2-D Model Test Study of the Suape Breakwater, Brazil

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Publication date:
2009

Document Version
Publisher's PDF, also known as Version of record

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Carried out under contract for:

ODEBRECHT-OAS-ANDRADE GUITERREZ Consortium – Brazil

AALBORG UNIVERSITY
Department of Civil Engineering

DCE Contract Report No. 70

Aalborg University
Department of Civil Engineering
Division of Water and Soil

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by

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October 2009

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Published 2009 by
Aalborg University
Department of Civil Engineering
Sohngaardsholmsvej 57,
DK-9000 Aalborg, Denmark

Printed in Aalborg at Aalborg University

DCE Contract Report No. 70

Introduction

This report deals with a two-dimensional model test study of the extension of the breakwater in Suape, Brazil. One cross-section was tested for stability and overtopping in various sea conditions. The length scale used for the model tests was 1:35. Unless otherwise specified all values given in this report are prototype values by assuming Froude scaling.

Ph.D. Thomas Lykke Andersen were in charge of the model tests and assisted by Aurimas Sipavicius and Alban Le Querré. Engineer assistant Niels Drustrup and Kurt S. Sørensen assisted in the laboratory with the building and instrumentation of the models. Professor, H. F. Burcharth assisted with the planning of the model testing, test observations and the reporting.

The model test work including planning, testing and reporting was performed in the period September 2009 – October 2009. For further information contact Thomas Lykke Andersen (tla@civil.aau.dk).

Sea States

The wave spectrum used in all tests is JONSWAP with peak enhancement factor $\gamma = 5$. The following sea states have been tested on respectively low water level and high water level:

	Water Level +0.2 m,			
Sign. wave height, H_{m0} [m]	2	3	4	4.5
Peak wave period, T_p [s]	8	8	8	11
Number of waves	3000	3000	3000+3000	1000

Table 1: Tested sea conditions on low water level.

	Water Level +2.4 m					
Sign. wave height, H_{m0} [m]	3	3.5	4	4.5	5	5.5
Peak wave period, T_p [s]	8/11	8/11	8/11	8/11	8	8
Number of waves	3000	3000	3000+3000	1000	3000	1000

Table 2: Tested sea conditions on high water level.

Test Setup

A two-dimensional model was constructed in scale 1:35 in a 1.5 m wide and 25 m long wave flume. Fig. 1 shows the test setup in the flume. The bottom in the flume was horizontal in the first 6.5 m (model scale), then a small step followed by an approximately 1:100 slope that continues till just before the model. In order to separate into incident and reflected waves three resistance type wave gauges were installed both at the deeper part near the paddle where the bottom is horizontal and at the toe of the breakwater.

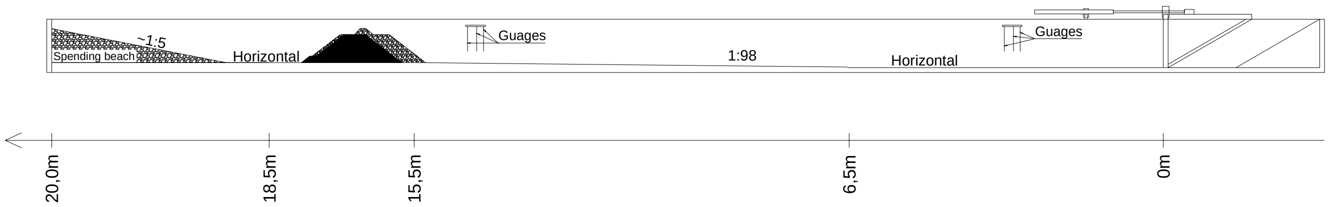

Figure 1: Layout in flume.

Pictures of examples of the cross-section tested are shown in Fig. 2. As seen from these pictures the overtopping water is led to an overtopping tank by a 0.3 m wide ramp extending from the rear corner of the crest. The overtopping volume was registered by weighting the tank after each test.

Figure 2: Pictures of the cross section and the overtopping ramp and tank.

Damage/deformation of the breakwater was registered by photo-technique in all tests and supplemented by some profiling by an in-house developed PC-controlled laser profiler with step motors to move the profiler in all three directions, cf. Fig. 3. The profiler is controlled from the computer program EPro also developed in-house [Aalborg University 2008b]. In the present case the profiler was setup to measure the profile in a grid with 1 cm spacing across the flume and 2 cm along the flume.

Figure 3: Pictures of laser profiler and the initial cross section viewed from the harbour side.

Tested Cross-Section

The cross-section proposed by the client for the Suape breakwater is shown in Fig. 4. The client has proposed three stone classes for the different layers of the breakwater. In the figures is also given the range of design water level variations (LWL to HWL).

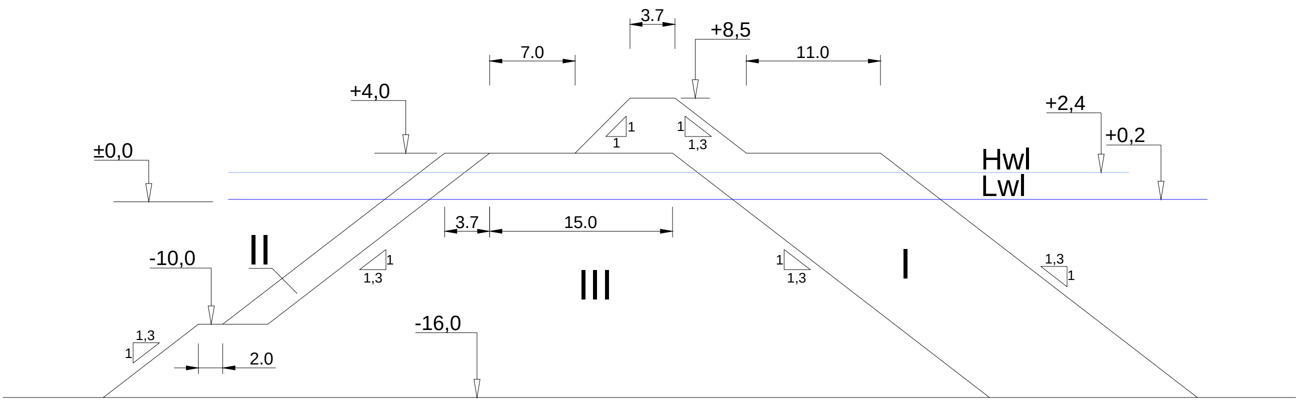

Figure 4: Cross section.

I: Primary Armour layer
II: Secondary Armour Layer
III: Core

Materials

Table 3 shows materials property as defined by the client for each layers in the prototype. The stones of the primary and secondary armour have been hand sorted. This has made it possible to quite accurately match the prescribed armour grading. For the core material a very wide gradation is used in prototype, cf. Table 3. In the model a narrow gradation with sizes close to the lower limit is conservatively used.

	Mass of each stones [tonnes]
Primary armour layer	Between 2.0 and 8.0
Secondary armour layer	Between 2.0 and 3.0
Core	Between 0.005 and 2.0

Table 3: Prototype material properties as defined by client.

Weight measurements of individual stones were performed for 100 randomly selected units of each layer. Table 4 shows the main material parameters and Fig. 6 show the distribution curves.

	Primary armour layer	Secondary armour layer	Core
$W_{15}\ [kg]$	2690	2015	18.2
$W_{50}\ [kg]$	4588	2465	30.7
$W_{85}\ [kg]$	6903	2993	46.1
Mass density $\rho_s\ [kg/m^3]$	2680	2530	2810
$D_{n,50}\ [m]$	1.20	0.99	0.22
$f_g = D_{n,85}/D_{n,15}$	1.37	1.14	1.36

Table 4: Material properties.

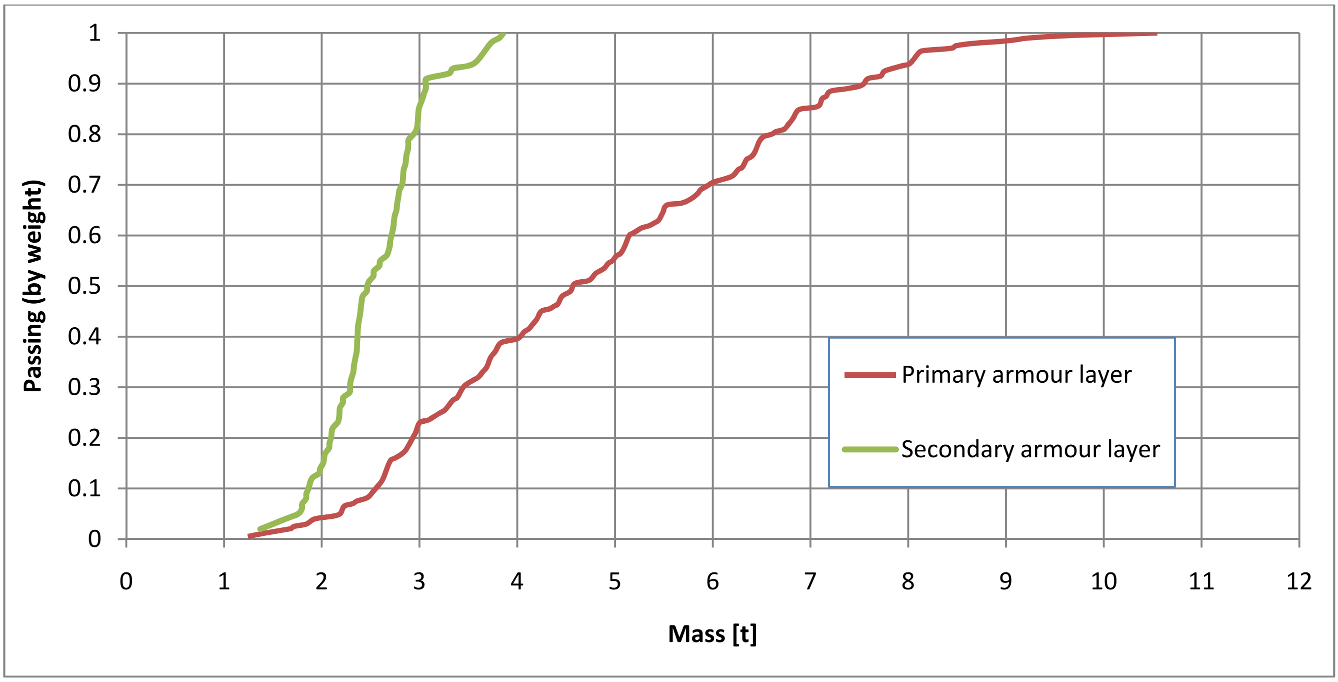

Figure 5: Gradation curve of secondary armour and primary armour material.

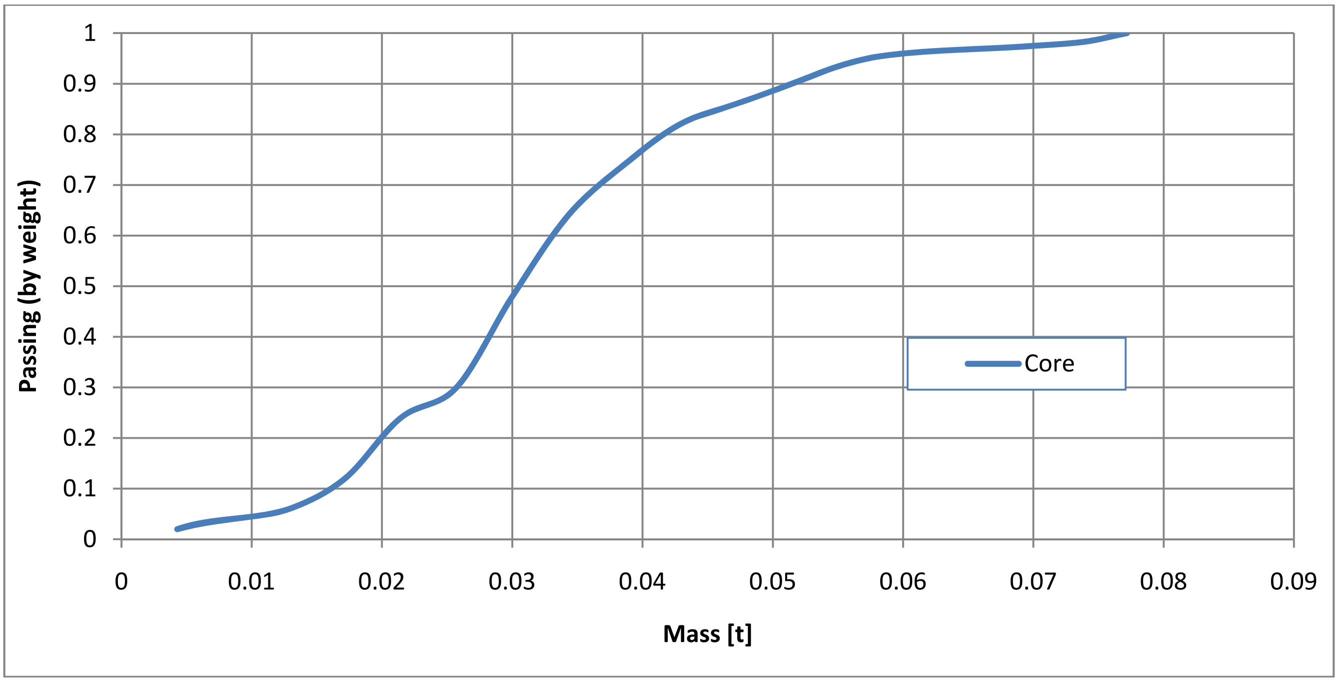

Figure 6: Gradation curve of core material.

Wave Generation and Data Analysis

The waves were generated using the AwaSys 5 program using a white noise filtering technique [Aalborg University 2008a]. Active absorption was used which in AwaSys is based on digital filtering of signals from two wave gauges positioned 3.0 and 3.3 metres from the paddle leading to a paddle correction signal.

The data acquisition was done using a DT9804 acquistion box and the WaveLab software package [Aalborg University 2008c]. The measured surface elevation time series was analysed and split into incident and reflected waves using the WaveLab software package. The separation into incident and reflected waves is based on the Mansard and Funke (1980) algorithm. Wave heights given below are the incident waves measured with the array close to the structure.

Average overtopping discharges per meter structure width were determined from the weight of the overtopping water.

Overview of Performed Tests and Overtopping Results

Table 5 gives an overview of the performed tests. Accumulative damages were observed as the profile was not rebuilt after each test.

File Name	$H_{m0,obtained}$ [m]	$T_{P,\ obtained}$ [s]	Water level [m]	Number of waves, N	$q_{prototype}$ [m^3/sm]
Test001	1.72	8.0	0.2	3000	0
Test002	3.17	8.0	0.2	3000	$1.5\cdot10^{-6}$
Test003	4.05	8.0	0.2	3000	$3.0\cdot10^{-5}$
Test004	3.91	8.0	0.2	3000	$7.8\cdot10^{-5}$
Test005	4.65	11.0	0.2	1000	$9.4\cdot10^{-4}$
Test006	2.99	8.0	2.4	3000	$1.6\cdot10^{-5}$
Test007	3.56	8.0	2.4	3000	$9.7\cdot10^{-5}$
Test008	4.03	8.0	2.4	3000	$3.8\cdot10^{-4}$
Test009	4.35	8.0	2.4	3000	$7.0\cdot10^{-4}$
Test010	4.91	8.0	2.4	3000	$4.0\cdot10^{-3}$
Test011	5.18	8.0	2.4	1000	$9.1\cdot10^{-3}$
Test012	3.07	11.0	2.4	3000	$7.5\cdot10^{-5}$
Test013	3.58	11.0	2.4	3000	$6.7\cdot10^{-4}$
Test014	4.03	11.0	2.4	3000	$3.2\cdot10^{-3}$
Test015	4.61	11.0	2.4	3000	$1.4\cdot10^{-2}$

Table 5: Overview of performed tests for cross-section

It can be concluded that at the low water level (+0.2 m), average overtopping discharge is below 1 l/sm for a wave height of 4.7 m. For high water level (+2.4) overtopping discharges are significantly larger due to the smaller freeboard, cf. Fig. 7. It should be mentioned that overtopping discharges are subjected to scale effects and especially the lower discharges will be significantly higher in prototype, cf. De Rouck et al. (2005) and Lykke Andersen (2006).

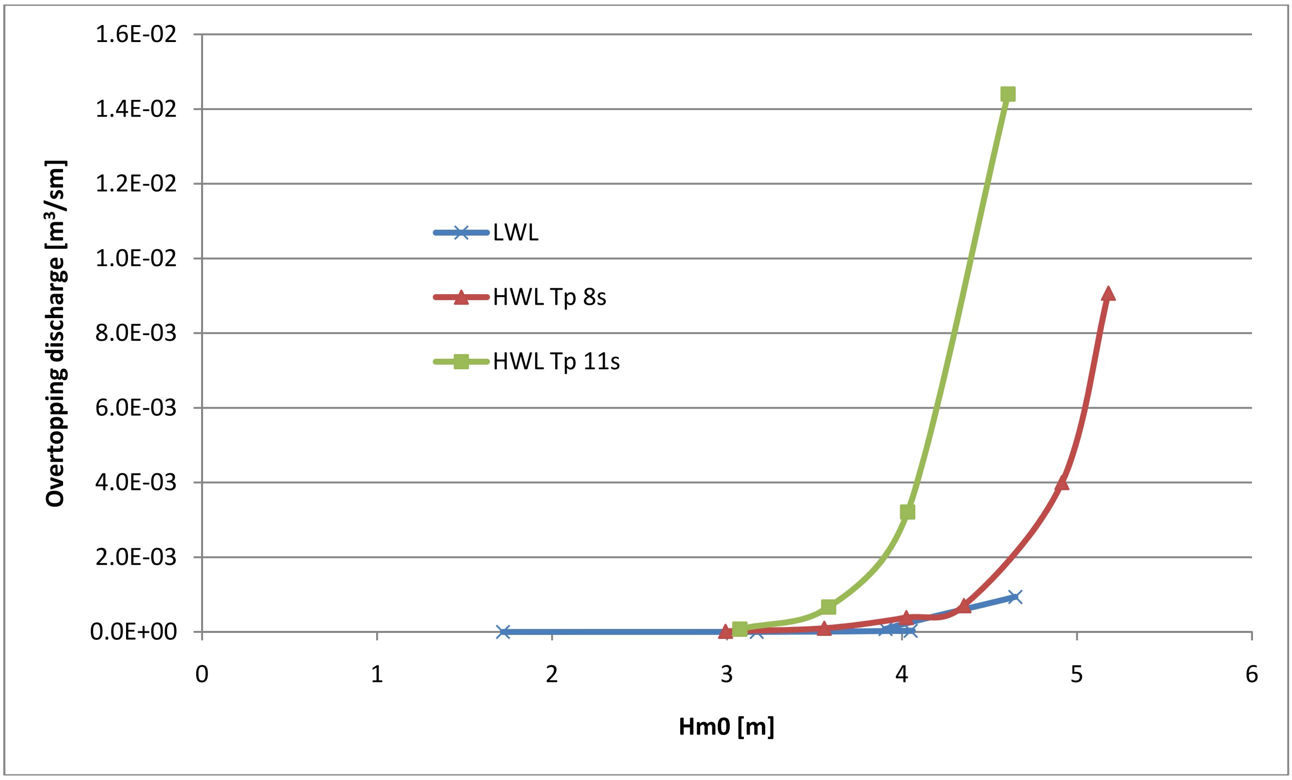

Figure 7: Overview of overtopping results.

At high water level overtopping discharges were so large that significant damages to the crest took place for significant wave heights above 4.0 m. However, erosion of the core material on the roadway started already around H_{m0} = 3 m.

Profile Deformations

The measured profiles presented in the report are the average profile over the width of the model but discarding approximately 20 cm at each side of the flume due to wall effects. The damage is observed not to be uniformly distributed over the width. This is mainly due to the wide gradation used for the armour stones which leads to some parts of the armour layer are stronger than other parts. Especially for smaller damages large variations occur, but when the berm recession (*Rec*) is close to the initial berm width (*B*) the damage is close to uniformly distributed, both parameters defined in Fig. 8.

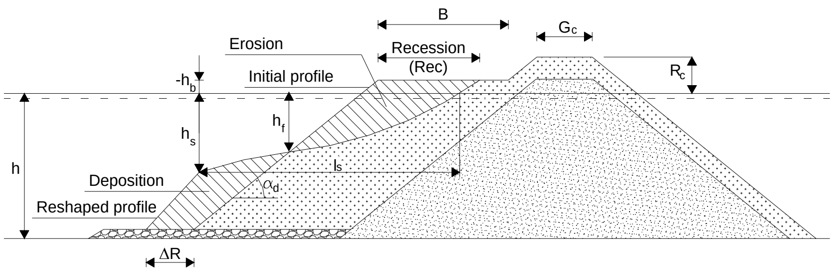

Figure 8: Definition of initial and reshaped profile parameters.

Figs. 9 and 10 shows comparison between the initial profile and the reshaped profile after each test as measured by the laser profiler. It can be seen that the recession is acceptable in all cases as *Rec*<*B*. However, for high water level conditions and H_{m0} = 4.61 m and T_P = 11 s (Test 015), some significant erosion of the crest took place. Fig. 11 shows a picture of the crest after test 15 and it can be seen damages to the crest and a significant part of the core material behind the crest is extracted and deposited on the rear armour. The damage to the unprotected core material in the roadway started approximately at H_{m0} = 3 m.

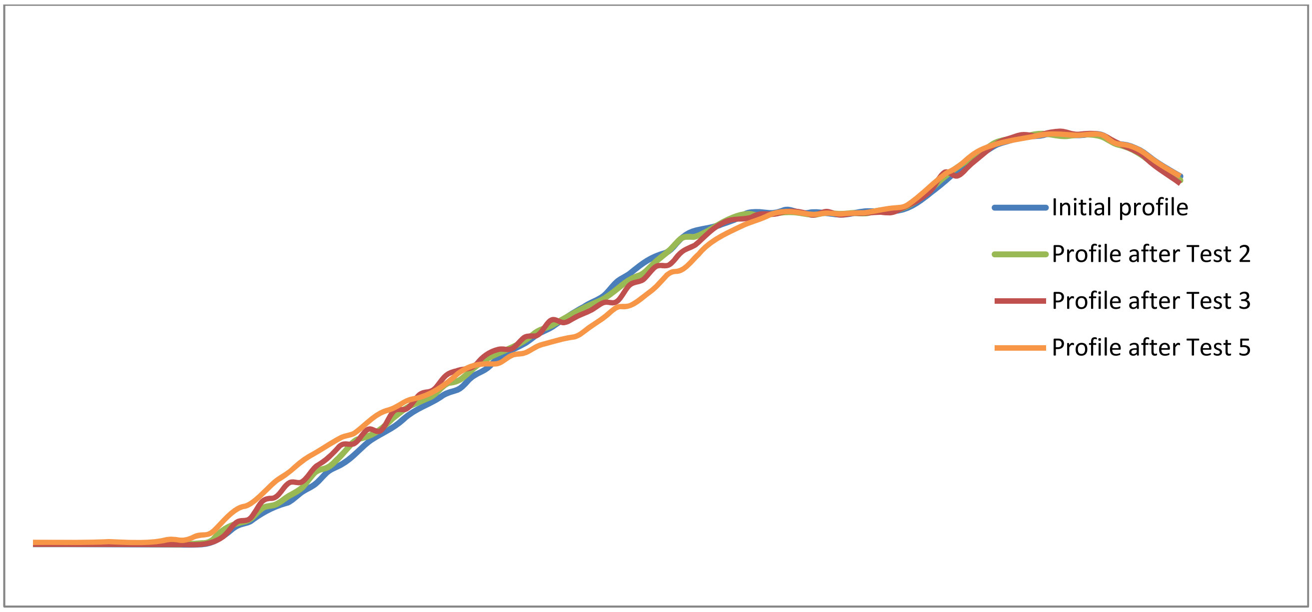

Figure 9: Measured profiles for LWL conditions.

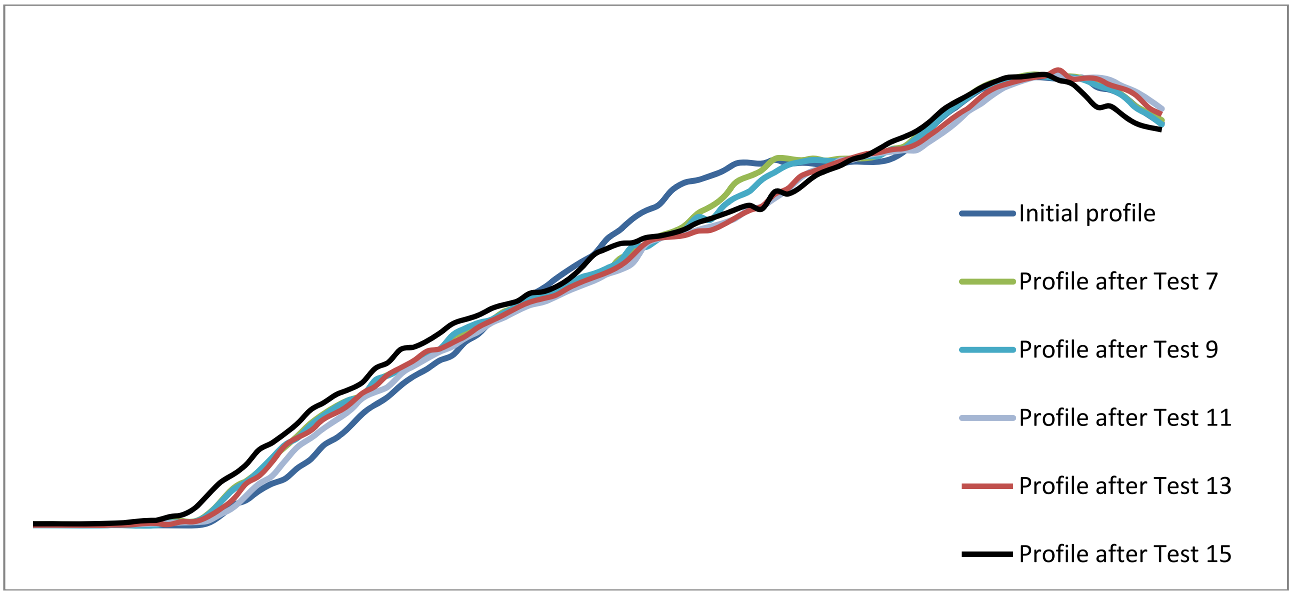

Figure 10: Measured profiles for HWL conditions.

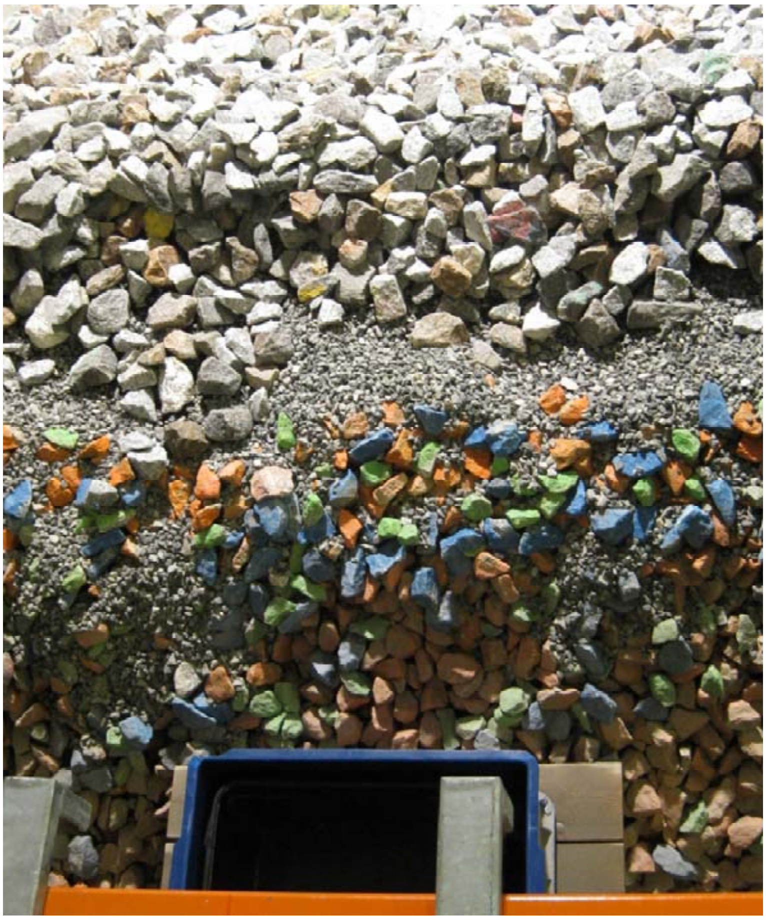

Figure 11: Damages to crest of structure after test 15.

Conclusions

A model test study for the new breakwater in Suape, Brazil has been carried out. Different design wave conditions were tested. For the low water level acceptable damages of the structure were measured and overtopping was below 1 l/sm even for extreme sea states. At high water level the recession is larger probably partly due to the cumulative effect. At high water level overtopping discharges are very high for the extreme sea states and significant damages to the crest occurred for significant wave heights above 4.0 m. Erosion of the unprotected core material in the roadway started for significant wave heights larger than app. 3.0 m for both wave periods tested. The eroded core material was deposited on the rear slope armour layer. This erosion could probably be prevented by covering the area with secondary armour stones (2t to 3t).

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Aalborg University (2008a) AwaSys homepage. http://hydrosoft.civil.aau.dk/AwaSys

Aalborg University (2008b) EPro homepage. http://hydrosoft.civil.aau.dk/epro

Aalborg University (2008c) WaveLab 3 homepage. http://hydrosoft.civil.aau.dk/wavelab

De Rouck, J., Geeraerts, J., Troch, P., Kortenhaus, A., Pullen, T. and Franco, L. (2005). New results on scale effects for wave overtopping. ICE2005, London.

Lykke Andersen, T. 2006. Hydraulic Response of Rubble Mound Breakwaters. Scale Effects – Berm Breakwaters. PhD thesis, Aalborg University.

Mansard, E. P. D., Funke, E. R. (1980). The Measurement of Incident and Reflected Spectra Using a Least Squares Method. Proc. 17th Coastal Engineering Conference, Sydney, Australia.

Appendix A: Photos of damages during individual tests

Test 01: WL = +0.2 m, H_{m0} = 1.7 m, T_p = 8.0 s, N = 3000

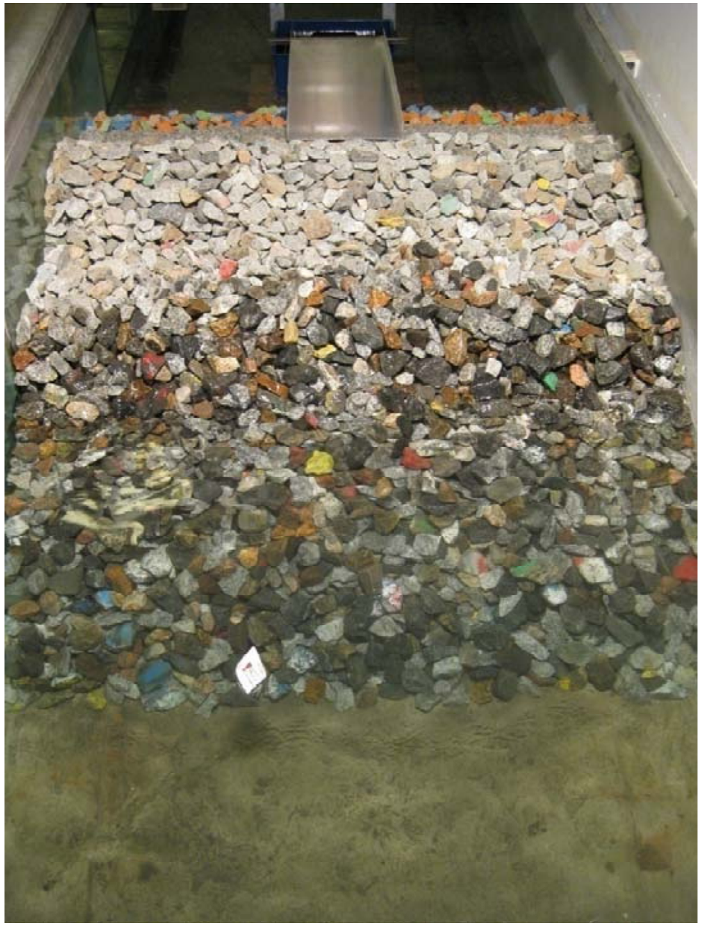

Figure 12: Before picture.

Figure 13: After picture.

Test 02: WL = +0.2 m, H_{m0} = 3.1 m, T_p = 8.0 s, N = 3000

Figure 14: Before picture.

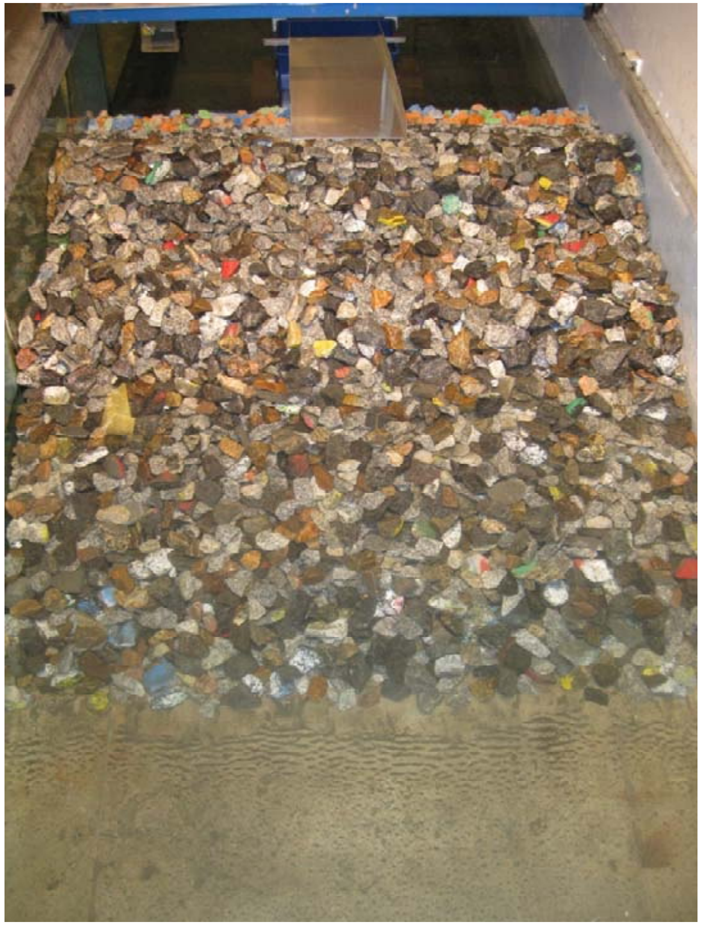

Figure 15: After picture.

Test 03: WL = +0.2 m, H_{m0} = 4.1 m, T_p = 8.0 s, N = 3000

Figure 16: Before picture.

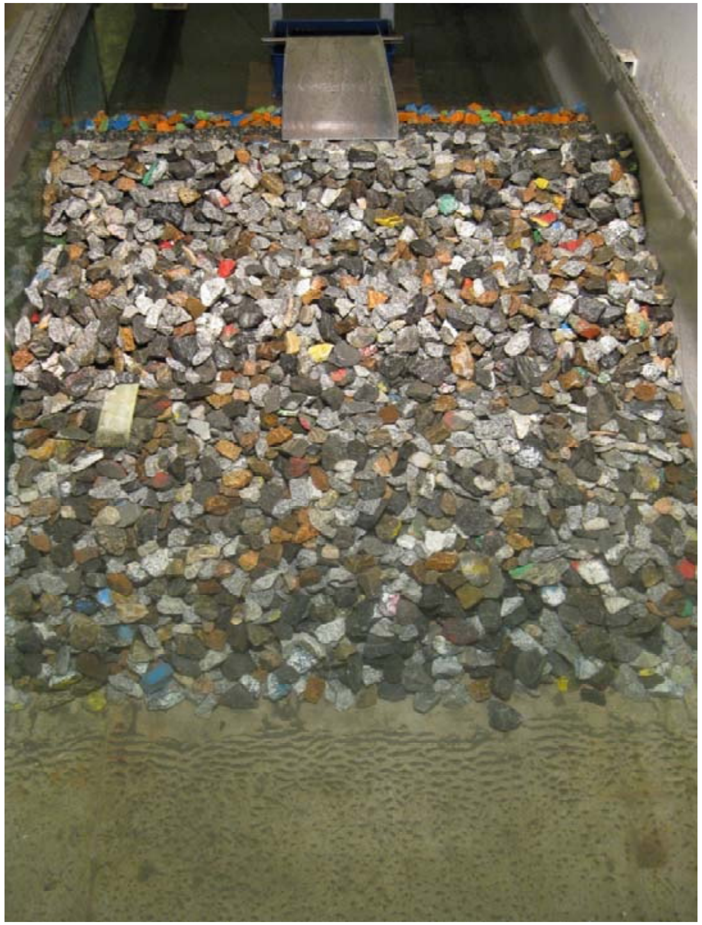

Figure 17: After picture.

Test 04: WL = +0.2 m, H_{m0} = 3.9 m, T_p = 8.0 s, N = 3000

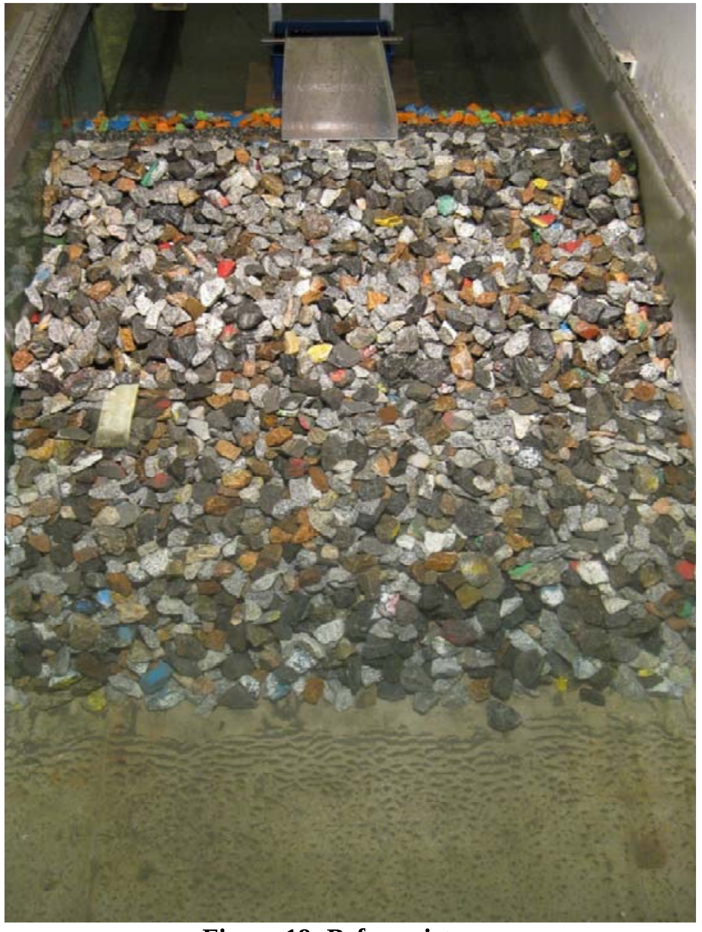

Figure 18: Before picture.

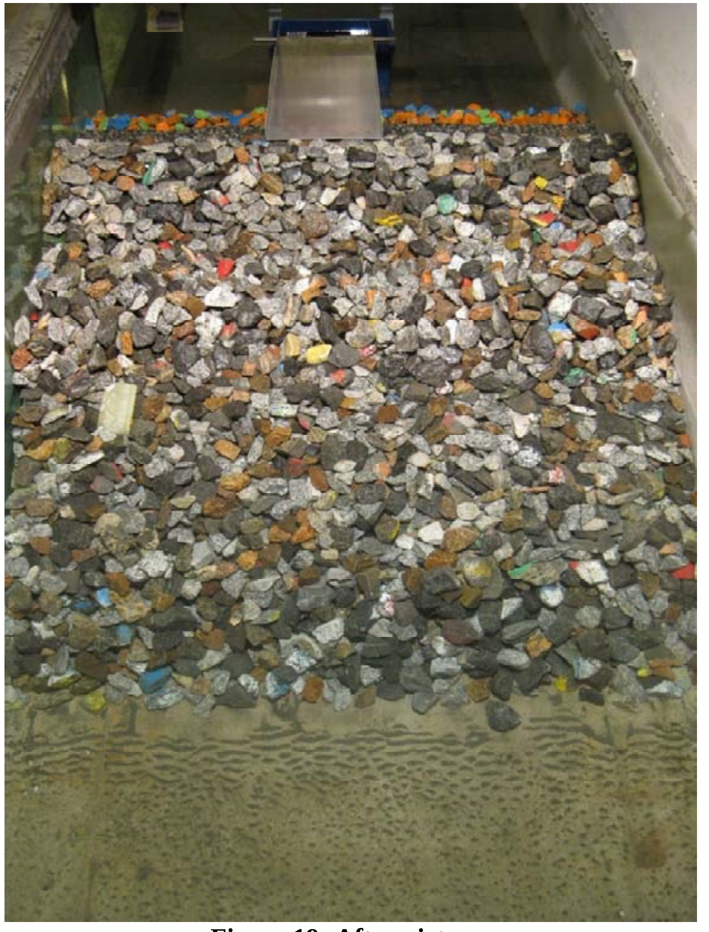

Figure 19: After picture.

Test 05: WL = +0.2 m, H_{m0} = 4.6 m, T_p = 11.0 s, N = 1000

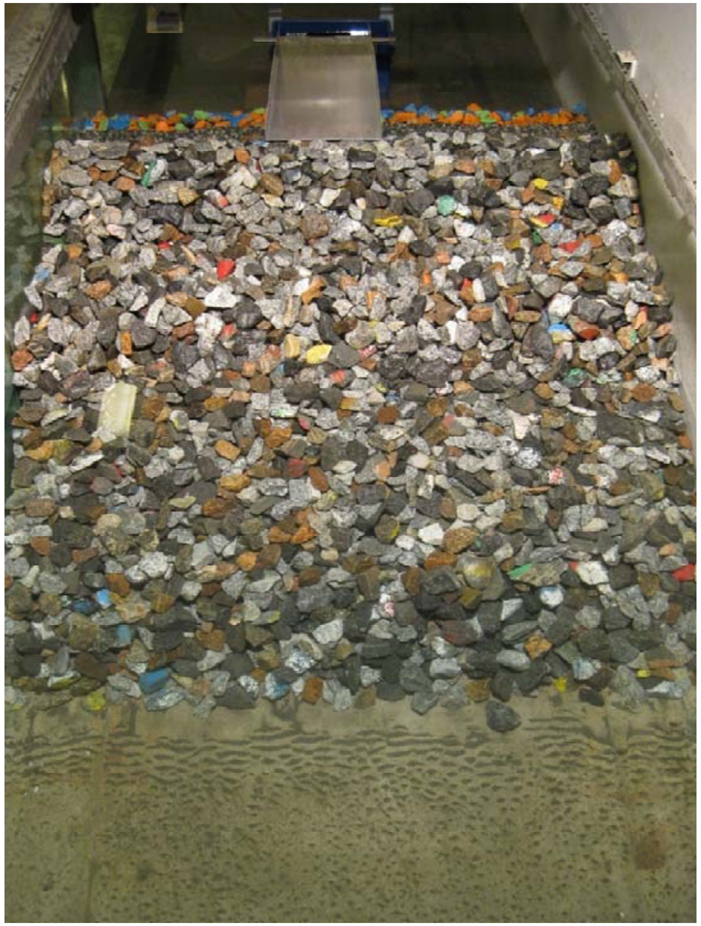

Figure 20: Before picture.

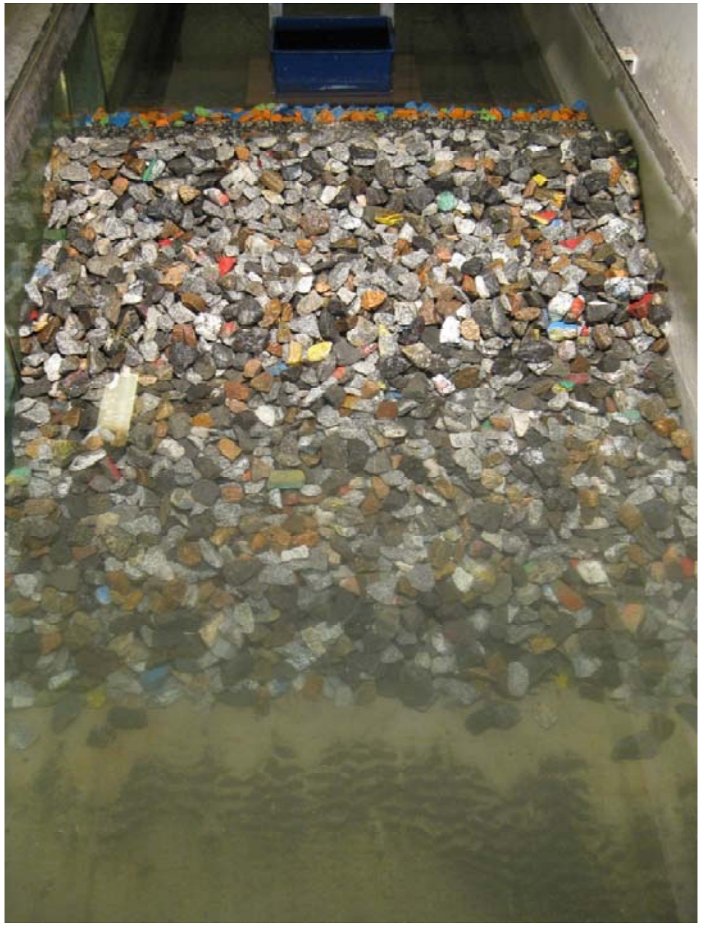

Figure 21: After picture.

Test 06: WL = +2.4 m, H_{m0} = 3.0 m, T_p = 8.0 s, N = 3000

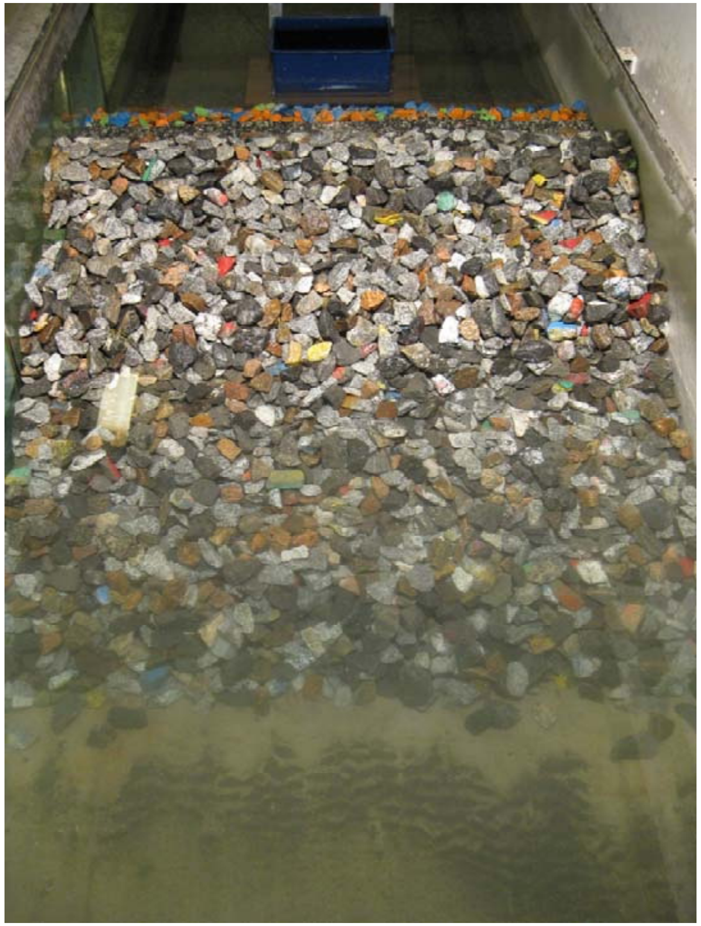

Figure 22: Before picture.

Figure 23: After picture.

Test 07: WL = +2.4 m, H_{m0} = 3.6 m, T_p = 8.0 s, N = 3000


Figure 24: Before picture.

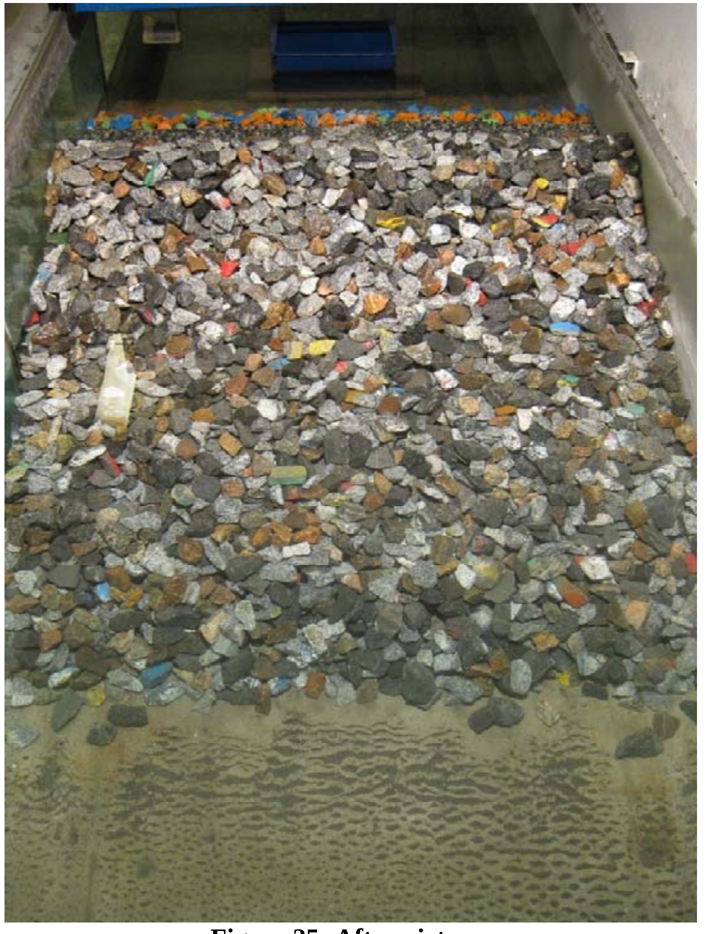
Figure 25: After picture.

Test 08: WL = +2.4 m, H_{m0} = 4.0 m, T_p = 8.0 s, N = 3000

Figure 26: Before picture.

Figure 27: After picture.

Test 09: WL = +2.4 m, H_{m0} = 4.4 m, T_p = 8.0 s, N = 3000

Figure 28: Before picture.

Figure 29: After picture.

Test 10: WL = +2.4 m, H_{m0} = 4.9 m, T_p = 8.0 s, N = 3000

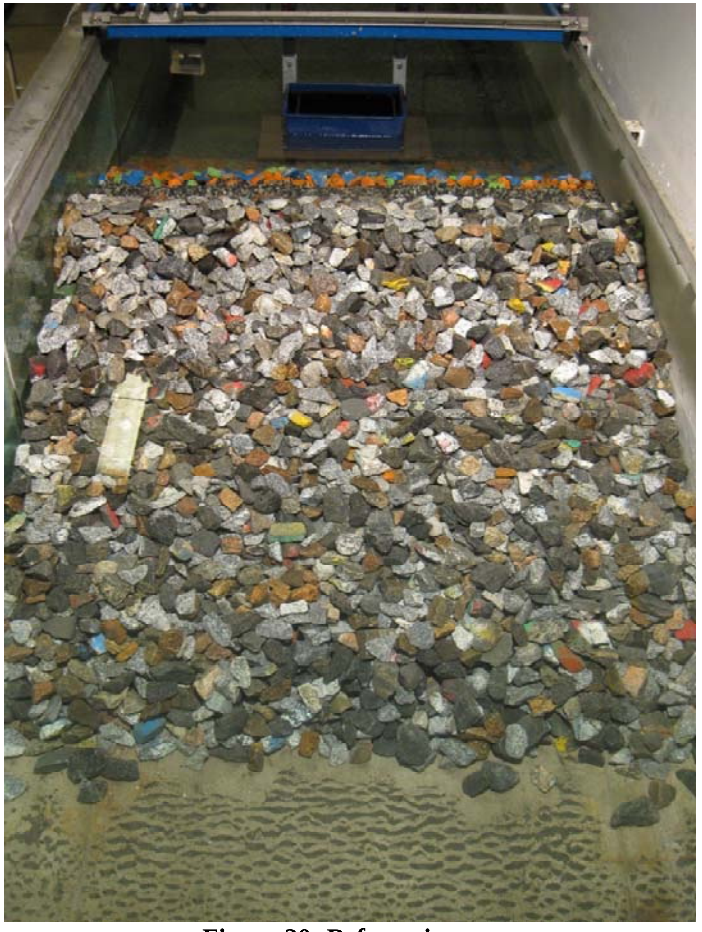

Figure 30: Before picture.

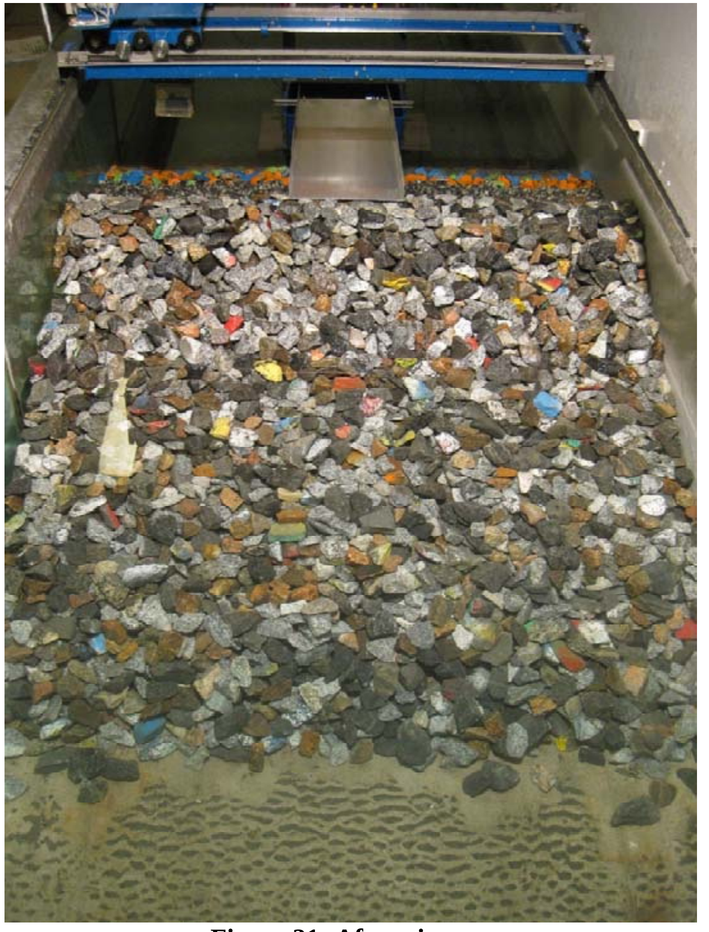

Figure 31: After picture.

Test 11: WL = +2.4 m, H_{m0} = 5.2 m, T_p = 8.0 s, N = 1000

Figure 32: Before picture.

Figure 33: After picture.

Test 12: WL = +2.4 m, H_{m0} = 3.1 m, T_p = 11.0 s, N = 3000

Figure 34: Before picture.

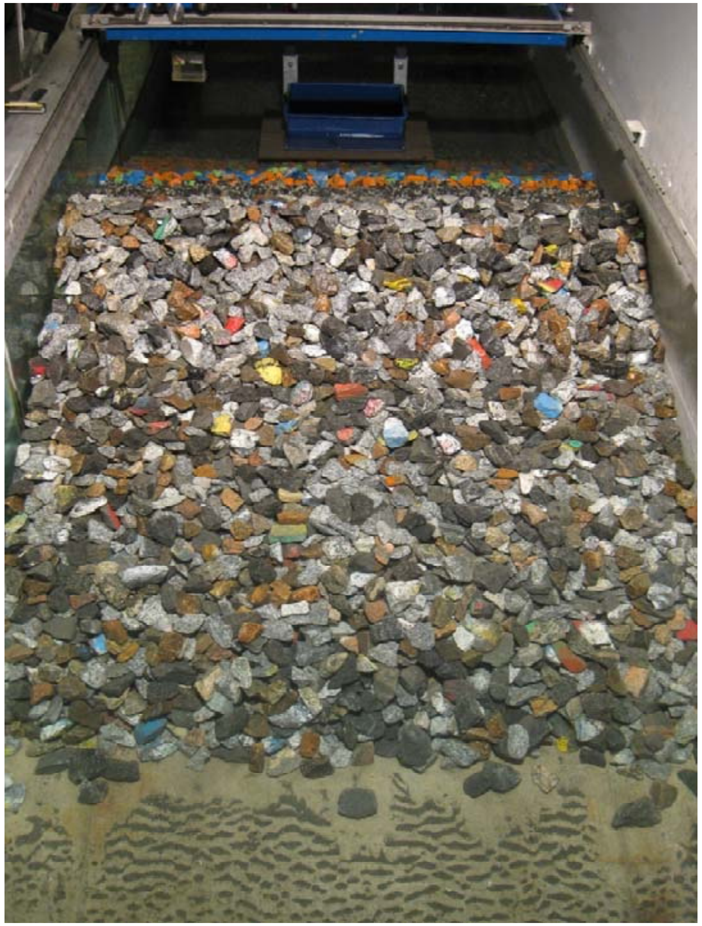

Figure 35: After picture.

Test 13: WL = +2.4 m, H_{m0} = 3.6 m, T_p = 11.0 s, N = 3000

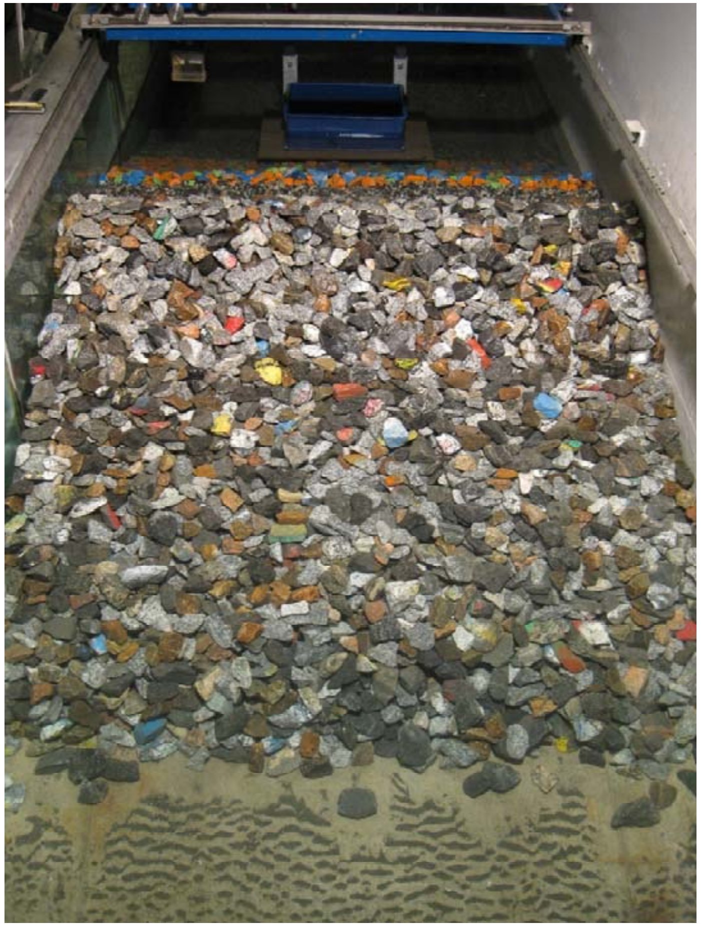

Figure 36: Before picture.

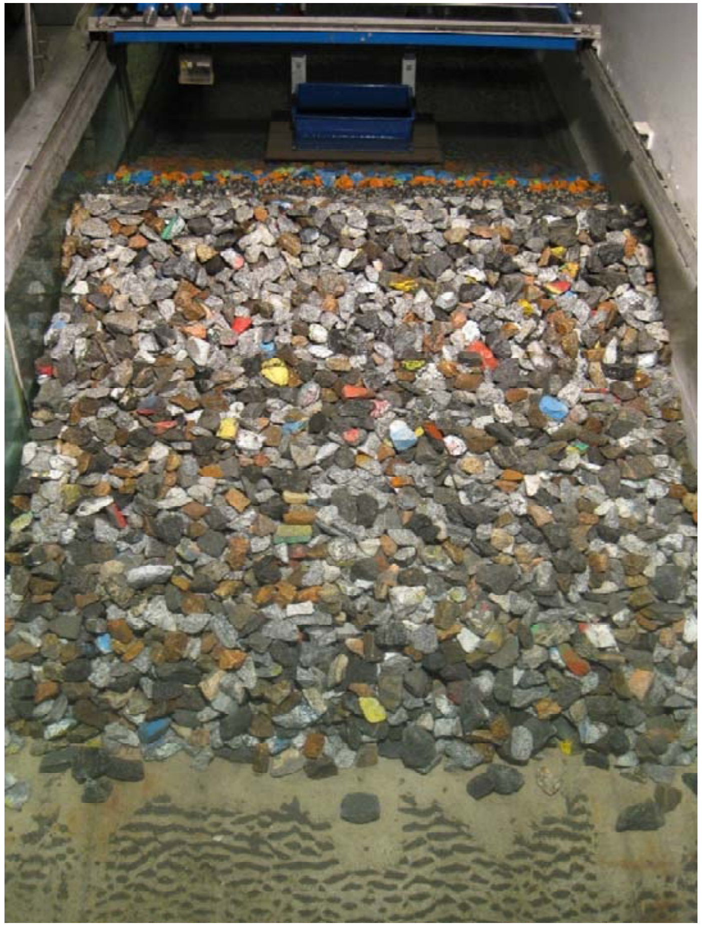

Figure 37: After picture.

Test 14: WL = +2.4 m, H_{m0} = 4.0 m, T_p = 11.0 s, N = 3000

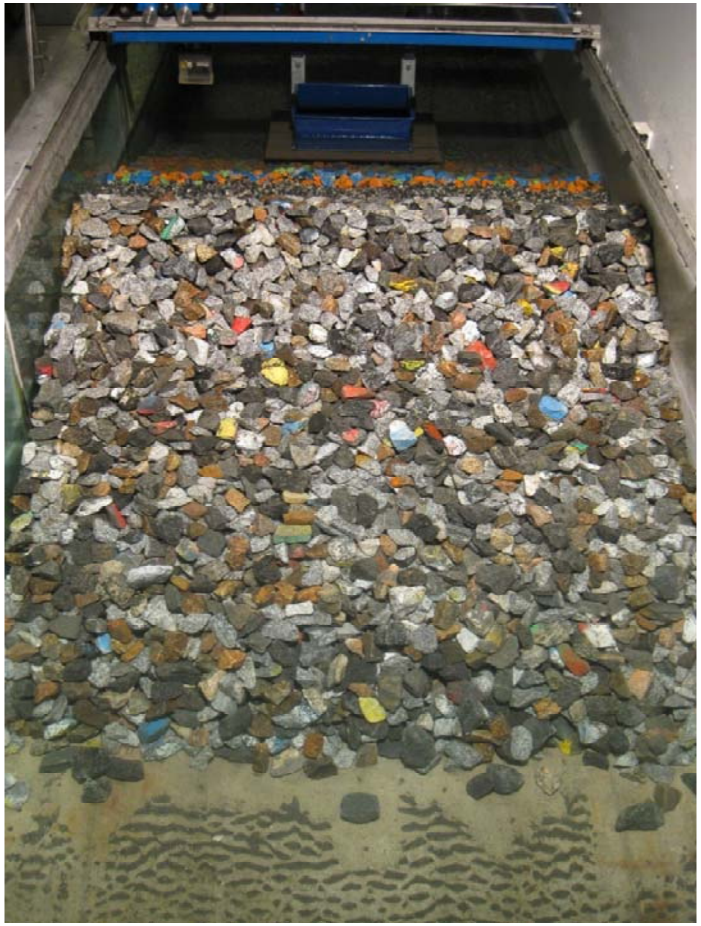

Figure 38: Before picture.

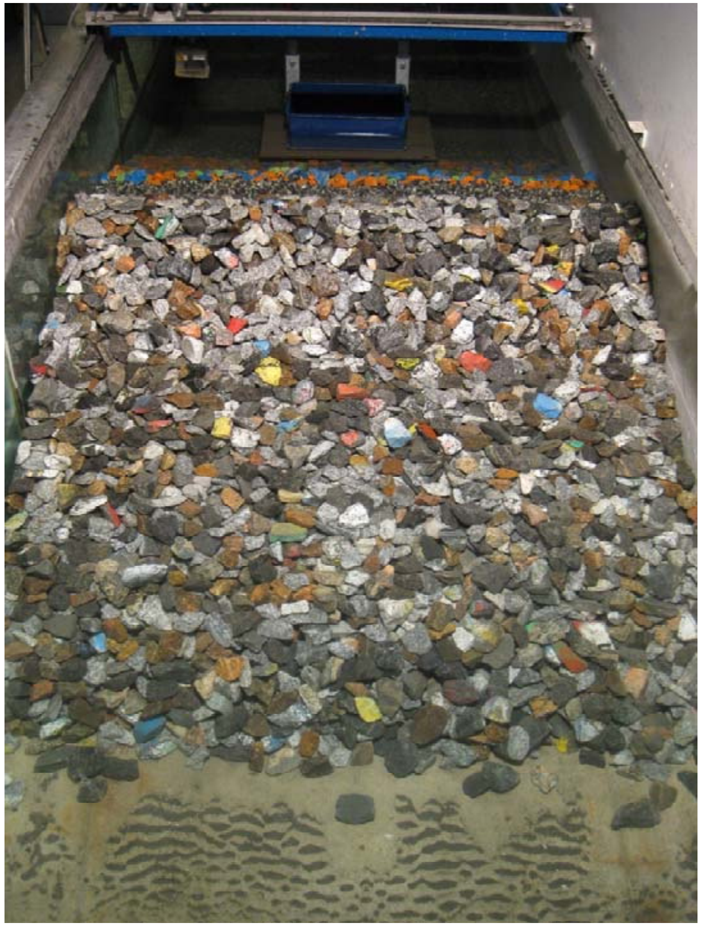

Figure 39: After picture.

Test 15: WL = +2.4 m, H_{m0} = 4.6 m, T_p = 11.0 s, N = 3000

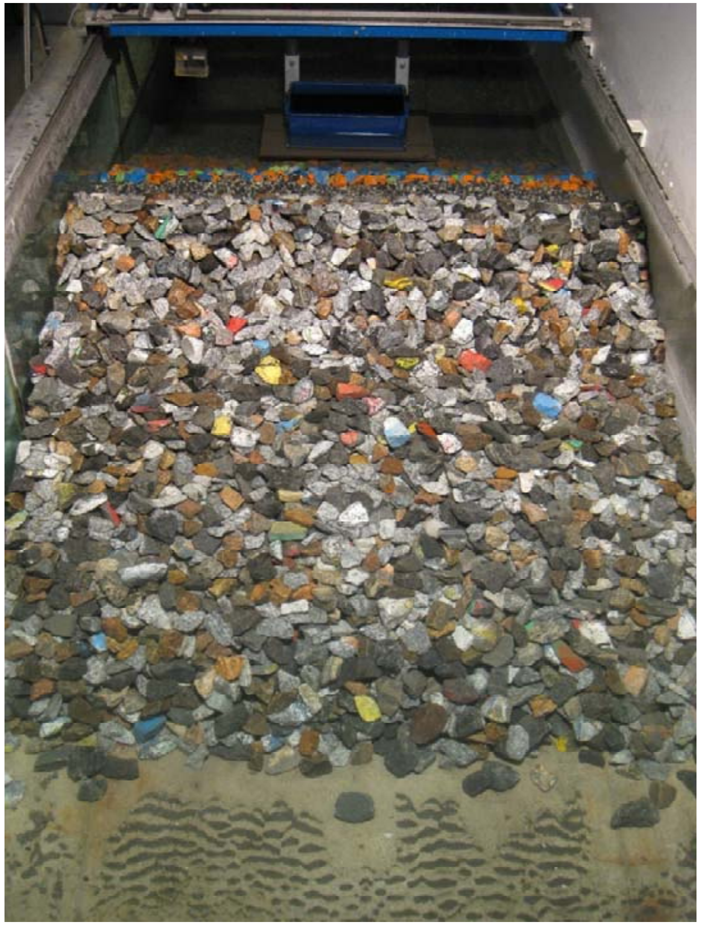

Figure 40: Before picture.

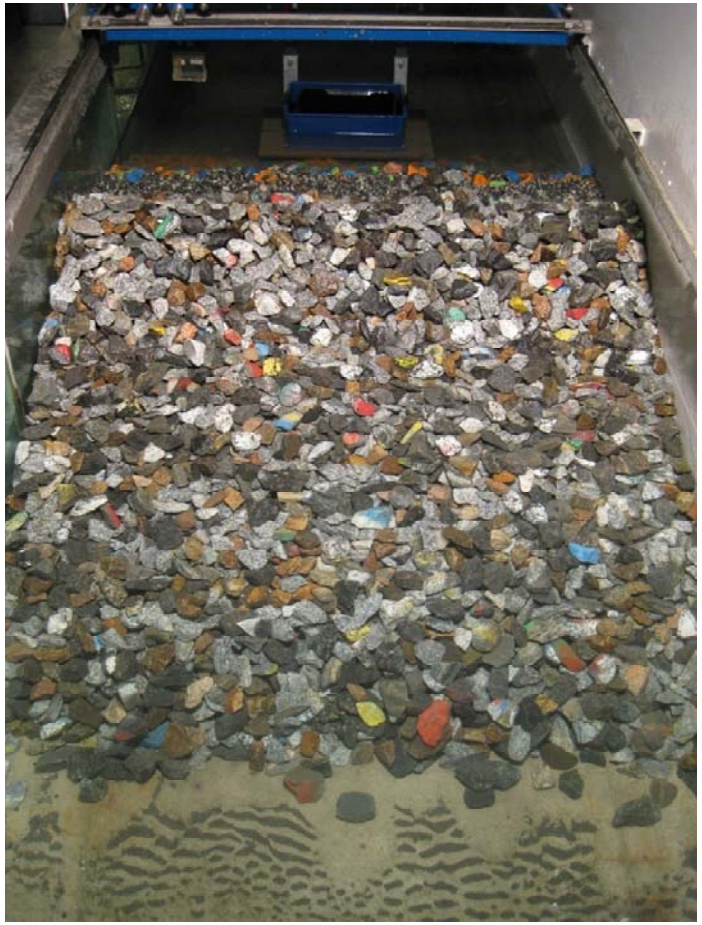

Figure 41: After picture.

Appendix B: Profiling structures

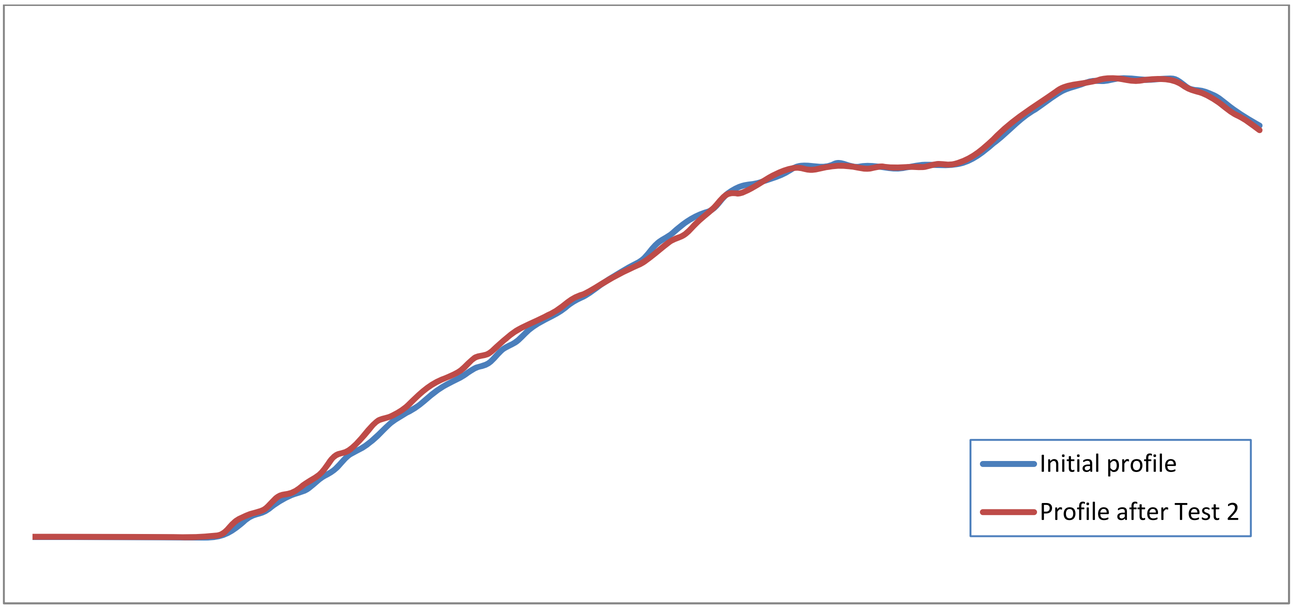

Figure 42: Profile after test 2 (H_{m0} = 3.17 m, T_P = 8 s and WL = +0.2 m).

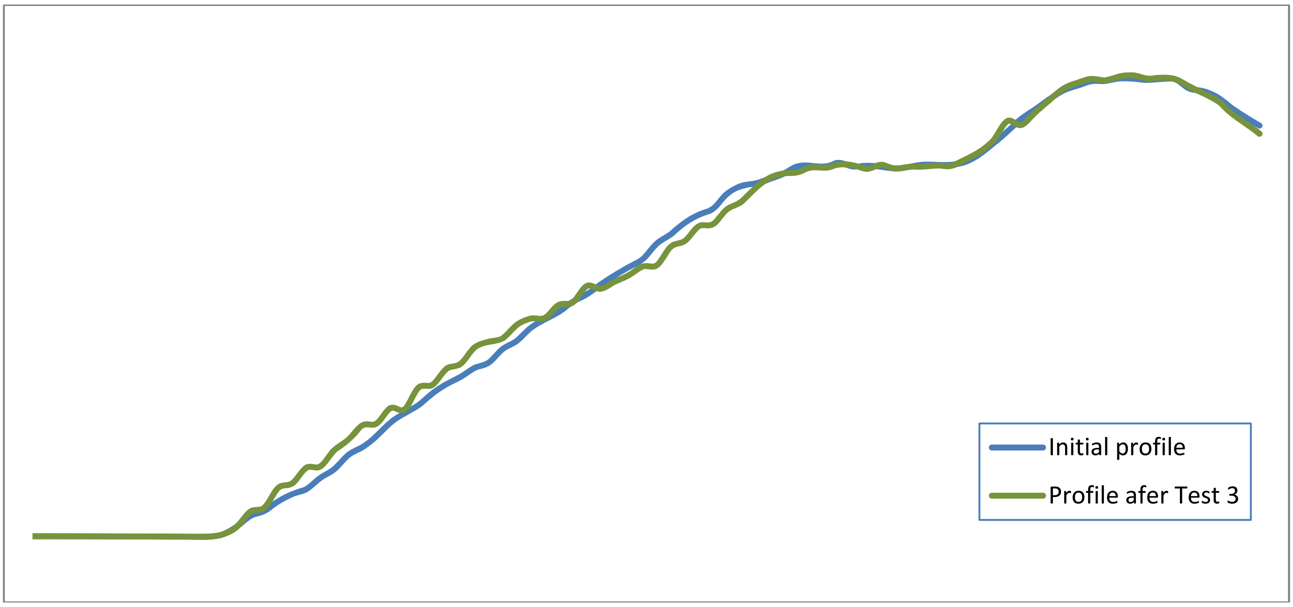

Figure 43: Profile after test 3 (H_{m0} = 4.05 m, T_P = 8 s and WL = +0.2 m).

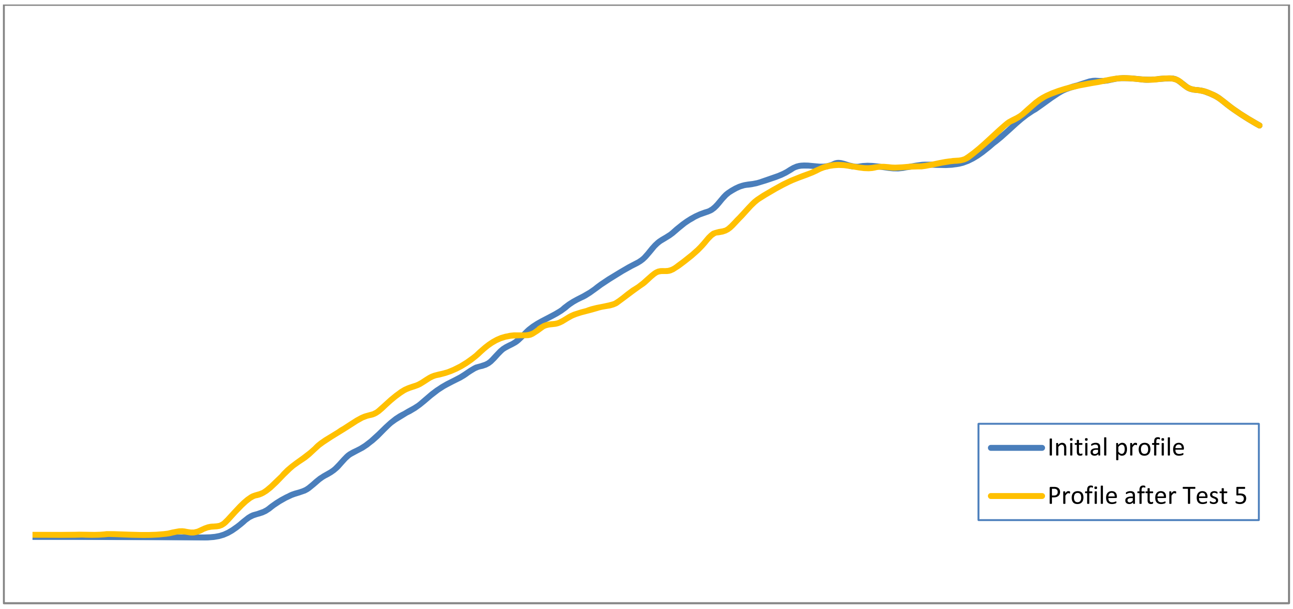

Figure 44: Profile after test 5 (H_{m0} = 4.65 m, T_P = 11 s and WL = +0.2).

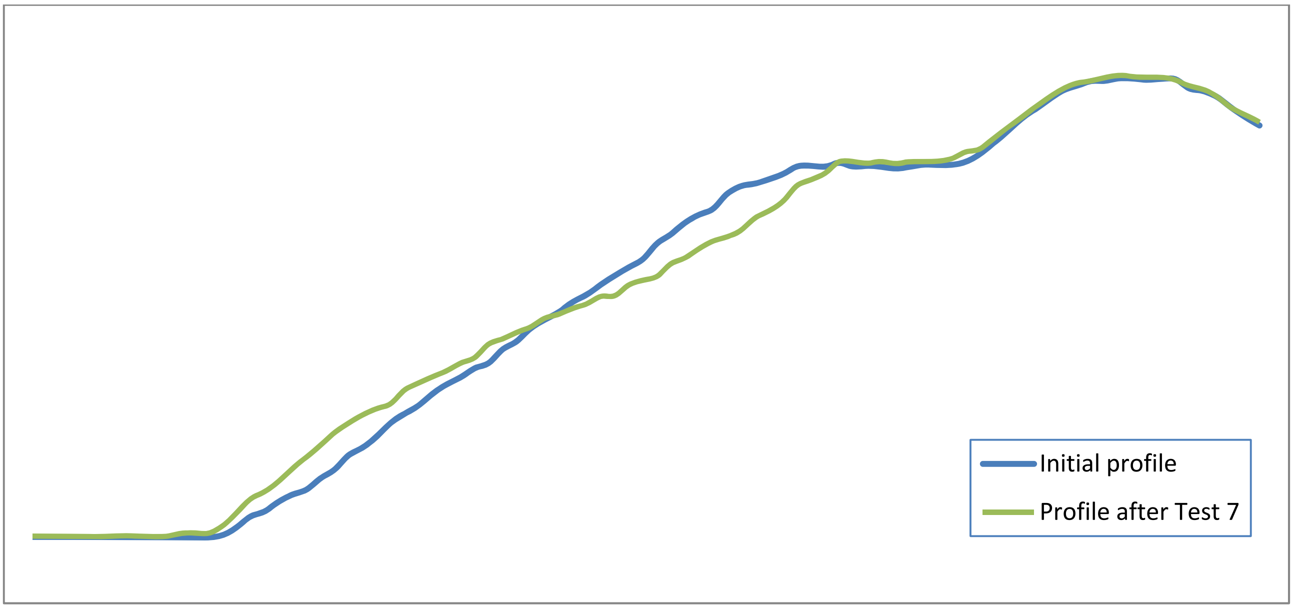

Figure 45: Profile after test 7 (H_{m0} = 3.56 m, T_P = 8 s and WL = +2.4).

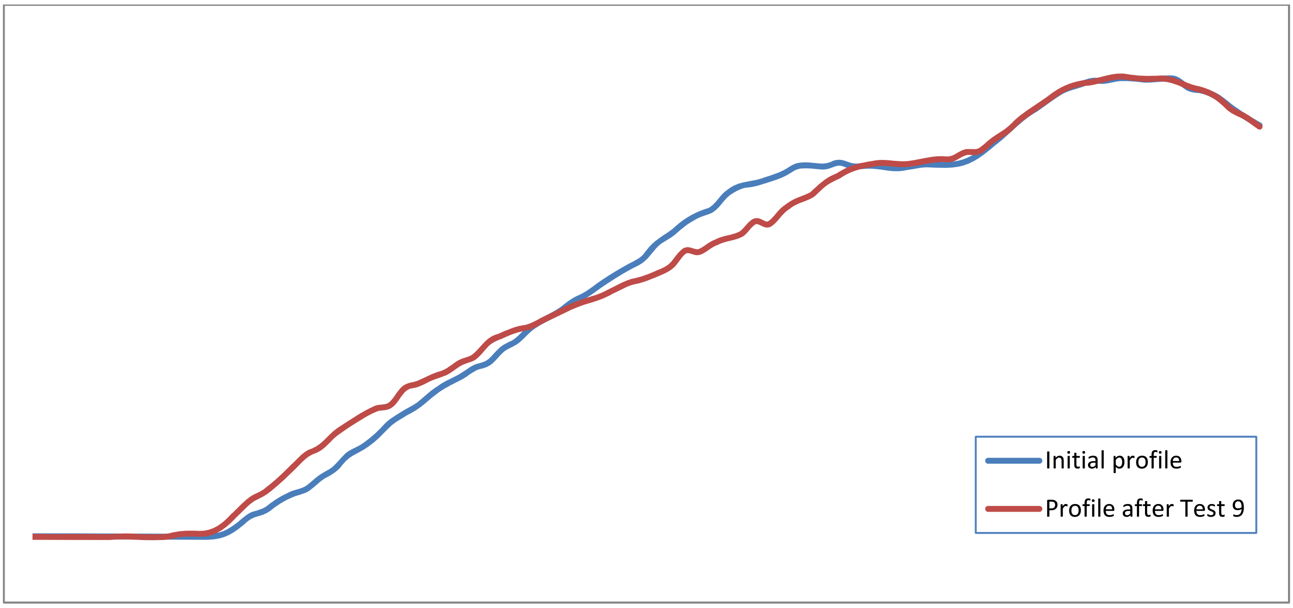

Figure 46: Profile after test 9 (H_{m0} = 4.35 m T_P = 8 s, WL = +2.4).

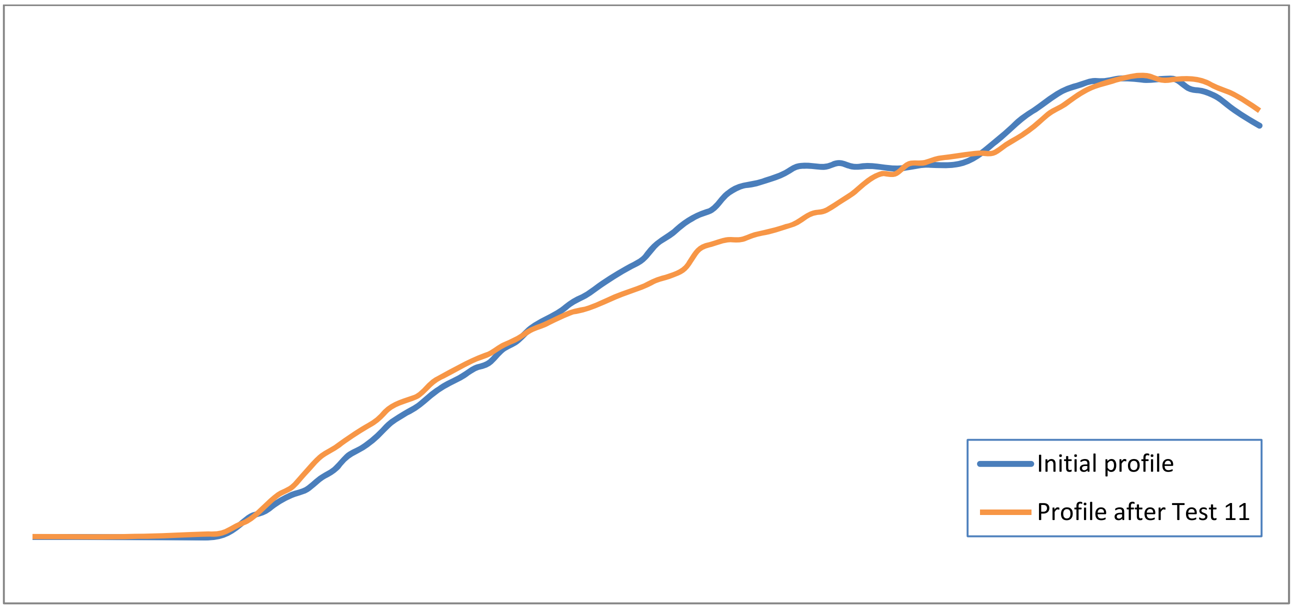

Figure 47: Profile after test 11 (H_{m0} = 5.18 m, T_P = 8 s and WL = +2.4).

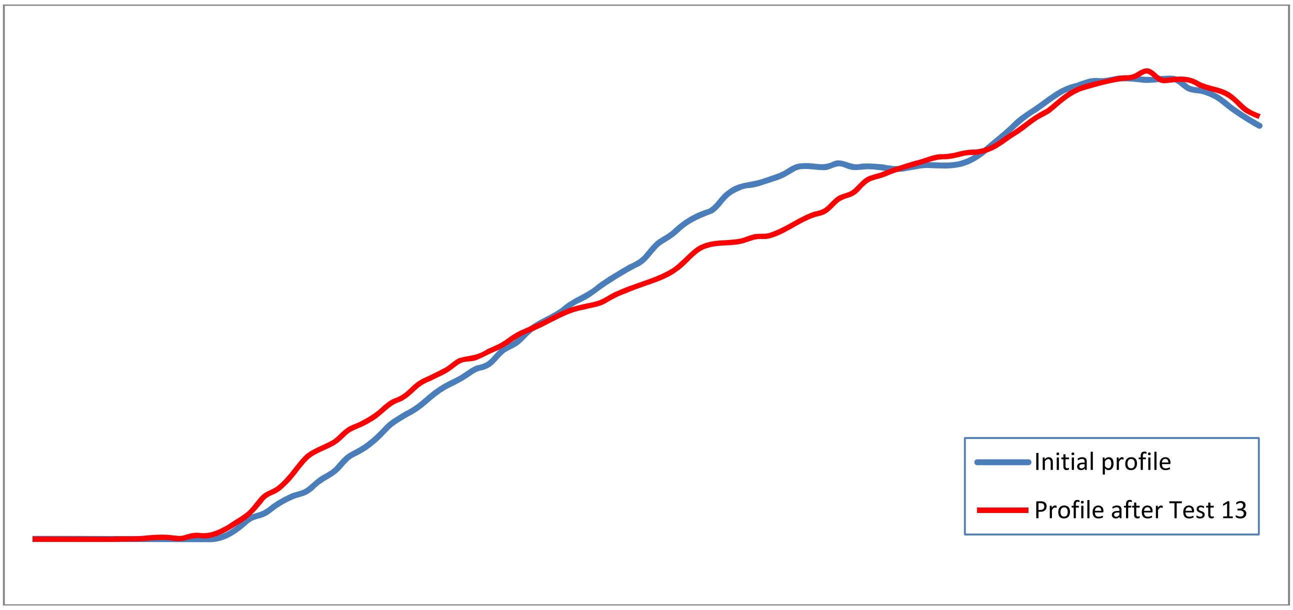

Figure 48: Profile after test 13 (H_{m0} = 3.58 m, T_P = 11 s and WL = +2.4).

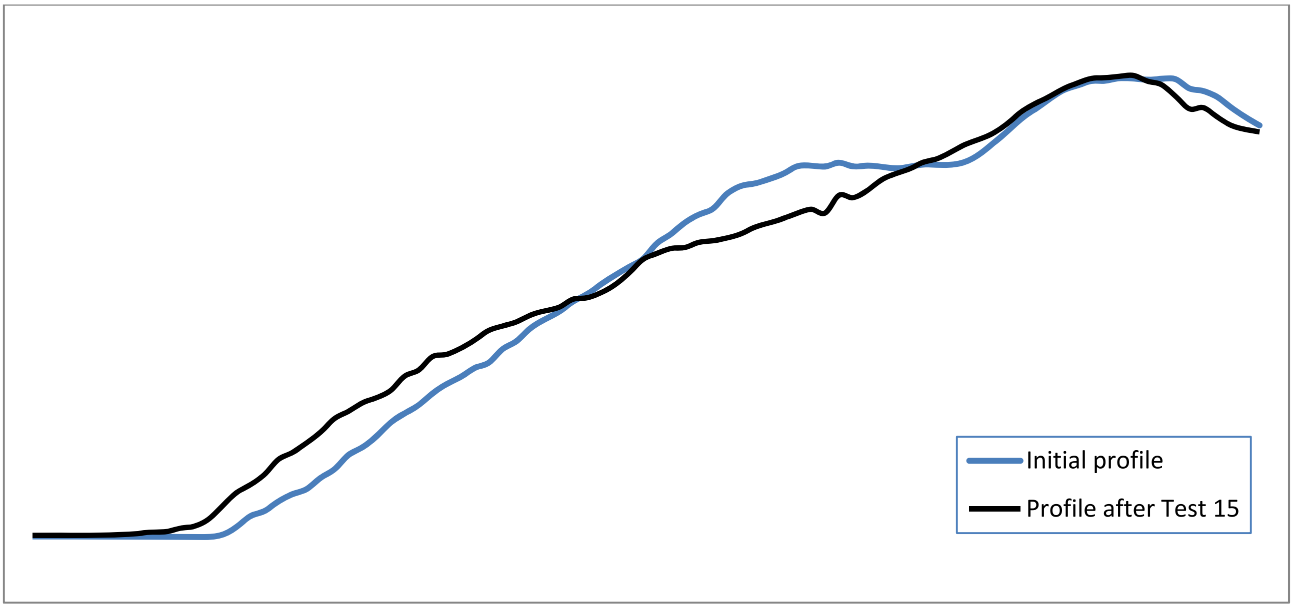

Figure 49: Profile after test 15 (H_{m0} = 4.61, T_P = 11 s and WL = +2.4).