wave periods with a Laser Doppler Velocimeter (LDV).

The measured water particle velocities have been compared with velocities computed with the numerical model ODIFLOCS. There is a fair agreement between the calculated and measured velocities.


## 1 INTRODUCTION

It is the velocity field on a breakwater front that is the main governing factor with respect to the stability of the armour cover blocks. This velocity field and the forces on a cover block have been poorly known.

The different formulae that have been presented on the required block weight, e.g. Iribarren (1938), Hudson (1985) and Hedar (1960) have been based on some approximate concept of the velocities and the forces, leading to formulaes with a single unknown coeffisient. The value of this coefficient has been determined from model tests.

One of the first attempts to calculate and measure the velocities for downrush on a rubble mound breakwater model was made by Brandtzæg and Tørum (1966), Brandtzæg, Tørum and Østby (1968). They measured velocities


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with a micro propeller. The measurements were made at one height above the slope and gave no details about the velocity variation from the slope face and up towards the water surface. There was a fair agreement between the measurements and the simple mathematical model that was derived to calculate the velocities in downrush.

Sawaragi et al. (1982) measured particle velocities on the breakwater slope by filming particles made of sponge with the same specific mass as water introduced in the water. The point to point movement of the particles was recorded on 16 mm colour films taken by a high speed film camera (50 frames per sec). From the film the particle velocities were obtained by superposition of projected film frames to give a distance and a time interval of movement. Sawaragi et al. found that the non-dimensional maximum velocity was a function of the surf similarity parameter and the ratio of the wave height to the water depth. Sawaragi et al. did not compare their velocity measurements with any theoretical results.

Kobayashi et al. (1987) and Kobayashi and Wurjanto (1989) developed a numerical model for the computation of the water particle velocities on an impervious rubble mound slope. This model is based on the finite amplitude shallow water wave equation. By use of this model they can calculate the vertically averaged horizontal velocities as well as run up and run down. They compared the calculated run up with measurements, which showed fair agreement, but did not make any comparisons between calculated and measured velocities.

Breteler and van der Meer (1990) report the measurement and computation of wave induced velocities on a smooth slope. The measurements were made with an electromagnetic current meter. The computations were made with the computer program developed by Kobayashi and Wurjanto (1989). Breteler and van der Meer concluded that there was a fair agreement between the measurements and the computations with respect to run up levels and run down velocities, the results for run up velocities were a little worse and the results for pressures and run down levels were bad.

Laser doppler velocity meters (LDV) offers the possibility to make good velocity measurements without any interference with or disturbance of the fluid. Since no detailed velocity measurements have been carried out as the waves run up and down a berm breakwater slope it was decided to carry out such measurements. The results have been compared with results obtained by the computer program ODIFLOCS, van Gent (1992).

## 2 TEST SET UP AND MEASUREMENT SYSTEMS

### 2.1 Wave flume and berm breakwater model

The measurements were carried out in a wave flume with the berm breakwater as shown in Figure 1. The width of the flume was 1.0 m.

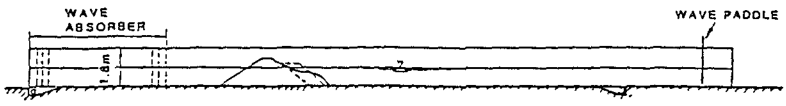


Figure 1 The wave flume with the breakwater model.

The breakwater cross section is shown in Figure 2. The shown section of the reshaped breakwater was obtained by using waves with heights up to 0.25 m.

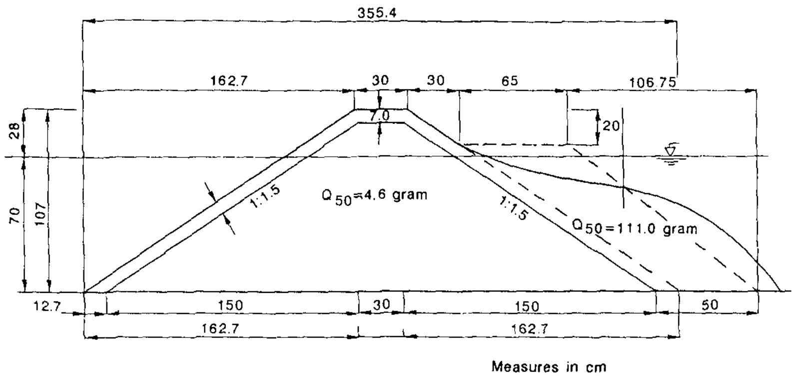


Figure 2 Berm breakwater cross section.

## 2.2 Laser Doppler Velocimeter (LDV)

The water particle velocities were measured with a Laser Doppler Velocimeter (LDV). The LDV system is a two component system based on the forward scatter mode. This LDV system was built in-house for a study of the kinematics of irregular water waves (potential flow). The noise to signal ratio was too large for this instrument to give any meaningful measurements of turbulence. The measurements were taken with a rate of 100 samples per second. In the analysis the data have been smoothed by using a gliding average of 11 data points.

The velocities were taken at several of the points shown in Figure 3. The main reason for concentrating the velocity measurement points in the area shown in Figure 3 was that less air entrainment due to breaking waves was expected in this area

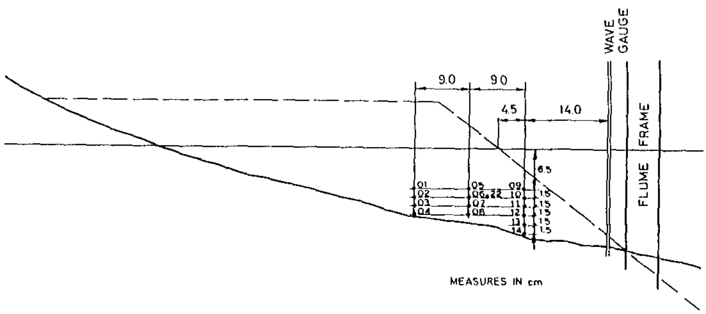


Figure 3 Velocity measurement points.

than closer to the breakwater crest. Air entrainment causes drop outs of the LDV measurements. Another reason is that it may be expected that the destructive velocities and forces downslope are largest in this region where the breakwater slope is flattest.

The berm breakwater profile shown in Figure 3 gives an average profile along the glass panel wall of the flume. The distance between this profile and the lowest measuring point is not necessarily representative for the distance between the measurement point and the closest stone. The measurement points are located approximately in the middle of the wave flume. The LDV system was orientated such that the velocities in horizontal and vertical direction was measured. However, during the analysis the instantaneous velocities in any direction could be obtained. In this paper velocities parallel and normal to the breakwater slope are given. Positive parallel velocities mean uprush while positive normal velocities mean velocities away from the slope.

## 2.3 Wave measurements

The waves were measured with wave gauges of the conductivity type. Prior to the velocity measurement runs the waves were calibrated in the wave flume. During the wave calibration runs the waves were measured in an area approximately 5 m ahead of the breakwater model. All the tests during the water particle velocity measurements were carried out with regular waves. Hence the waves were calibrated by moving a wave gauge along the wave flume to obtain the maximum and minimum wave heights. The height of the incoming wave was then set as the average of the maximum and minimum wave height.

During the velocity measurement runs the waves were measured close to the velocity measurement points, Figure 3. The main purpose of this gauge is to give phase information between the wave elevation and the velocity measurements. The wave elevation measurements at this gauge may be inaccurate, partly because of the shallow water and partly by air entrainment during the breaking of the largest waves.

## 3 VELOCITY MEASUREMENTS - ANALYSIS AND RESULTS

Water particle velocities have been measured primarily at the measurement points, see Figure 3: 08, 09, 10, 11, 13 and 22 for the three wave periods T = 1.5, 1.8 and 2.1 s for several wave heights for each wave period. Not all data have been analysed, but we will present some main features of the analysis. Figure 4 shows waves measured at the location of the reference wave gauge shown in Figure 3. Figures 5 and 6 show parallel and normal velocities in point 08 measured simultaneously with the waves. Figure 7 shows a time expanded diagram of the wave and the parallel and normal velocity at point 08. The time reference is the same as in Figures 4, 5 and 6. Further details on the measurements are given by Tørum (1992).

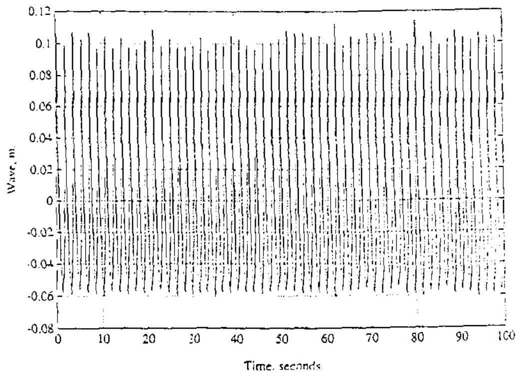


Figure 4 Measured wave at the location of the reference wave gauge shown in Figure 3.

Although the waves are "regular" there are slight variations in their heights at the reference gauge. In this case the waves broke after they passed the wave gauge and the measurements are not influenced by any air entrainment.

There are also slight variations in the parallel velocities. It is though not necessarily such that a high wave generate a large uprush velocity. The normal velocities are more irregular than the parallel velocities.

The maximum, mean and minimum velocities measured in the points 08, 09, 10, 13 and 22 are shown in Tables 1 and 2 for uprush and downrush. Point 9 "went dry" during downrush. Hence no "maximum" downrush velocities were taken for this point.

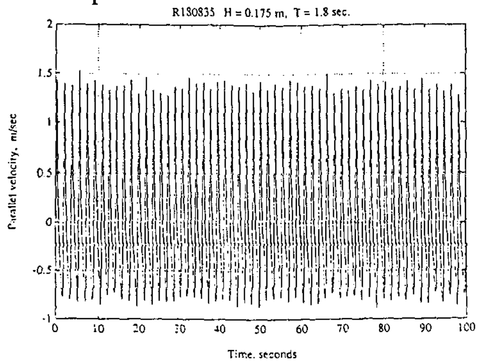


Figure 5 Measured parallel velocity at point 08. Positive velocity means uprush.

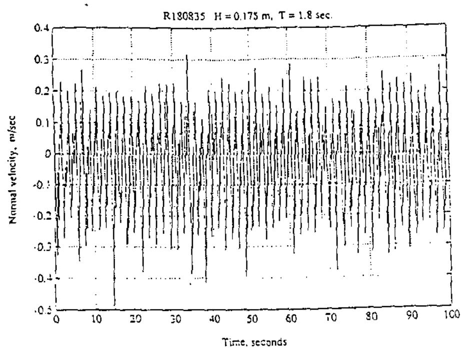


Figure 6 Measured normal velocity at point 08. Positive velocity means velocity away from the slope.

There is a tendency that the maximum parallel velocities in uprush are largest closest to the berm breakwater slope. This might be due to amplification effects close to cover stones or overshoot effects in the wave boundary layer.

Since the velocities were not measured simultaneously in the different points it is not possible to draw a "true" velocity profile through the measurement points 09, 10 and 13. An order of magnitude analysis indicates that the boundary layer thickness is 0.01 - 0.015 m during maximum velocities.

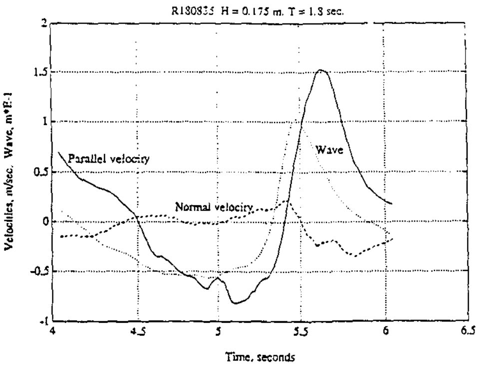


Figure 7 Measured velocities in point 08 and wave at reference wave gauge. Positive velocity means uprush.

Table 1 Minimum, mean and maximum parallel velocities in m/sec during uprush and downrush. H = 0.175 m, T = 1.8 sec.

| Measure-ment point | $u_{up}$ | | | $u_{down}$ | | |
|---|---|---|---|---|---|---|
| | min | mean | max | min | mean | max |
| 09 | 0.6 | 1.0 | 1.25 | - | - | - |
| 10 | 0.6 | 0.9 | 1.1 | 0.75 | 0.8 | 0.90 |
| 13 | 1.15 | 1.20 | 1.30 | 0.7 | 0.8 | 1.0 |
| 08 | 1.25 | 1.4 | 1.55 | 0.7 | 0.75 | 0.8 |
| 22 | 0.95 | 1.05 | 1.15 | 0.85 | 0.9 | 1.0 |

Table 2 Minimum, mean and maximum normal velocities in m/sec. H = 0.175 m, T = 1.8 sec.

| Measure-ment point | $v_{up}$ | | | $v_{down}$ | | |
|---|---|---|---|---|---|---|
| | min | mean | max | min | mean | max |
| 09 | 0.6 | 0.65 | 0.7 | 0.20 | 0.30 | 0.4 |
| 10 | 0.40 | 0.45 | 0.50 | 0.25 | 0.30 | 0.35 |
| 13 | 0.20 | 0.22 | 0.25 | 0.20 | 0.23 | 0.25 |
| 08 | 0.13 | 0.20 | 0.31 | 0.21 | 0.25 | 0.41 |
| 22 | 0.4 | 0.45 | 0.5 | 0.12 | 0.18 | 0.25 |

# 4. COMPARISON OF MEASUREMENTS WITH THE NUMERICAL MODEL ODIFLOCS

## 4.1 Description of the numerical model ODIFLOCS

The model ODIFLOCS (One Dimensional Flow on and in Coastal Structures) describes the wave motion on and in several types of structures. The model takes various phenomena into account. For instance reflection, permeability, infiltration, desorption, overtopping, varying roughness along the slope, linear and non-linear porous friction (Darcy- and turbulent friction), added mass, internal set-up and the disconnection of the free surface and the phreatic surface are all implemented. The model couples a hydraulic model to a porous flow model. Kobayashi et al. (1987 and 1989) proved that long wave equations can be used for the description of the external flow. The way in which the wave front is treated is also done in a similar way as by Kobayashi et al. (1987 and 1989). In the model ODIFLOCS long wave equations are applied for the internal flow as well. Long wave equations use hydrostatic pressures and imply a simulation of a breaking wave like a bore. The external flow and the internal flow are computed in two layers, a hydraulic layer and a porous layer, that partially overlap. The flow between both layers is determined by the pressure gradients. This flow has a maximum caused by the equilibrium of the pressure gradient and the friction. The pressure gradient in the vertical direction is assumed not to be larger than one. For a detailed description of this aspect and the model in general, see Van Gent (1992).

## 4.2 Comparison of measurements with the numerical model ODIFLOCS

The model can deal with only one porous layer. For a berm breakwater with a core, the choice has to be made whether the breakwater will be modelled as a homogeneous structure or as a structure with an impermeable core. The permeability of the core was very much the same as the material of the berm itself. Therefore, modelling as a homogeneous structure has been applied. The friction factor, depending on the roughness of the surface and the flow characteristics, was derived by using the empirical formula for fully rough turbulent flow on a uniform sloping breakwater by Madsen and White (1975):

$$f_w = 0.29 \left(\frac{d}{d_s}\right)^{-0.5} \left(\frac{d}{R \cot\alpha}\right)^{0.7}$$

The depth in front of the structure $d_s$ was 0.79 m; for the size of the armour unit, d, the $D_{n50}$=0.034 m was taken; the run-up R is about equal to the wave height for which 0.175 was used and for the angle of the slope, the angle from the berm section was taken ($\cot\alpha=5$). This gives a friction factor $f_w=0.15$. For the porosity, 0.35 was used. For the simulation added mass was not included. It might be inappropriate to compare calculated depth-averaged velocities with measured velocities in one point. However, an approximation of the maximum boundary layer thickness gives 0.01-0.015 m. This is rather low compared to the local water depth. Measured velocities in points above the boundary layer are assumed to be representative for the depth-averaged velocities. Measured velocities in different points above the slope, but in the same cross section, show differences in the order of magnitude of 20%.

For comparisons, two measuring points have been selected. The velocities measured in point 8 and 10, both above the berm and about 0.1 m away from each other, were used. Point 8 was positioned very close to the bottom and point 10 was about 0.07 m above the slope. Wave heights were measured above the berm and a comparison of those wave heights has been made as well, although the measured wave heights may be inaccurate as explained before. The simulated wave conditions were the nine combinations of wave heights of about 0.10, 0.15 and 0.20 m and wave periods of 1.5, 1.8 and 2.1 s. The combination H=0.175 m and T=1.8 s was added. Measuring point 8 is about at the level of the boundary thickness for these wave conditions. Point 10 is assumed to be above the boundary layer.

The calculated velocities are the horizontal velocities while the given measured velocities are the velocities along the slope. In principle the given measured velocities should be slightly larger than the calculated velocities.

The calculated velocities are the depth averaged velocities while the given measured velocities are velocities in a point. It is though believed that the measurement points are outside the boundary layer, except point 8.

The results of the comparisons of measured surface elevations with output from the numerical model, are summarized in Table 3.

The differences are rather low. A comparison between the maximum and minimum surface elevation is made to exclude the influence of a slightly different water level. The numerical model underestimates the fluctuation of the surface elevation with an average of 12.6 % difference (about 0.02 m) with the measured elevations. The wave condition T=1.5 s and H=19.5 cm gives a difference (10.9%) in the same order of magnitude as the average difference (12.6%). Therefore this computation is supposed to give a representative impression of the differences, see Figure 8.

Table 3 Differences between measured and calculated surface elevations.

| Surface elevation (H in cm) | | Measured | | ODIFLOCS | | Difference (in %) |
|---|---|---|---|---|---|---|
| | | max | min | max | min | |
| T=1.5 | H=11.7 | 32.0 | 14.9 | 28.8 | 14.7 | 17.5 |
| | H=15.0 | 35.0 | 15.0 | 31.0 | 14.5 | 17.5 |
| | H=20.8 | 36.0 | 13.5 | 37.0 | 13.0 | -6.7 |
| T=1.8 | H= 9.7 | 25.9 | 17.5 | 26.1 | 17.5 | -2.4 |
| | H=14.0 | 30.8 | 16.8 | 29.2 | 15.8 | 4.3 |
| | H=19.8 | 34.5 | 14.5 | 33.0 | 14.0 | 5.0 |
| T=2.1 | H= 9.9 | 26.2 | 16.4 | 24.8 | 18.2 | 32.7 |
| | H=14.2 | 30.7 | 15.0 | 27.0 | 16.8 | 35.0 |
| | H=19.5 | 32.1 | 12.8 | 29.0 | 11.8 | 10.9 |
| | | | | | Average | 12.6 |

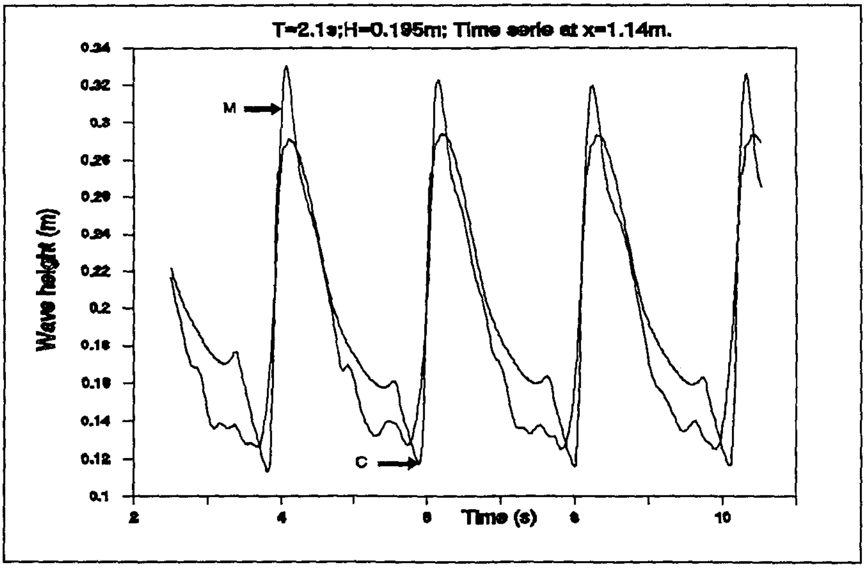


Figure 8 Comparison of the measured surface elevation with ODIFLOCS results.

The comparison of simulated depth-averaged velocities with the measured (point) velocities are summarized in Table 4 and Table 5. Two measurements in point 8 were not carried out. Differences for point 8 were to be expected because this point is so close to the bottom that the influence of the boundary layer is present here. However, an underestimation of the measured velocities with an average of 15.3% (maximum uprush velocity + maximum downrush velocity) is not so bad regarding the assumptions made for comparisons. The velocities in the direction of the breakwater (max) show an average underestimation of 18.4%. The velocities in the opposite direction give an average underestimation of 8.4%.

Table 4 Differences between measured and calculated velocities in point 8.

| VELOCITIES point 8 (H in cm) | | Measured | | ODIFLOCS | | Difference (in %) | | |
|---|---|---|---|---|---|---|---|---|
| | | max | min | max | min | mx-mn | max | min |
| T=1.5 | H=20.8 | 1.19 | -0.80 | 0.92 | -0.92 | 7.5 | 22.7 | -15.0 |
| T=1.8 | H= 9.7 | 0.75 | -0.44 | 0.52 | -0.35 | 26.9 | 30.7 | 20.5 |
| | H=14.0 | 1.12 | -0.63 | 0.70 | -0.60 | 25.7 | 37.5 | 4.8 |
| | H=19.8 | 1.31 | -0.82 | 0.90 | -0.88 | 16.4 | 31.3 | -7.3 |
| T=2.1 | H= 9.9 | 0.90 | -0.62 | 0.62 | -0.38 | 34.2 | 31.1 | 38.7 |
| | H=14.2 | 1.15 | -0.85 | 0.92 | -0.66 | 21.0 | 20.0 | 22.4 |
| | H=19.5 | 1.20 | -1.02 | 1.18 | -0.90 | 6.3 | 1.7 | 11.8 |
| | | | | | Average | 15.3 | 18.4 | 8.4 |

Table 5 Differences between measured and computed velocities for point 10.

| VELOCITIES point 10 (H in cm) | | Measured | | ODIFLOCS | | Difference (in %) | | |
|---|---|---|---|---|---|---|---|---|
| | | max | min | max | min | mx-mn | max | min |
| T=1.5 | H=11.7 | 0.38 | -0.43 | 0.42 | -0.40 | -1.2 | -10.5 | 7.0 |
| | H=15.0 | 0.55 | -0.55 | 0.52 | -0.60 | -1.8 | 5.5 | -9.1 |
| | H=20.8 | 0.90 | -0.83 | 0.73 | -0.88 | 6.9 | 18.9 | -6.0 |
| T=1.8 | H= 9.7 | 0.34 | -0.34 | 0.47 | -0.33 | -17.6 | -38.2 | 2.9 |
| | H=14.0 | 0.58 | -0.56 | 0.64 | -0.55 | -4.4 | -10.3 | 1.8 |
| | H=19.8 | 0.94 | -0.83 | 0.92 | -0.90 | -2.8 | 2.1 | -8.4 |
| T=2.1 | H= 9.9 | 0.52 | -0.54 | 0.57 | -0.38 | 10.4 | -9.6 | 29.6 |
| | H=14.2 | 1.05 | -0.90 | 0.77 | -0.60 | 29.7 | 26.7 | 33.3 |
| | H=19.5 | 1.42 | -1.20 | 1.05 | -0.90 | 25.6 | 26.1 | 25.0 |
| | | | | | Average | 5.0 | 1.2 | 8.5 |

Comparisons with data from measuring point 10 give better results than for point 8. As mentioned before, the relatively higher differences for point 8 are probably due to the overshoot effect in the boundary layer. The average underestimation is now 5% (average of max. uprush velocity + max. downrush velocity) of the measured velocity. The average difference independent of whether an underestimation or an overestimation is found, is higher than 5%. The average (absolute) deviation of the sum of the maximum velocities in both directions is 11.1% (max-min). The average deviation is 16.4% in the direction towards the breakwater and 13.7% in the direction away from the breakwater. Table 4 shows that the underestimation is relatively high for the combination with high wave heights and long wave periods. For these cases, the boundary layer may be relatively thick. Measuring point 10 may be influenced by the higher velocities of the boundary layer. In general, the predicted velocities show differences with the measured velocities in the same order of magnitude as the differences which appear between measured velocities at different positions above the slope in the same cross section. The results prove that the numerical model ODIFLOCS predicts velocities rather good although differences of about 35% occurred sometimes. Figure 9 and 10 show the results of two comparisons. Figure 11 shows differences for a wave height of 0.099 m and a period of 2.1 s in measuring point 10. This combination gives a difference with the measurement of 10.4% which gives a representative impression of the deviations.

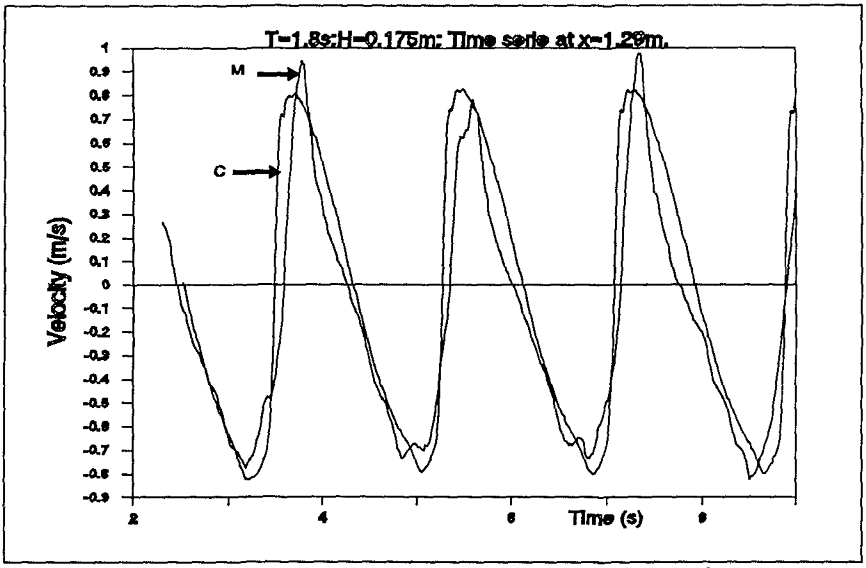


Figure 9 Comparison of the measured velocity with ODIFLOCS results - point 10.

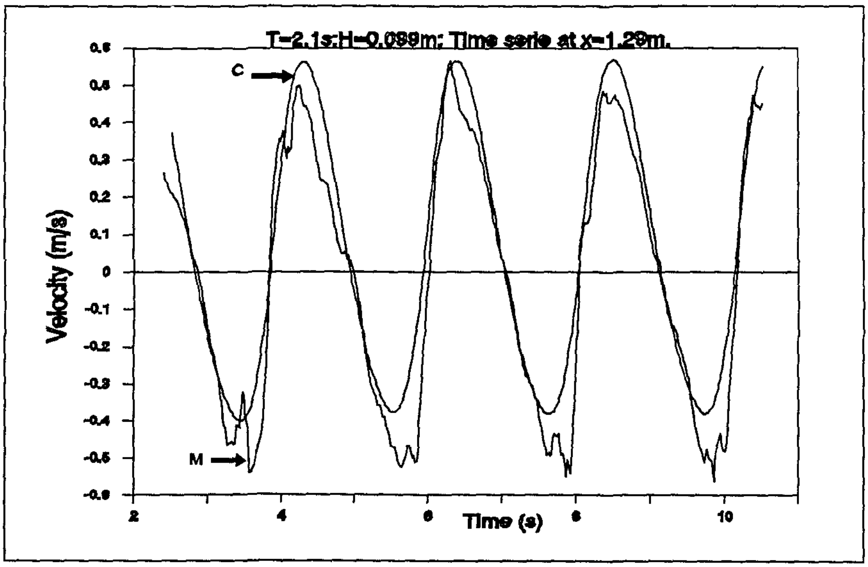


Figure 10 Comparison of the measured velocity with ODIFLOCS results - point 10.

## 4.3 Calculated extreme velocities

The comparison between the measured velocities and the calculated velocities give a fair agreement. Therefore, it is interesting to compute the maximum velocities appearing somewhere along the slope. The calculated maximum velocities show that the maximum upward velocities (U-max) are higher than the maximum downward velocities (U-min). These extreme velocities appeared to be just below the still water level. Only for the computations with the relatively high wave heights, the extreme velocities U-min were found further down the slope. For these three cases local maximums occurred just below still water level.

| MAXIMUM VELOCITIES | | | | | |
|---|---|---|---|---|---|
| T | H | Umax | Umin | x-Umax | x-Umin |
| 1.5 | 0.117 | 1.33 | -0.52 | -0.03 | -0.03 |
| | 0.150 | 1.61 | -0.69 | -0.03 | 0.00 |
| | 0.208 | 2.01 | -0.92 | -0.03 | -0.48 |
| 1.8 | 0.097 | 0.94 | -0.50 | -0.03 | -0.09 |
| | 0.140 | 1.55 | -0.71 | -0.09 | -0.06 |
| | 0.198 | 2.02 | -0.93 | -0.06 | -0.69 |
| 2.1 | 0.099 | 0.87 | -0.49 | 0.03 | -0.12 |
| | 0.142 | 1.53 | -0.70 | -0.12 | -0.09 |
| | 0.195 | 2.02 | -0.95 | -0.12 | -0.81 |

Table 6 Maximum velocities with the positions along the slope.

## 5 CONCLUSIONS

Although the measurements were carried out for "regular" waves, there were variations in the heights of consecutive waves and also in the maximum velocities for each wave.

On this background the conclusion from the few comparisons we have made between the ODIFLOCS calculations and the measurements are that there is a fair agreement between the measurements and the calculations except in measurement point 08. The velocities in measurement point 08 are as previously remarked possibly influenced by proximity to the stone cover layer.

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