COMMISSION
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FP5- EESD

CREST LEVEL ASSESSMENT OF
COASTAL STRUCTURES BY
FULL-SCALE MONITORING,
NEURAL NETWORK PREDICTION
AND HAZARD ANALYSIS
ON PERMISSIBLE WAVE OVERTOPPING

CLASH

EVK3-CT-2001-00058

D24 Report on additional tests

Part D

REPORT

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Contents

1 Introduction

The international CLASH project of the European Union (Crest Level Assessment of coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping, www.clash-eu.org) under contract no. EVK3-CT-2001-00058 is focussing on wave overtopping for different structures in prototype and in laboratory (see De Rouck et al., 2002). The main scientific objectives of CLASH are (i) to solve the problem of possible scale effects for wave overtopping and (ii) to produce a generic prediction method for crest height design or overtopping assessment.

Based on a database of laboratory data on wave overtopping (≈ 10,000 tests) a neural network technique is used for producing a generic prediction method. In the first database (≈ 7,000 tests) the following white spots were detected where additional tests could improve the generic prediction method:

- Influence of surface roughness/permeability
- Effect of obliqueness, short-crested waves and directional spreading
- Influence of roughness around still water level
- Low steepness ($s_{0p} < 0.01$)
- Influence of G_c and A_c
- Angle of berm
- Toe details

The first two were considered as the most important ones. Besides from those two, tests have been performed with low steepness and reshaping breakwaters at Ghent University and Aalborg University respectively. The reshaping breakwater is a type of breakwater where the information on overtopping is very limited. The additional tests are described in the following four reports:

- D24 – Part A: Effect of obliqueness, short-crested waves and directional spreading
- D24 – Part B: Influence of surface roughness/permeability
- D24 – Part C: Low steepness tests
- D24 – Part D: Reshaping breakwater tests

This report deals with the laboratory tests performed to give additional information on overtopping of reshaping breakwaters performed at Aalborg University, 2003-2004.

2 Experimental Setup

Overtopping and front and rear slope stability is studied on reshaping berm breakwaters with a homogenous berm. 82 tests has been performed to describe the influence of sea state, crest freeboard and crest width.

Only tests with head-on waves were performed, although pure two-dimensional head-on waves almost never occur in nature. Oblique waves may cause longshore transport which can be an important factor for the stability of the breakwater, but normally overtopping is larger for head-on waves than for oblique waves. The model tests were performed in the deep wave flume at Aalborg University. The flume has the dimensions 21.5 × 1.2 ×1.5 m (length × width × depth). In a short part at both ends of the flume the bottom was flat but in the main part the bottom had a slope of 1:20. Thus the water depth was 0.60 meter greater at the wavemaker than at the toe of the structure. The waves were measured both at deep water and at the toe of structure with resistance type wave gauges. The waves were measured with an array of three wave gauges so the waves could be separated into incident and reflected waves.

Overtopping was measured with a water surface amplitude gauge in a tank. The overtopping water was measured at the back of the crest and led to the tank via a ramp as shown in Fig. 1. The ramp and the tank was approximately 0.30 meter wide and placed in the middle of the flume. A water surface elevation gauge was installed behind the breakwater to measure wave set-up.

The rear side erosion was observed at each side of the ramp as the ramp prevented erosion of the middle of the structure.

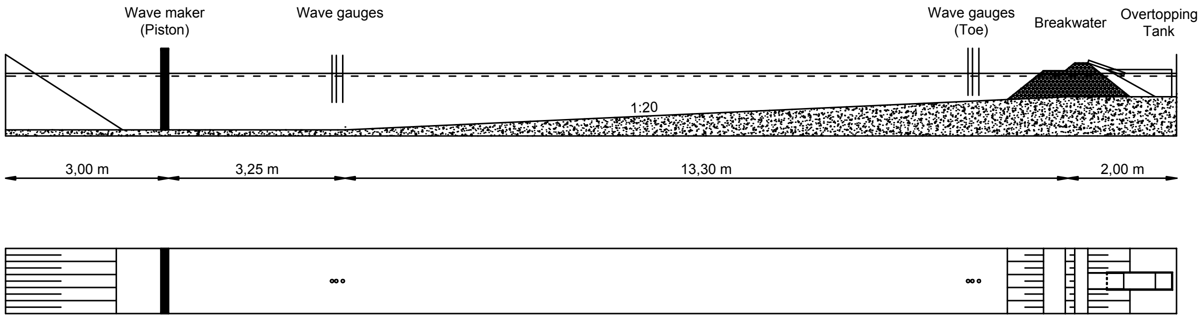

Figure 1: Layout in flume (side and top view).

A reshaping breakwater is usually constructed with a steeper slope than a conventional rubble mound breakwater. Due to the construction method the front and rear slope of a reshaping breakwater are around the natural angle of response of the material. A slope of 1:1 to 1:1.5 is typical for a berm breakwater, where a conventional breakwater is typically constructed with a front slope of 1:2. In the present experiments, the slopes were 1:1.25 in all tests which was close to the natural angle of repose for the materials used.

According to Alikhani (2000) the berm is, due to the construction method, typically placed 0.5 to 1 meter above design high water level. In the present tests the berm was initially located 0.04m above SWL.

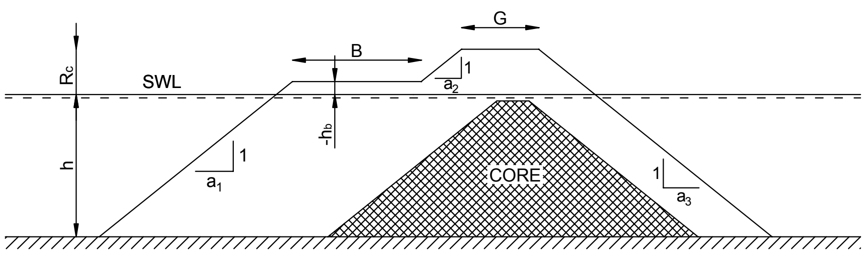

Figure 2: Initial geometry of breakwater.

The core was not extended into the berm and the size of the core was kept constant in all experiments. The influence of the core configuration on the results is assumed to be small [Lissev and Tørum (1996)]. The properties of the materials used for the breakwater are given in the following Table 1:

	Armour	Core
W_{50} [kg]	0.0202	0.0069
Density ρ [kg/m^3]	2610	2700
$D_{n,50}$ [m]	0.0198	0.0137
$f_g = D_{n,85}/D_{n,15}$	1.45	1.59
Length ratio, l/b	2.01	2.04

Table 1: Material properties.

3 Data Analysis

All signals were filtered using an analog lowpassfilter with a cut-off frequency of 8Hz. A digital filter with cut-off frequencies of $1/3 \cdot f_p$ and $3 \cdot f_p$ was applied to the wave signals. The Mansard & Funke method was applied to calculate the incident wave spectrum and the SIRW method of Frigaard & Brorsen (1995) was used to calculate the incident wave trains. All signal analysis was performed with the WaveLab software package [http://hydrosoft.civil.auc.dk/wavelab].

4 Test Series – Range of Parameters

When measuring overtopping on a reshaping breakwater it is essential to distinguish between two situations which can be equally important:

1. The design situation has not yet taken place and only some small waves have reshaped the breakwater.
2. The design situation has already taken place and only some minor stone movements occur for other sea states.

In the present study was studied situation 1 as overtopping was measured when the profile had stabilized in a static or dynamically stable profile corresponding to the applied sea state in the specific test.

All tests were carried out with irregular waves generated from a JONSWAP spectrum with a peak enhancement factor (γ) of 3.3 using a white noise filtering method. The tests were run with increasing peak period (T_P) and wave height (H_s) so that the wave steepness (s_{0p}) was kept constant. Therefore, the breakwater was only rebuilt when changing the geometry or the wave steepness, which will say approximately 20 times. Each test consisted of approximately 1500 waves and was run twice. The first run was only used to make the breakwater profile statically or dynamically stable for the waves. In the second run, the measurements were carried out. The main part of the tests were performed with a wave steepness of approximately 0.04, but also tests were performed to cover the range from 0.017 to 0.054 in peak wave steepness as illustrated in Figure 3. Four different crest freeboards and four different crest widths were tested, i.e. 16 combinations. For one of these geometries, the influence of the wave steepness was tested. After each test the reshaped profile was measured through the glass wall of the flume.

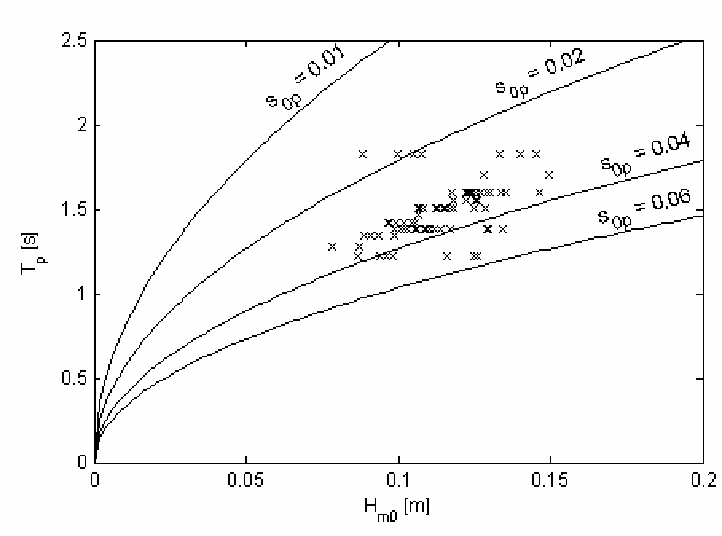

Figure 3: Tested sea states.

The following range of parameters is covered in the tests:

- Peak wave steepness (s_{0p}): 0.17 – 0.54
- Wave height at toe of structure (H_{m0}): 0.078m – 0.149m
- Water depth at toe of breakwater (h): 0.44m
- Crest freeboard (R_c): 0.08m, 0.11m, 0.14m and 0.17m
- Crest width (G_c): 0.17m, 0.24m, 0.31m and 0.38m
- Berm width (B): 0.40m
- Berm elevation (h_b): -0.04m
- Front slope below berm (a_1): 1:1.25
- Front slope above berm (a_2): 1:1.25
- Rear slope (a_3): 1:1.25
- Bottom slope: 1:20
- Stability number (N_S): 2.4 – 4.7
- Reynolds number for armour stones (Re): $1.74 \cdot 10^4$ - $2.40 \cdot 10^4$

5 Front Slope Stability

A typical reshaping breakwater profile, before and after reshaping has taken place, is shown in Fig. 4. Just below water level, the reshaped profile typically has a slope of about 1:4. In front of this slope, stones are deposited with a steeper slope approaching the natural angle of repose. Wave energy is dissipated by wave breaking over the berm and by porous flow in the mound. The flat slope around the water level and a highly energy absorbing porous media give small value of reflection from a berm breakwater, thus better maneuvering conditions in front of the entrance. Further run-up and overtopping are generally smaller than for a conventional straight and steeper breakwater slope.

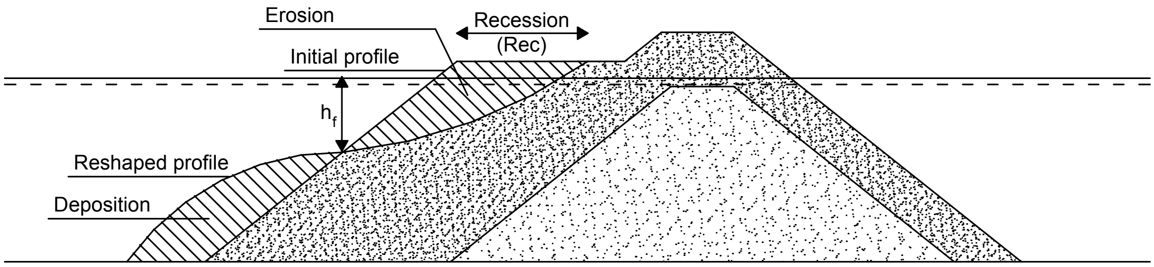

Figure 4: Typical initial and reshaped profile.

The breakwater can be characterized by the stability parameter N_s.

$$N_s = \frac{H_s}{\Delta \cdot D_{n,50}} \tag{1}$$

However, N_s do not include the influence of the wave period, and a modified stability number which also includes the effect of the wave period as well can be used instead, e.g.:

$$H_0T_0 = \frac{H_s}{\Delta \cdot D_{n,50}} \cdot \sqrt{\frac{g}{D_{n,50}}} \cdot T_Z \tag{2}$$

Table 2 shows the mobility criterion for the three types of berm breakwaters.

Type of breakwater	Stability index N_s	Stability index H_0T_0
Little movement, statically stable non-reshaped berm breakwater	N_s < 1.5-2	H_0T_0 < 20-40
Limited movement during reshaping, statically stable reshaped berm breakwater	1.5< N_s < 2.7	40 < H_0T_0 < 70
Relevant movement, dynamically stable reshaped berm breakwater	N_s > 2.7	H_0T_0 > 70

Table 2: Mobility criterion (the criterion depends on stone gradation) [PIANC (2003)].

The reshaped profile can be calculated by the equations/routines developed by Van der Meer (1992), Van Gent (1995) and Archetti (1996). A comparison between meassured and calculated profiles by the method of Van der Meer (1992) is available in Appendix A. The most important measure for the reshaping is the recession of the berm (Rec). A comparison of the measured and calculated recession is given in Figure 5 for the Van der Meer (1992) method.

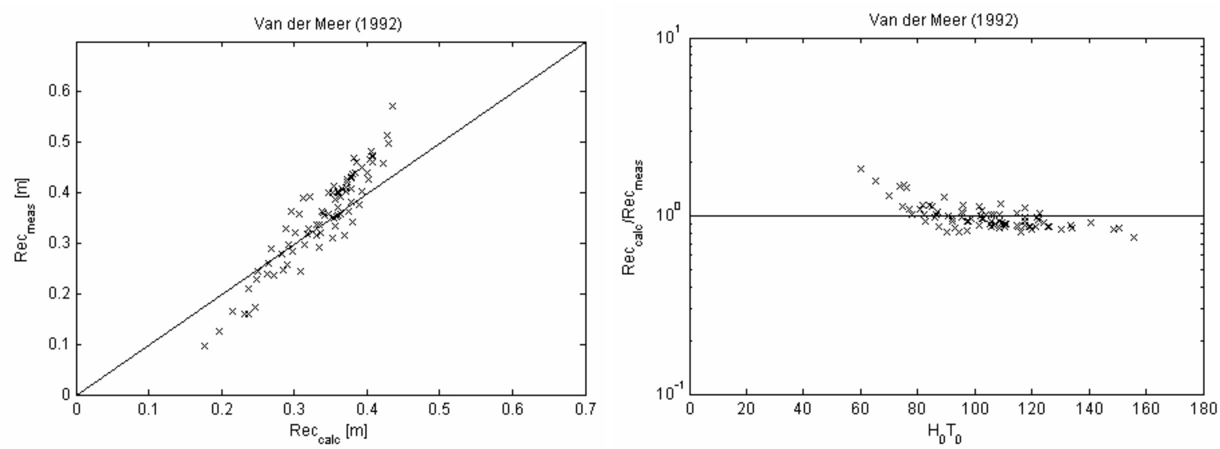

Figure 5: Comparison of meassured and calculated recession by the method of Van der Meer (1992).

More simple formulas to calculate the recession of the berm has be presented by Hall & Kao (1991) and Tørum & Krogh (2000). The formula of Hall & Kao (1991) and Tørum & Krogh (2000) under predicts the amount of recession for the present data. The formulae of Hall & Kao (1991) and Tørum & Krogh (2000) do not include the influence of berm elevation and front slope which are two very important parameters for the recession. The reshaped profile will approximately be independent on the front slope, but much more material has to be moved for the steep slope which therefore leads to much more recession.

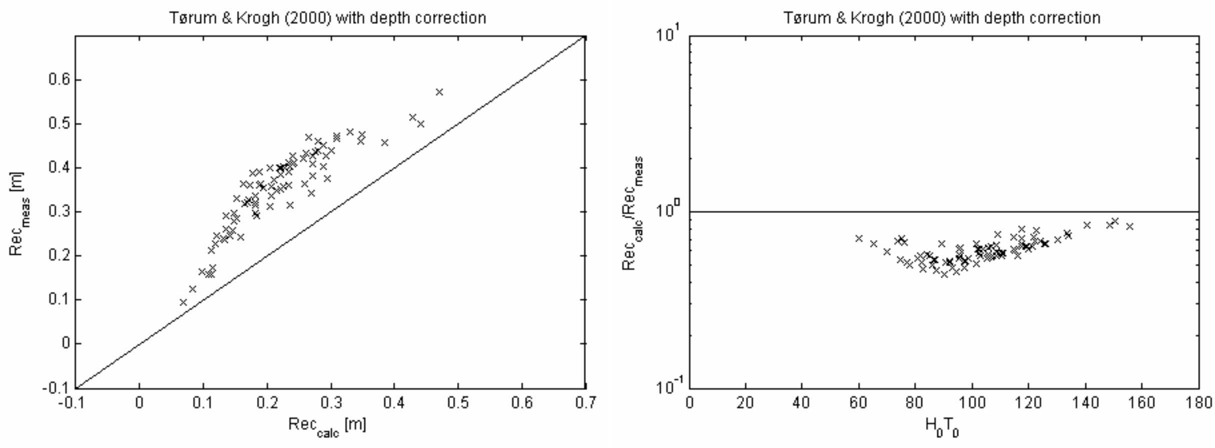

Figure 6: Comparison of meassured and calculated recession by the method of Tørum & Krogh (2000).

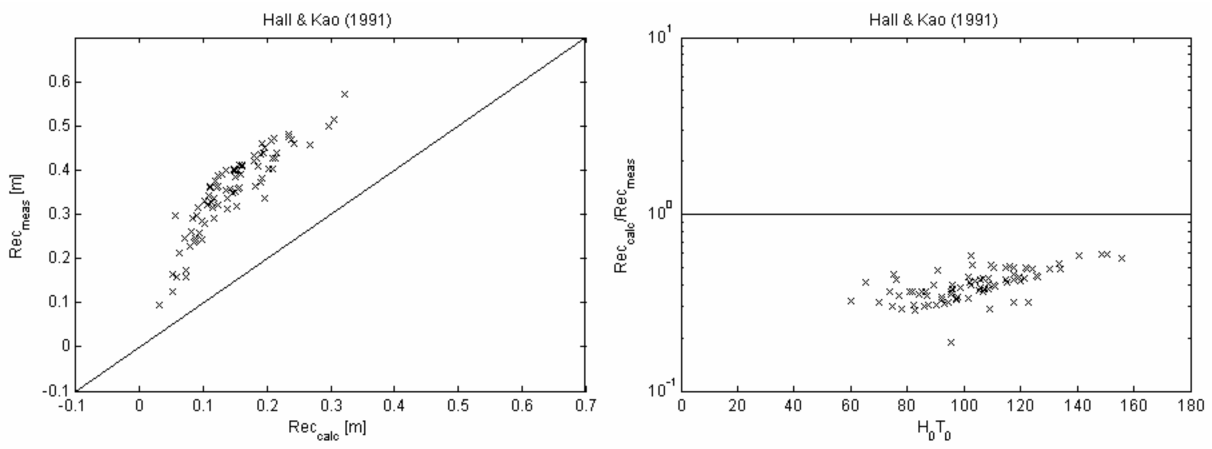

Figure 7: Comparison of meassured and calculated recession by the method of Hall & Kao (1991).

Also other researchers' data confirms that the formulae of Hall & Kao (1991) and Tørum & Krogh (2000) are not generally valid. The method of Van der Meer (1992) overcomes all these problems and is very good for $H_0T_0 > 70$, corresponding to dynamically stable profiles. The method has however the following limitations:

- The method assumes conservation of the total volume of the breakwater. However it has been observed in the tests that some compaction can occur.
- The method predicts too much damage for statically stable reshaped and non-reshaped breakwaters ($H_0T_0<70$).
- In principle the method of Van der Meer (1992) could use the reshaped profile as input and then continuing from this profile with another seastate. However it was observed that this resulted in less damage than always starting with the initial profile and worse agreement

with measurements. Unless changing water level it is therefore suggested always to start with the initial profile in the calculations.

6 Overtopping

Overtopping measurements on berm breakwaters have been made by Lissev (1993), Lissev and Tørum (1996) and Kuhnen (2000). Lissev and Tørum (1996) measured irregular wave overtopping on berm breakwaters for two different core configurations and different sea states. However, only one value of the crest freeboard and the crest width were tested. Lissev and Tørum (1996) concluded that the core could be extended into the berm without significant influence on the reshaped profile or on overtopping. Lissev found a formula that could describe the overtopping discharges in his experiments. Lissevs formula is not dimensionless and therefore, some authors have tried to rewrite Lissevs formula to make it dimensionless. In the authors' opinion, this is a waste of energy because Lissev only tested one cross section which is far from being sufficient for making a general valid overtopping formula.

Due to characteristics of the reshaped profile, with a sloping berm no existing simple overtopping formulae cover reshaping berm breakwaters. Based on approximately 700 tests with berm breakwaters (including the present tests) an overtopping formula for berm breakwaters was presented by Lykke Andersen and Burcharth (2004).

In the CLASH project a neural network prediction method was used to develop a prediction method for estimating overtopping discharge. The neural network was based on a large database containing more than 10,000 tests with a lot of different kind of structures. The present 82 tests were included in the neural network analysis.

The formulae of Van der Meer and Janssen (1994) should cover the non-reshaping berm breakwaters, eventually with the correction factor of Besley (1999). Due to the flat slope around SWL the waves are breaking on the structure (ξ<2).

Just to give a rough view of the data they are plotted in the Van der Meer & Janssen (1994) plot for non-breaking waves when using no reduction due to the berm. As expected the data shows a lot of scatter in this plot. Inclusion of the reduction factor by Besley (1999) to take into account the permeable berm does not lead to less scatter.

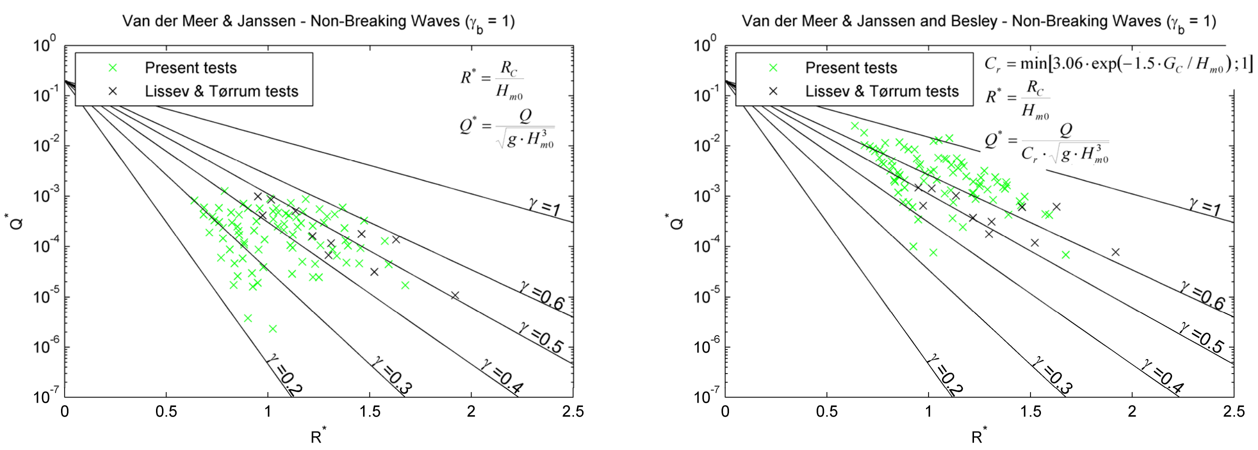

Figure 8: Overview of overtopping data in the Van der Meer plot for non-breaking waves.

7 Rear Slope Stability

After each test, the rear side damage was estimated by visual observations. The 4 categories used by Van der Meer and Veldman (1992) were also used in the present study, but in the present study a step between each category were used as well. The rear slope damage typically starts with a few stones at the rear side just above still water level being displaced downwards during wave overtopping. This was also observed by Andersen et al. (1992).

The rear side damage is plotted against the mean overtopping discharge in Fig. 9. The figure shows some correlation between rear slope stability and mean overtopping discharge and stone size as expected but some scatter is present.

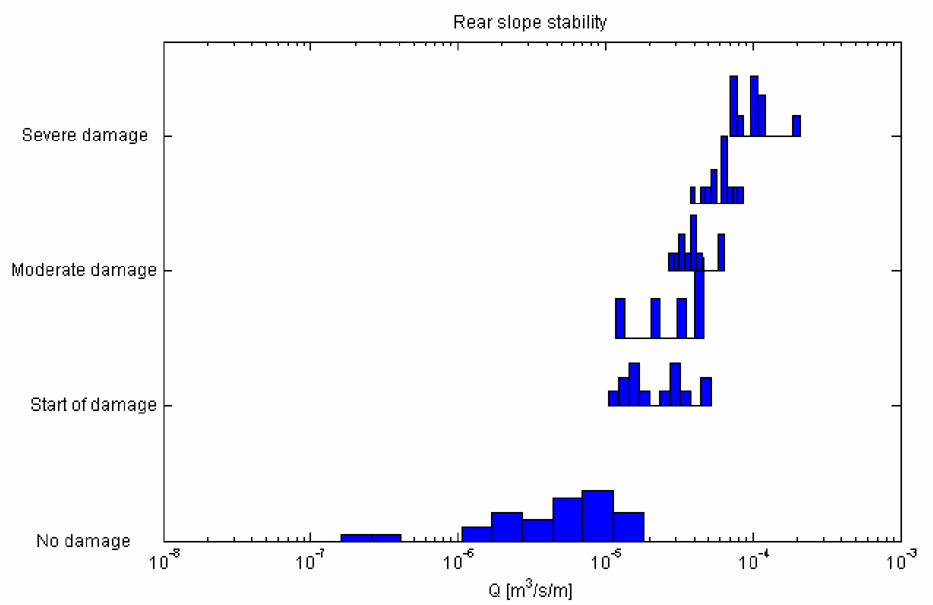

Figure 9: Rear side stability against measured mean overtopping.

8 Conclusions

82 tests have been performed to give more information on overtopping of reshaping breakwaters. The conclusions can be summarized as follows:

- Very good agreement between measured and calculated profiles by the method of Van der Meer (1992) is observed in all cases with dynamically stable profiles. For statically stable reshaping breakwaters the method over predicts the amount of damage.

- Based on overtopping measurements from approximately 700 tests including the present tests, an overtopping formula was derived by Lykke Andersen and Burcharth (2004).

- A fair agreement between mean overtopping discharge and rear slope erosion was observed.

9 References

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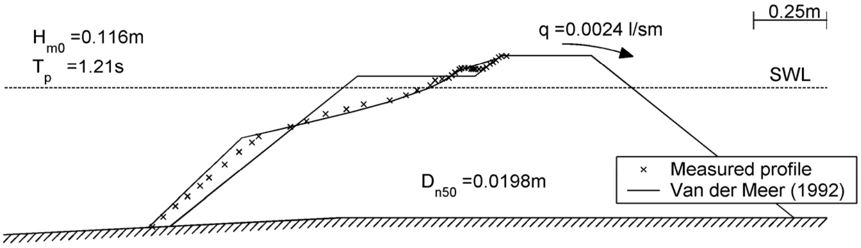
Appendix A: Test Results

In this appendix is given an overview of the tests performed and the results. The meassured profiles is compared to the calculated by the formulas given by Van der Meer (1992).

Test No.1



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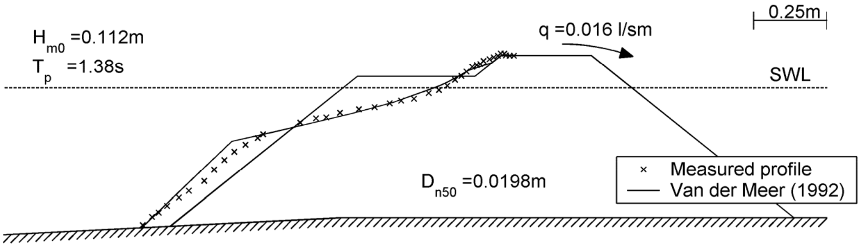

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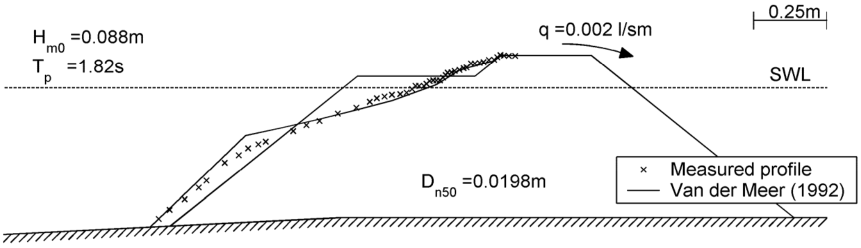

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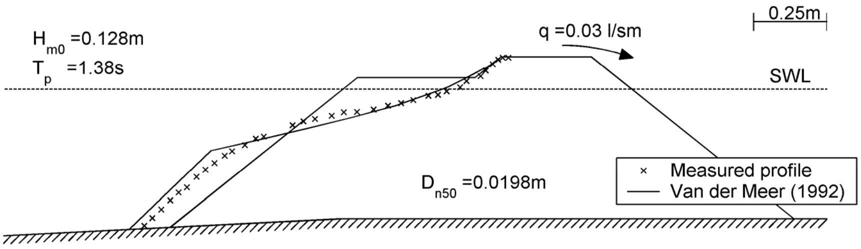
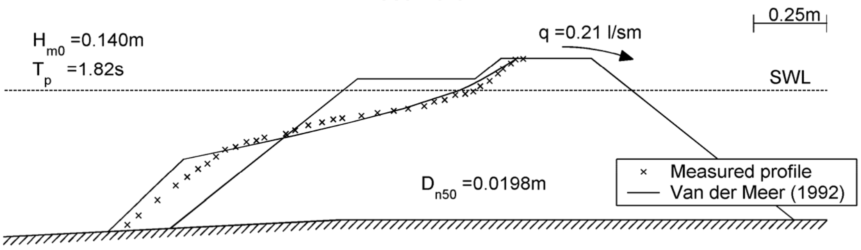
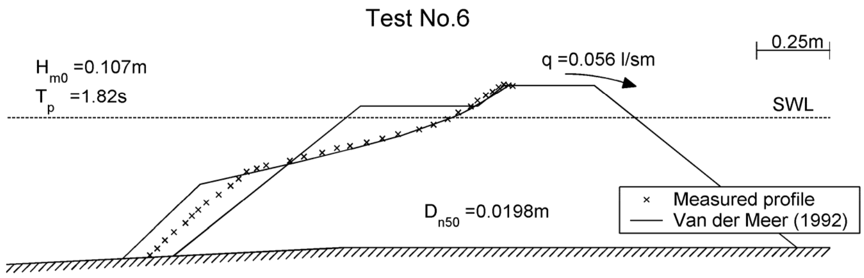
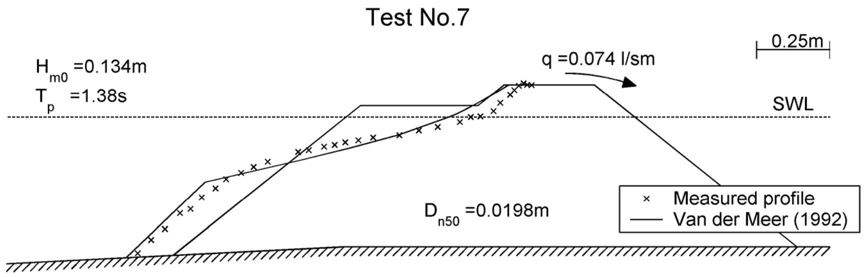


Test No.5




Test No.6
H m0 =0.107m
T p =1.82s
q =0.056 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.7
H m0 =0.134m
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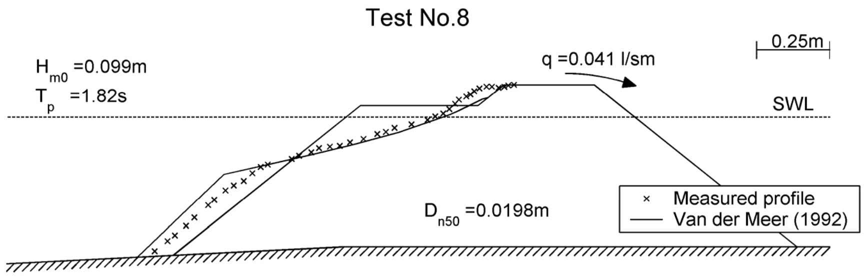
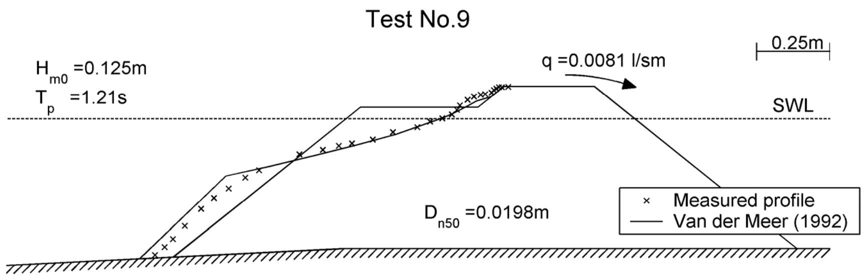
q =0.074 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.8
H m0 =0.099m
T p =1.82s
q =0.041 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.9
H m0 =0.125m
T p =1.21s
q =0.0081 l/sm
0.25m
SWL
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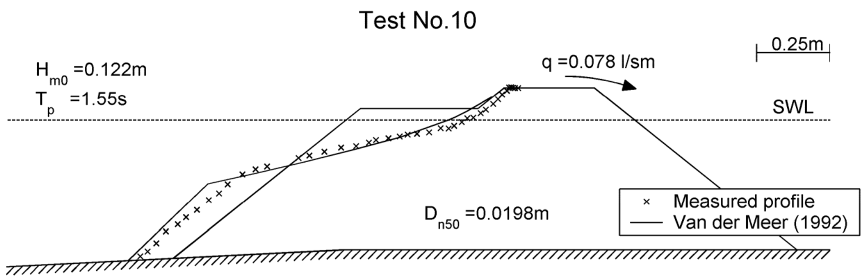
Measured profile
Van der Meer (1992)


Test No.10
H m0 =0.122m
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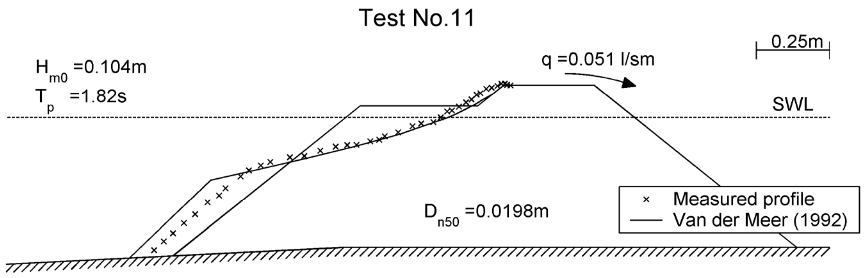
q =0.078 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)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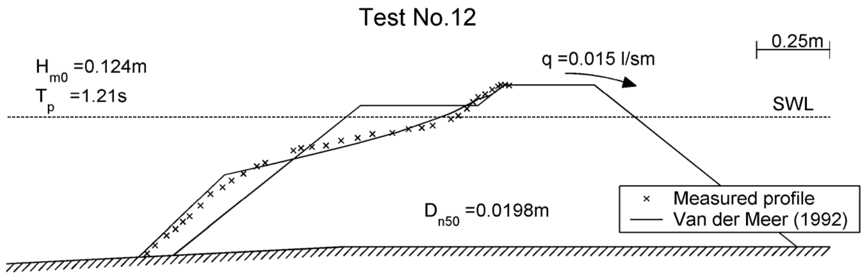
SWL
D_n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.12
H_m0 =0.124m
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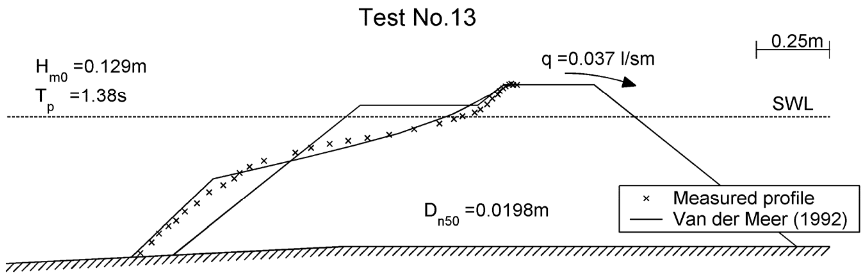
0.25m
SWL
D_n50 =0.0198m
Measured profile
Van der Meer (1992)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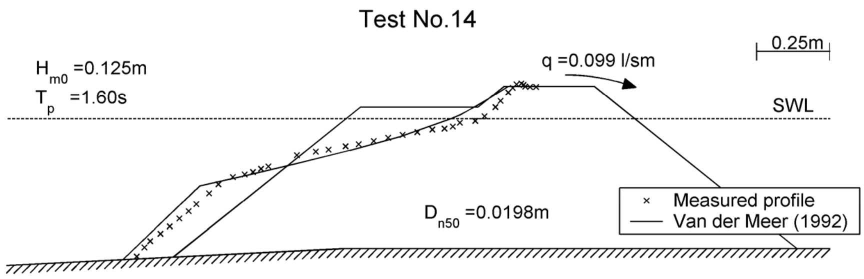
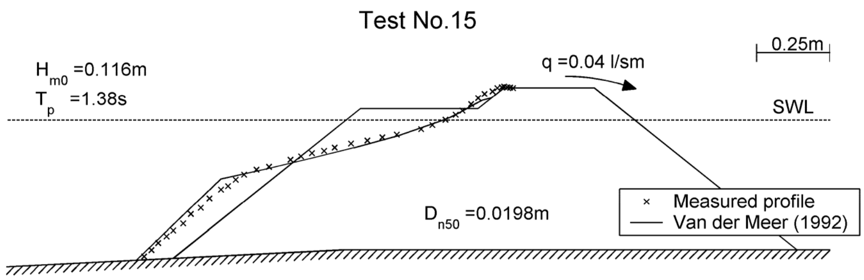
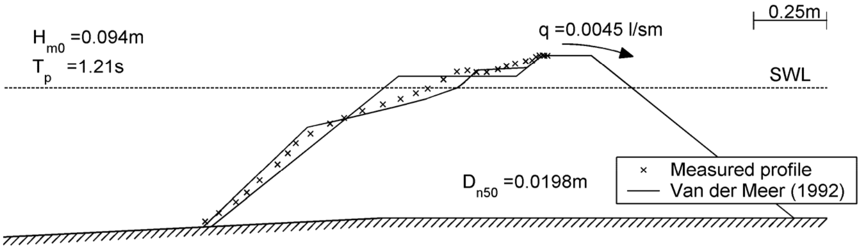
0.25m
SWL
D_n50 =0.0198m
Measured profile
Van der Meer (1992)

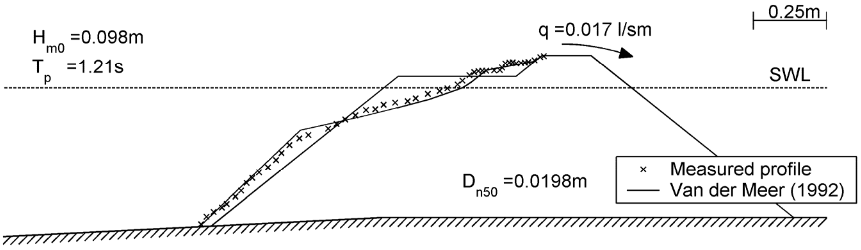

Test No.14
H_m0 =0.125m
T_p =1.60s
q =0.099 l/sm
0.25m
SWL
D_n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.15
H_m0 =0.116m
T_p =1.38s
q =0.04 l/sm
0.25m
SWL
D_n50 =0.0198m
Measured profile
Van der Meer (1992)

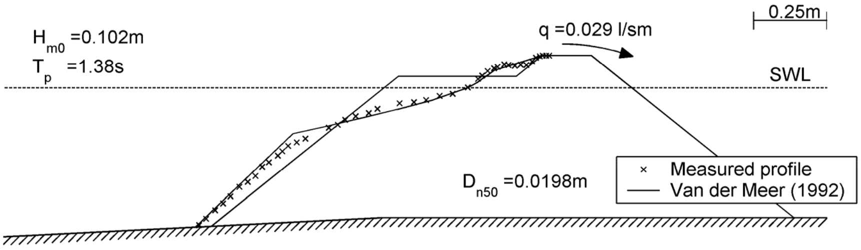
Test No.16



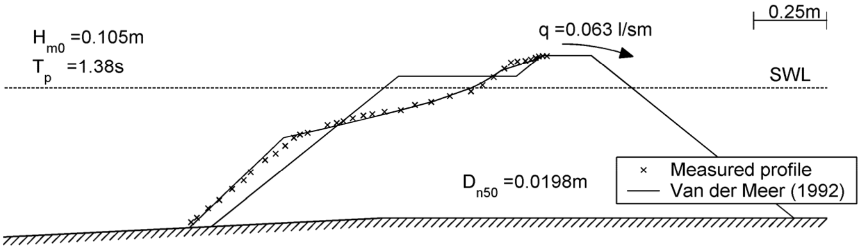
Test No.17



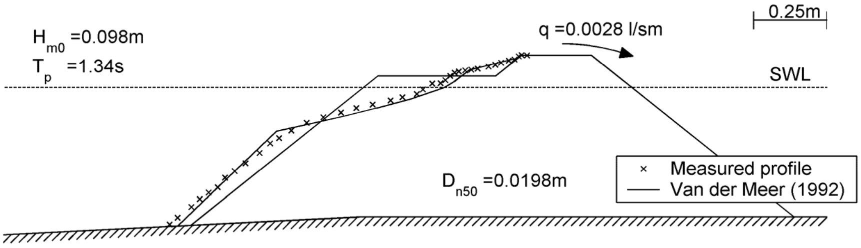
Test No.18



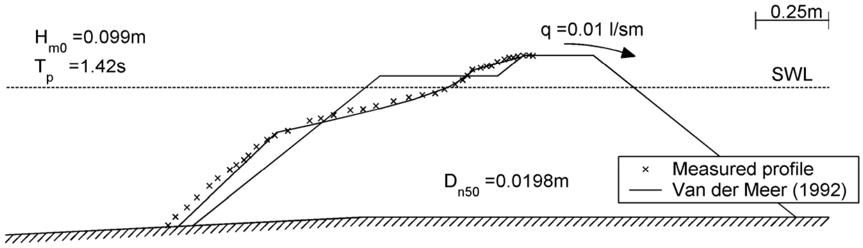
Test No.19



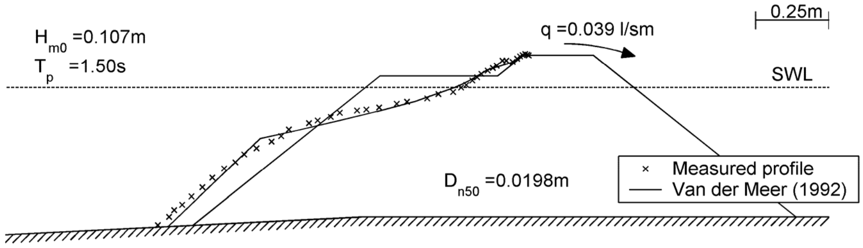
Test No.20



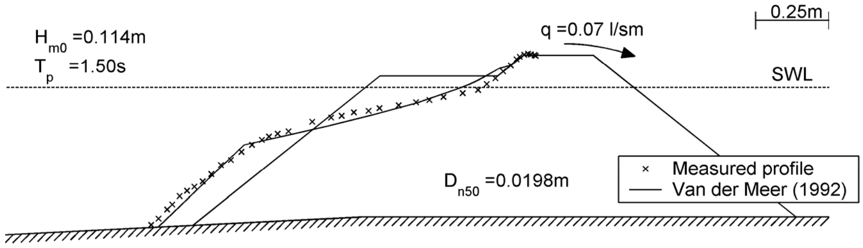
Test No.21



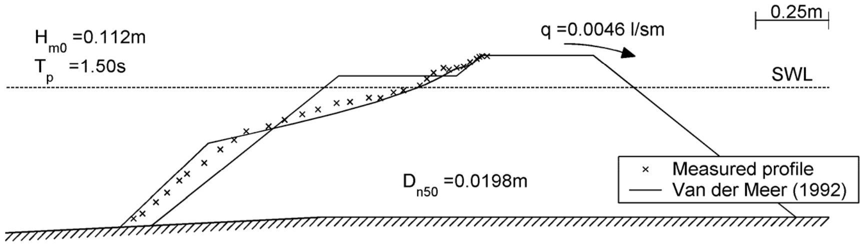
Test No.22



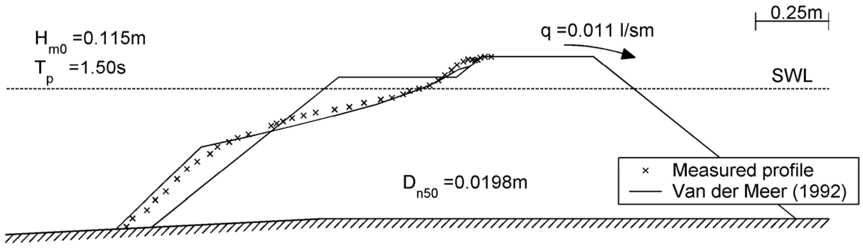
Test No.23



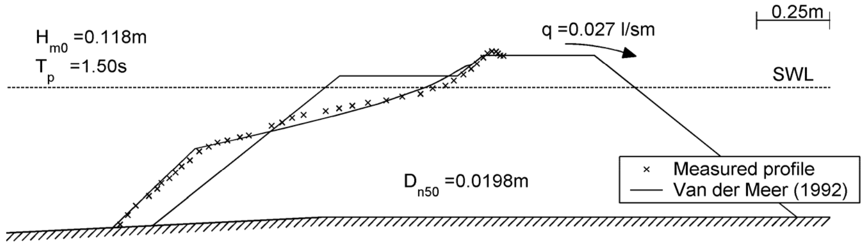
Test No.24



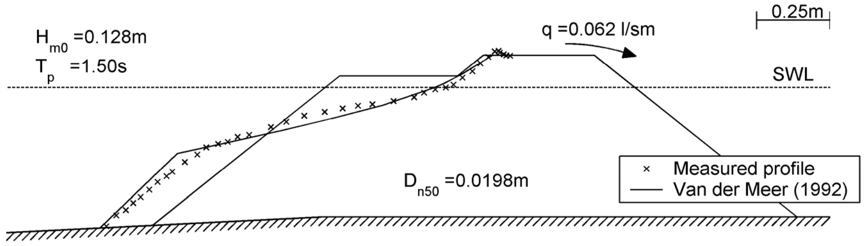
Test No.25



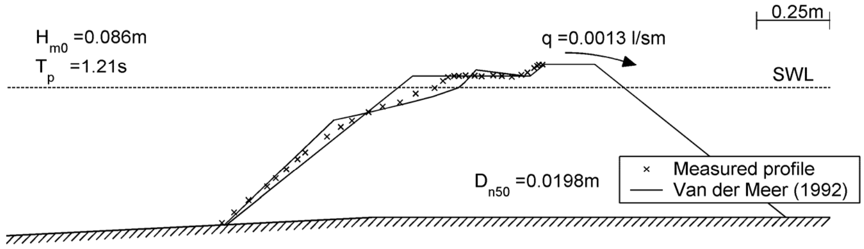
Test No.26



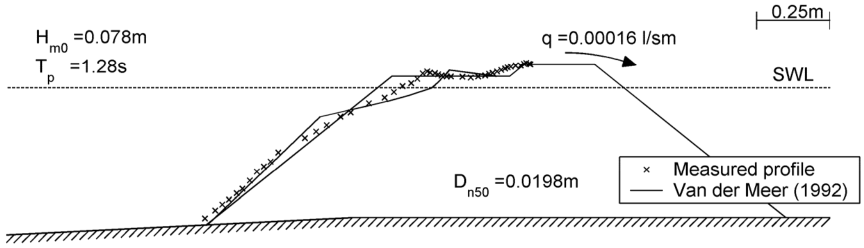
Test No.27



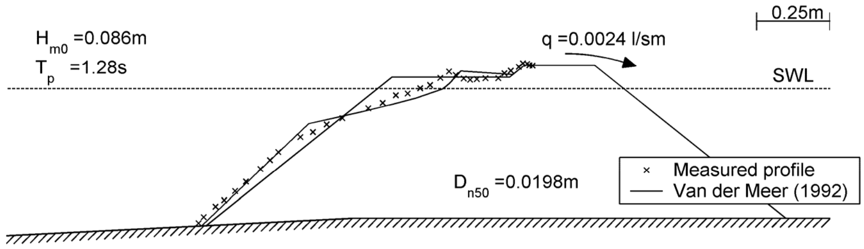
Test No.28



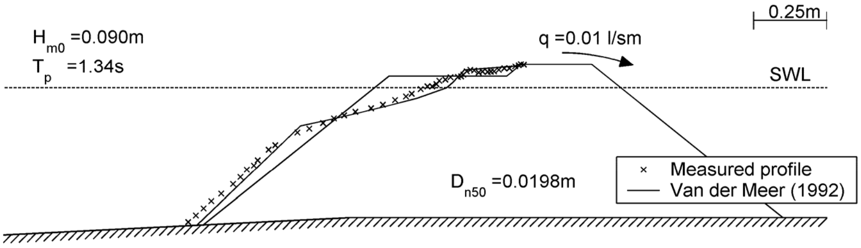
Test No.29



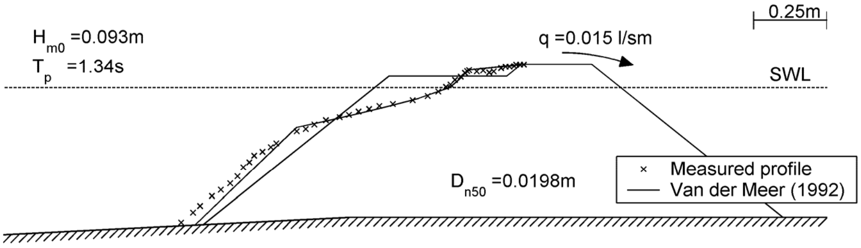
Test No.30



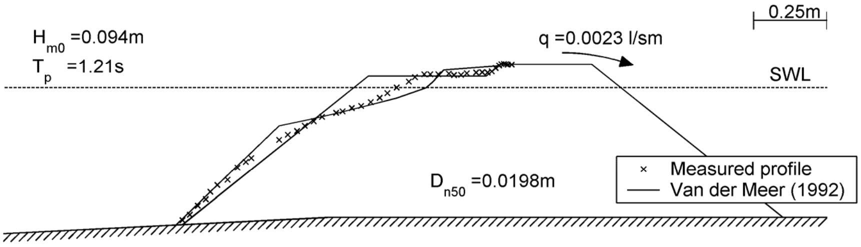
Test No.31



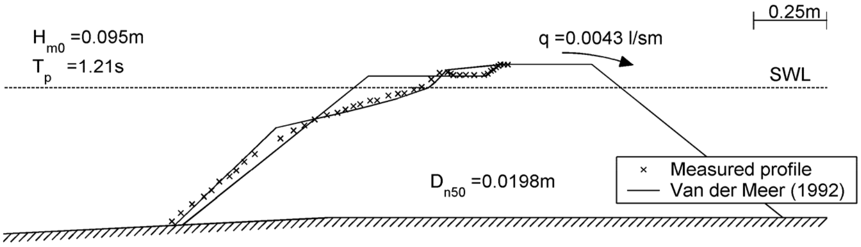
Test No.32



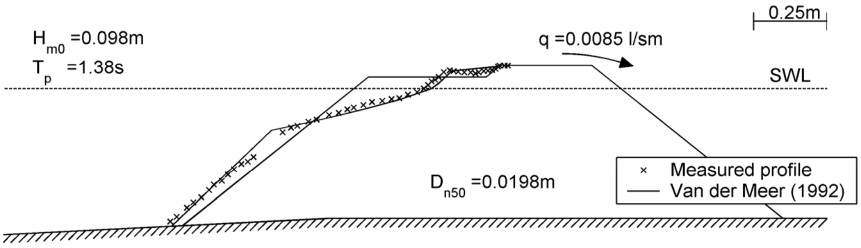
Test No.33

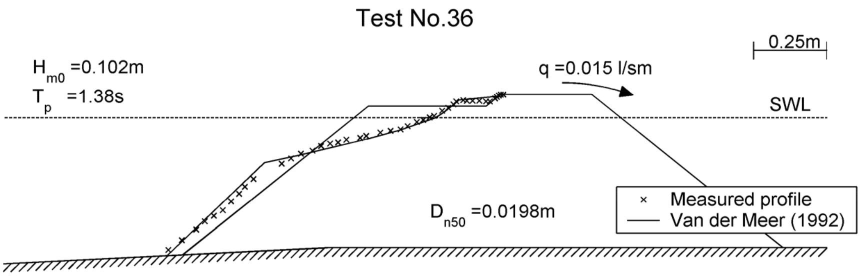


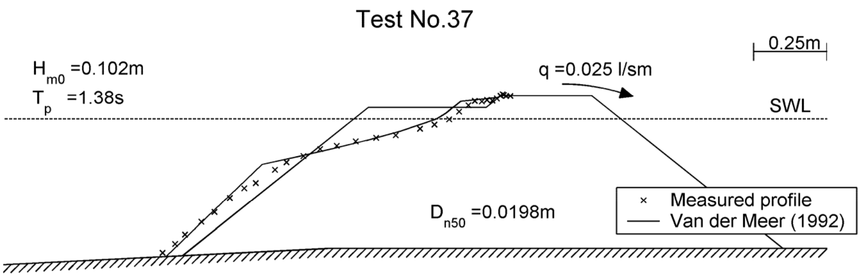
Test No.34

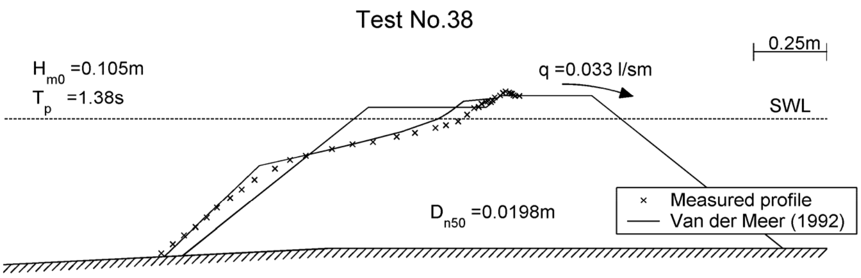


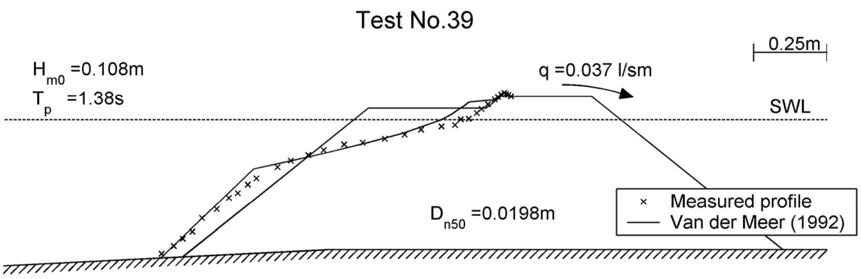
Test No.35

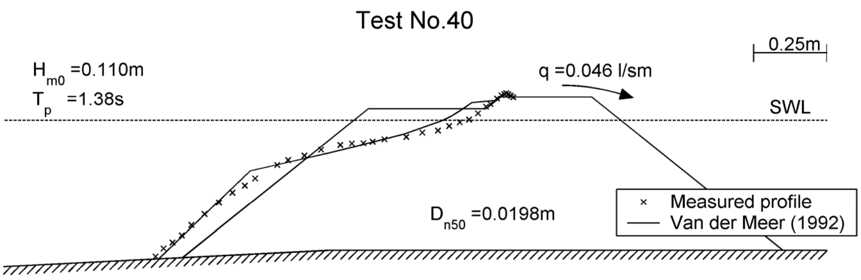



Test No.36
H m0 =0.102m
T p =1.38s
q =0.015 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

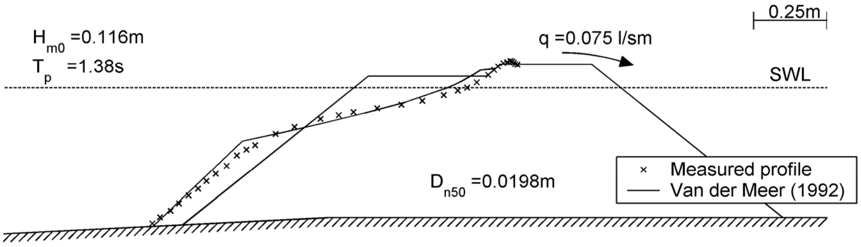

Test No.37
H m0 =0.102m
T p =1.38s
q =0.025 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.38
H m0 =0.105m
T p =1.38s
q =0.033 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

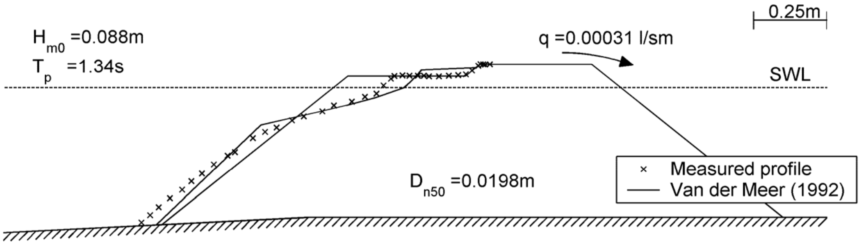

Test No.39
H m0 =0.108m
T p =1.38s
q =0.037 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.40
H m0 =0.110m
T p =1.38s
q =0.046 l/sm
0.25m
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

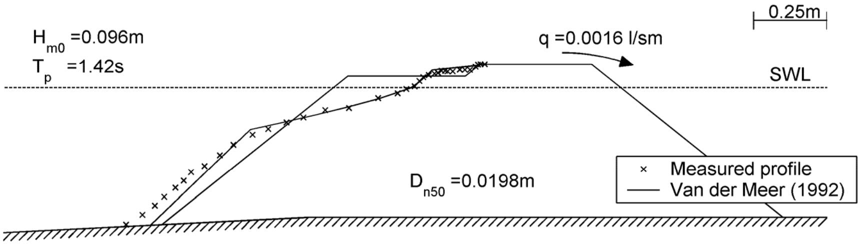
Test No.41



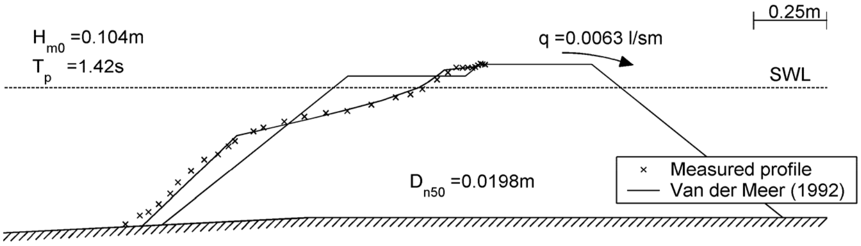
Test No.42



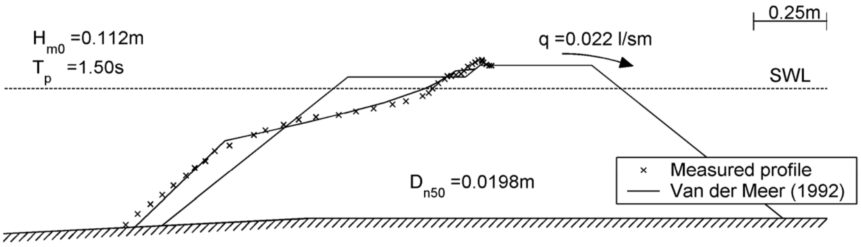
Test No.43

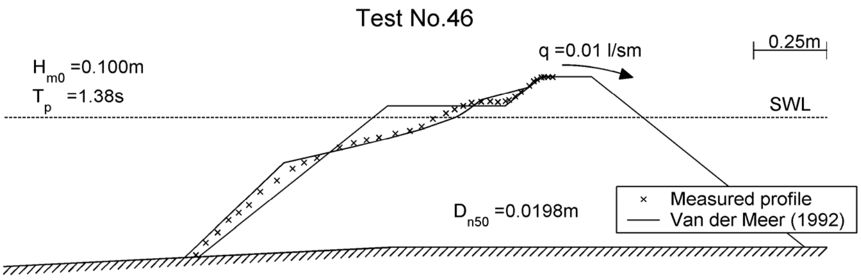


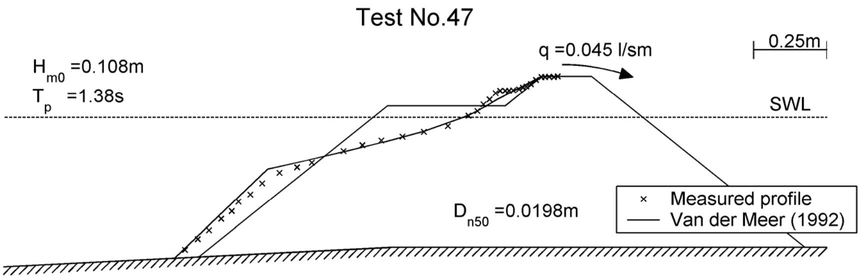
Test No.44

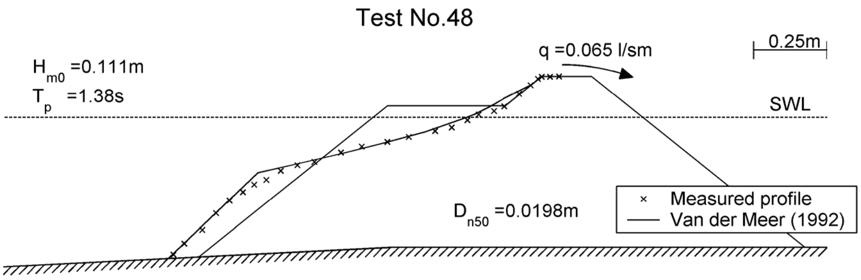


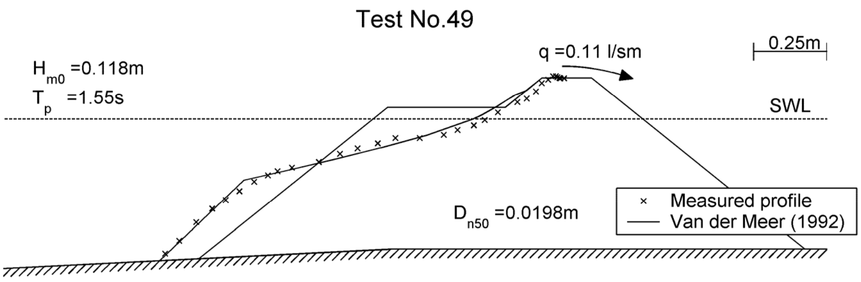
Test No.45

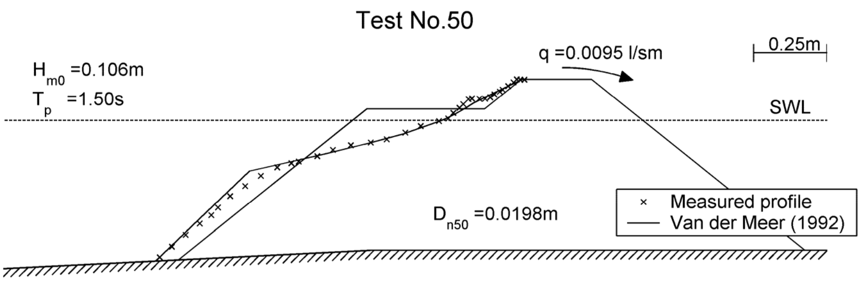



Test No.46
q =0.01 l/sm
0.25m
H m0 =0.100m
T p =1.38s
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

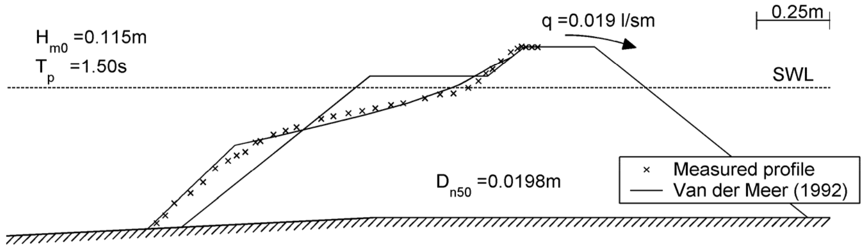

Test No.47
q =0.045 l/sm
0.25m
H m0 =0.108m
T p =1.38s
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.48
q =0.065 l/sm
0.25m
H m0 =0.111m
T p =1.38s
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

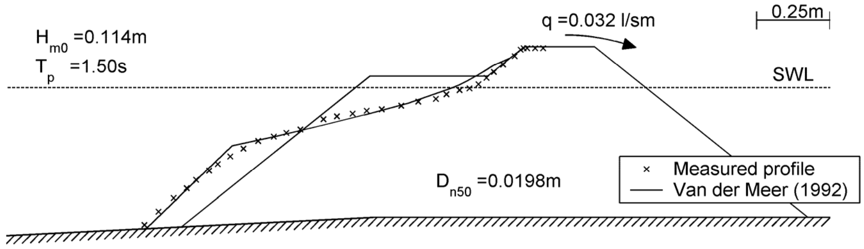

Test No.49
q =0.11 l/sm
0.25m
H m0 =0.118m
T p =1.55s
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.50
q =0.0095 l/sm
0.25m
H m0 =0.106m
T p =1.50s
SWL
D n50 =0.0198m
Measured profile
Van der Meer (1992)

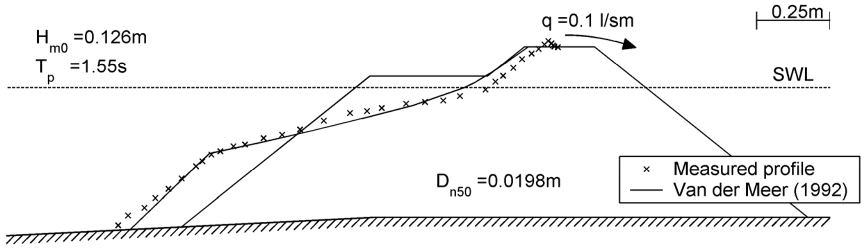
Test No.51



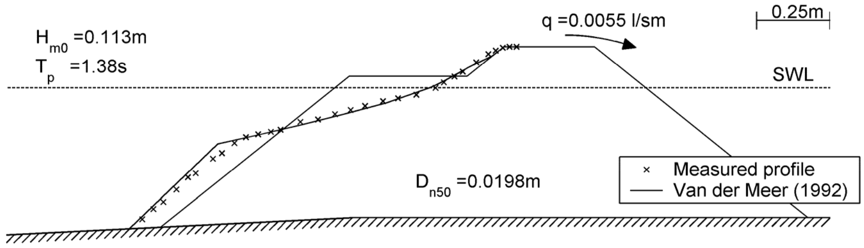
Test No.52



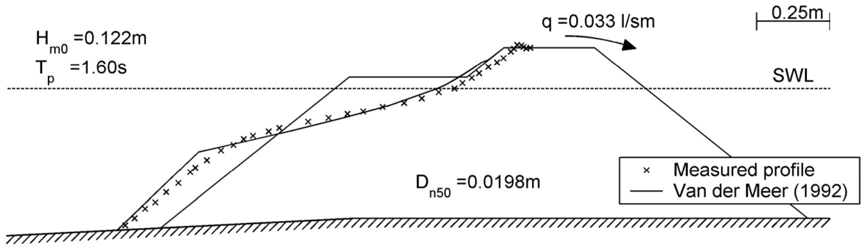
Test No.53

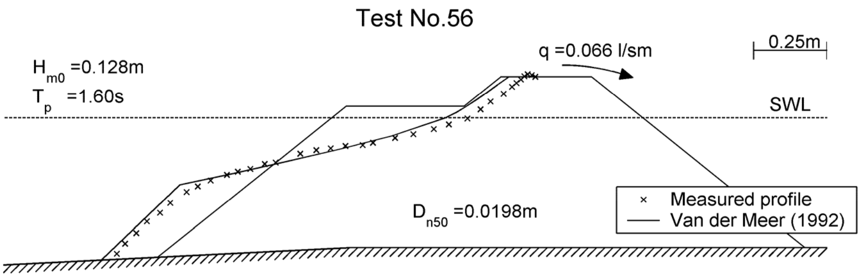


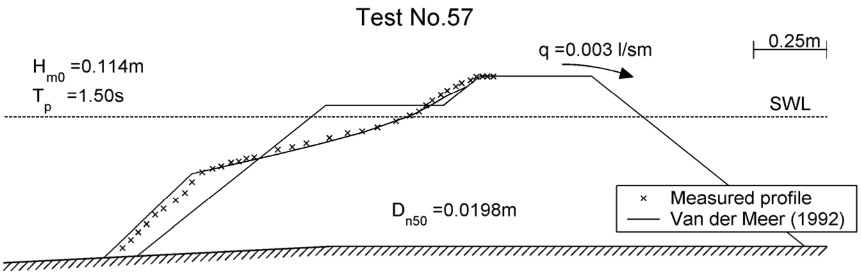
Test No.54

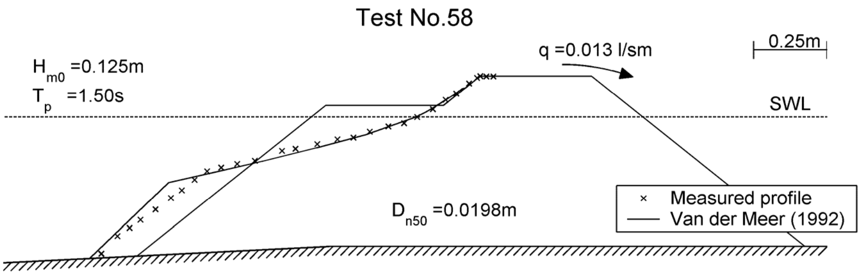


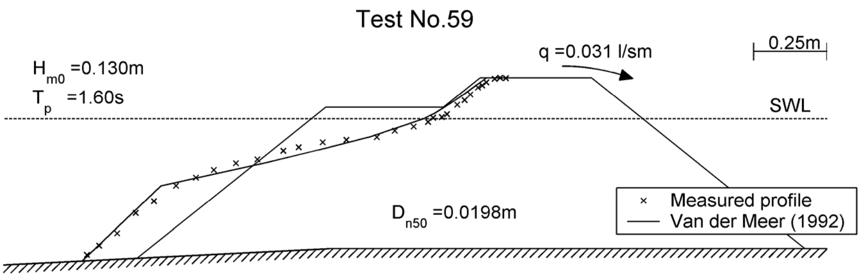
Test No.55

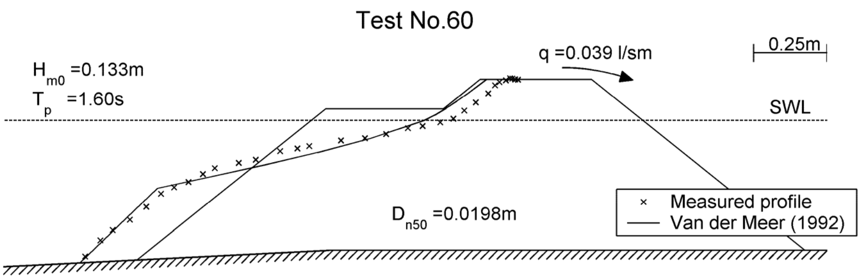



Test No.56
H_{m0} =0.128m
T_p =1.60s
q =0.066 l/sm
0.25m
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

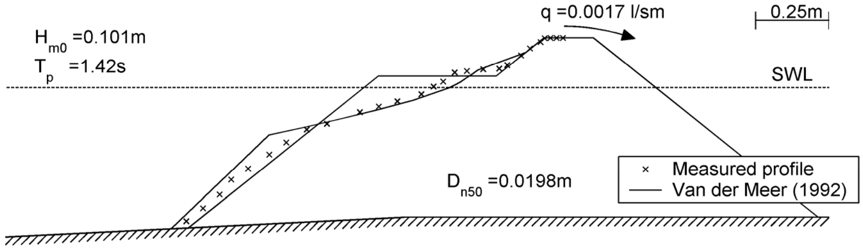

Test No.57
H_{m0} =0.114m
T_p =1.50s
q =0.003 l/sm
0.25m
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)


Test No.58
H_{m0} =0.125m
T_p =1.50s
q =0.013 l/sm
0.25m
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

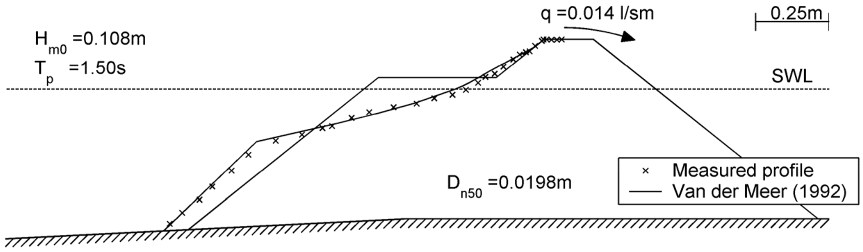

Test No.59
H_{m0} =0.130m
T_p =1.60s
q =0.031 l/sm
0.25m
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)


Test No.60
H_{m0} =0.133m
T_p =1.60s
q =0.039 l/sm
0.25m
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

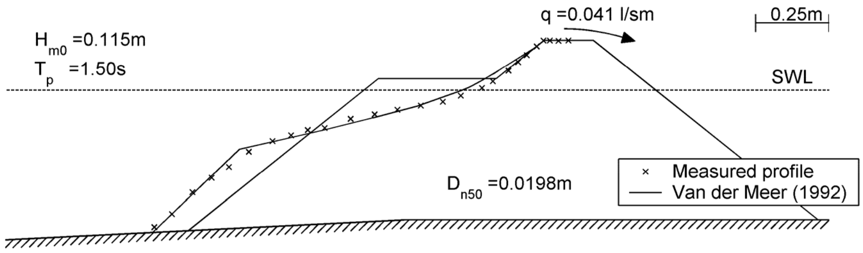
Test No.61



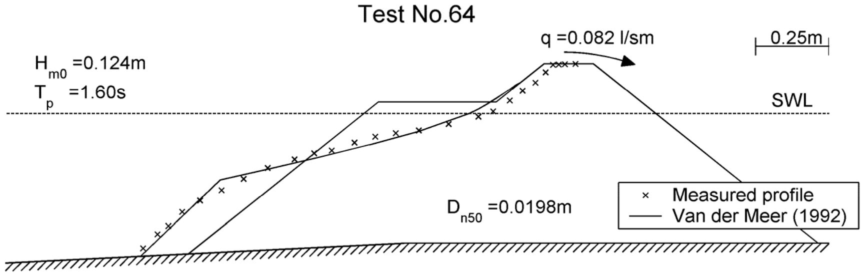
Test No.62



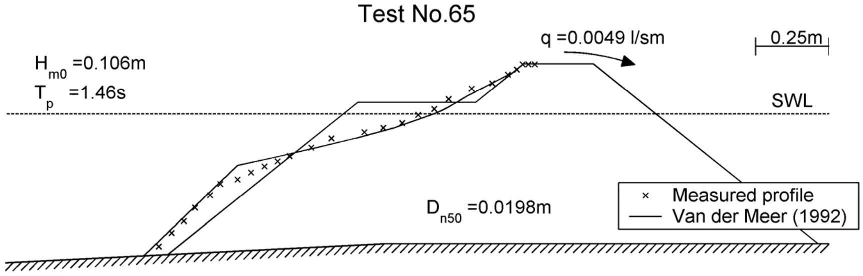
Test No.63

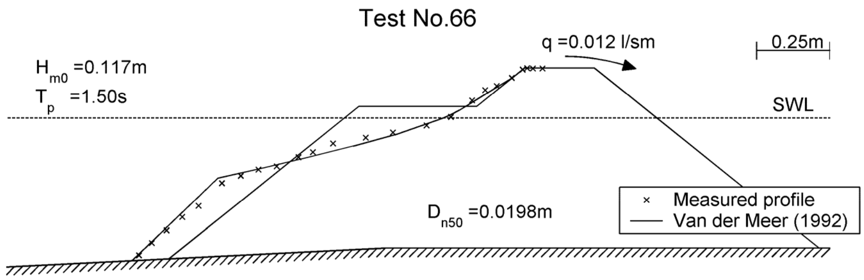


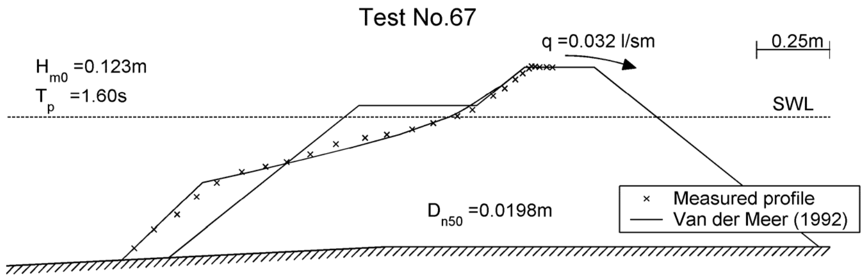
Test No.64

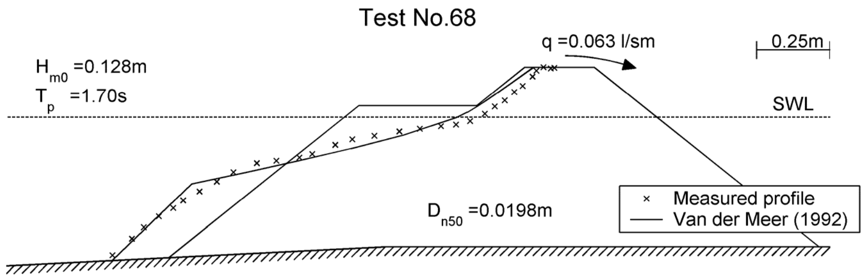


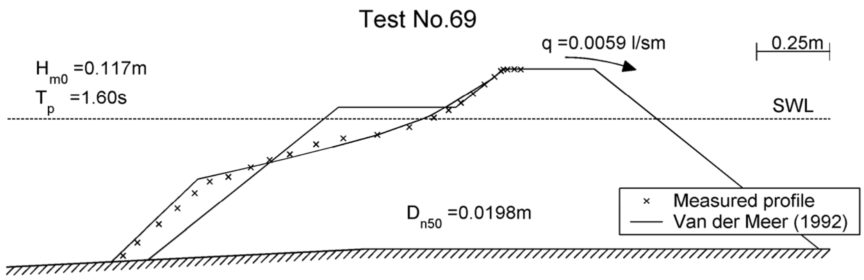
Test No.65

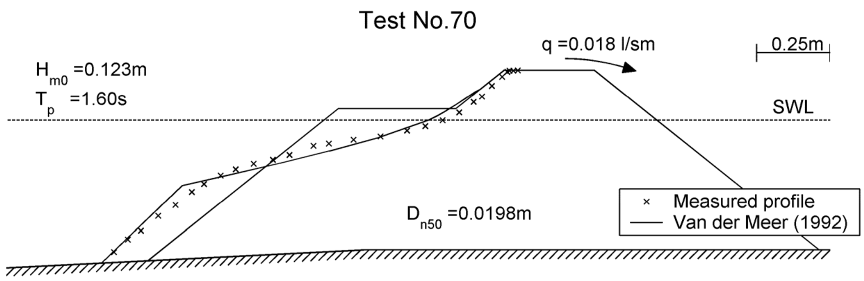



Test No.66
q =0.012 l/sm
0.25m
H_{m0} =0.117m
T_p =1.50s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

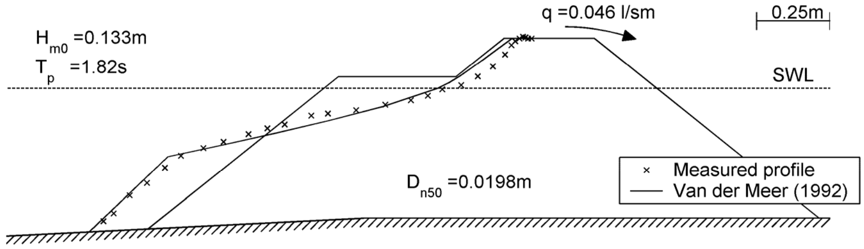

Test No.67
q =0.032 l/sm
0.25m
H_{m0} =0.123m
T_p =1.60s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)


Test No.68
q =0.063 l/sm
0.25m
H_{m0} =0.128m
T_p =1.70s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

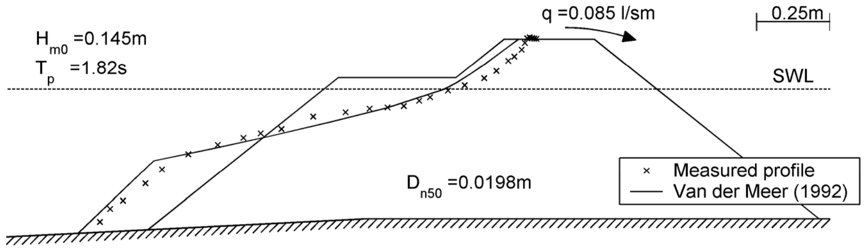

Test No.69
q =0.0059 l/sm
0.25m
H_{m0} =0.117m
T_p =1.60s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)


Test No.70
q =0.018 l/sm
0.25m
H_{m0} =0.123m
T_p =1.60s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)

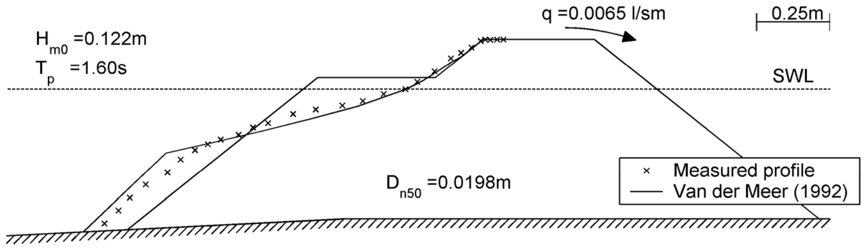
Test No.71



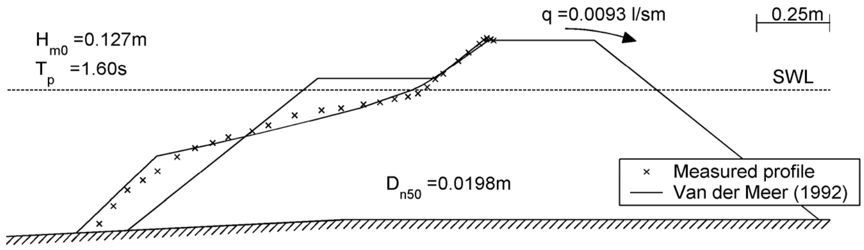
Test No.72



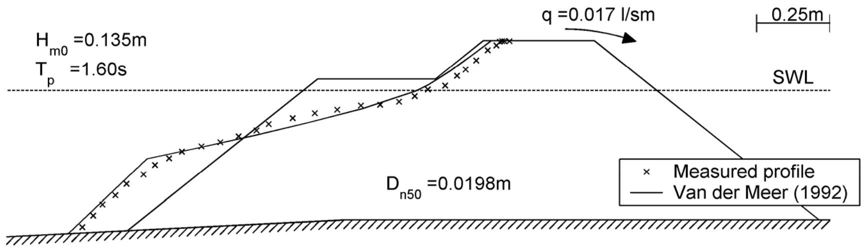
Test No.73

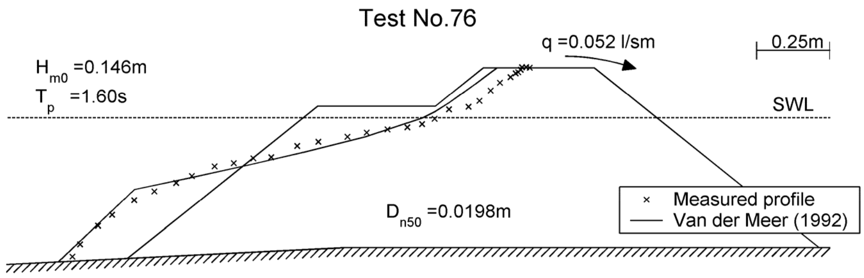


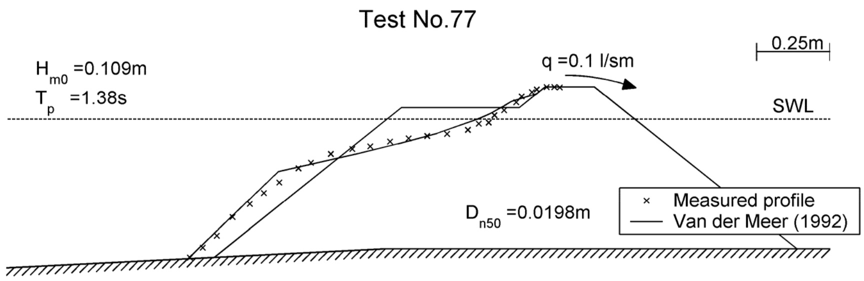
Test No.74

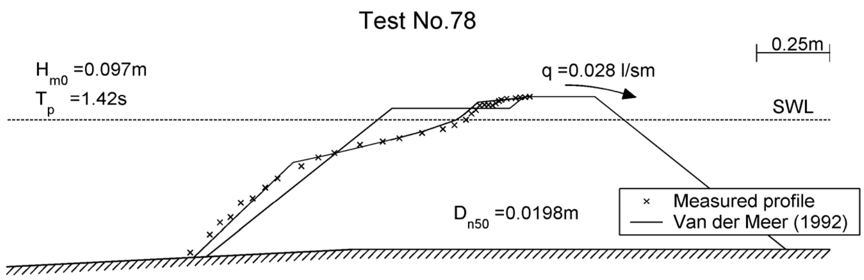


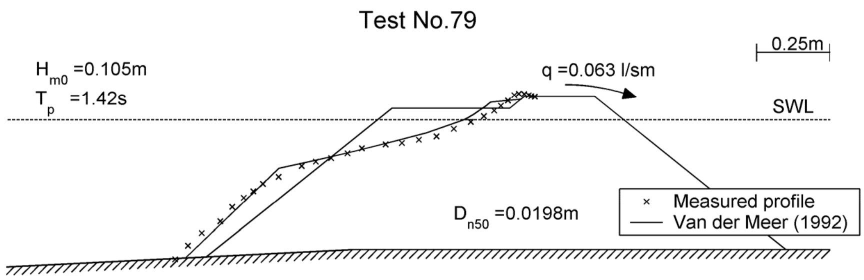
Test No.75

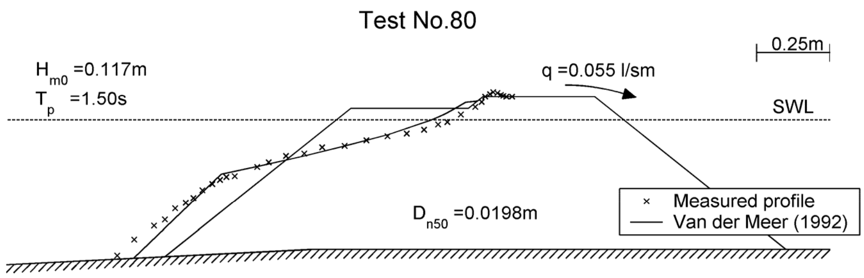


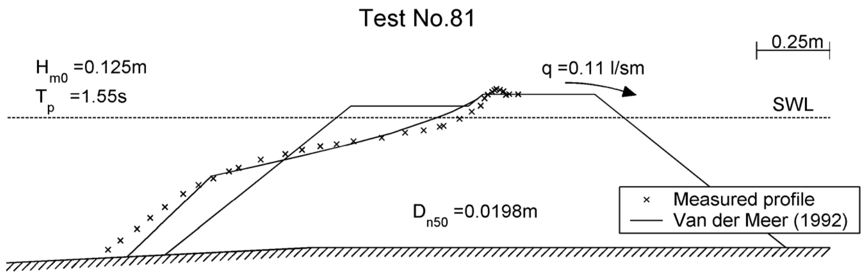

Test No.76
Hm0 =0.146m
Tp =1.60s
q =0.052 l/sm
0.25m
SWL
Dn50 =0.0198m
Measured profile
Van der Meer (1992)

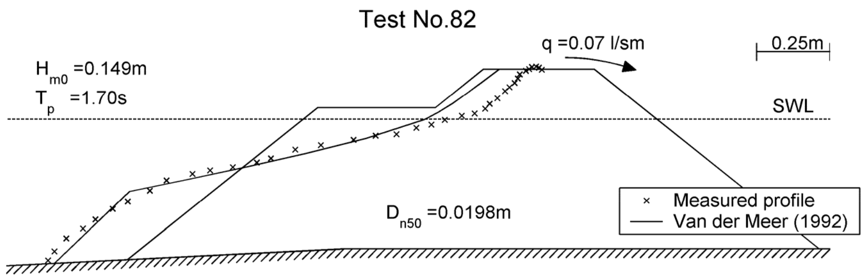

Test No.77
Hm0 =0.109m
Tp =1.38s
q =0.1 l/sm
0.25m
SWL
Dn50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.78
Hm0 =0.097m
Tp =1.42s
q =0.028 l/sm
0.25m
SWL
Dn50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.79
Hm0 =0.105m
Tp =1.42s
q =0.063 l/sm
0.25m
SWL
Dn50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.80
Hm0 =0.117m
Tp =1.50s
q =0.055 l/sm
0.25m
SWL
Dn50 =0.0198m
Measured profile
Van der Meer (1992)


Test No.81
0.25m
H_{m0} =0.125m
T_p =1.55s
q =0.11 l/sm
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)


Test No.82
q =0.07 l/sm
0.25m
H_{m0} =0.149m
T_p =1.70s
SWL
D_{n50} =0.0198m
Measured profile
Van der Meer (1992)