CHAPTER 157

Large verification tests on rock slope stability

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ABSTRACT

A number of large scale tests on stability of rock slopes and gravel beaches is described and compared with small scale test results. The following topics are treated: the stability of a rock armour layer, the profile formation of a berm breakwater, the profile formation of gravel beaches, including ripple formation, and reflection and overtopping on rock slopes. The general conclusion is that scale effects could not be found.

INTRODUCTION

An extensive research program has been performed on static and dynamic stability of rubble mound revetments, breakwaters and gravel beaches. The first part was based on statically stable rubble mound breakwaters. Based on roughly 300 tests two new practical stability formulae were derived, including the wave period, storm duration, permeability of the structure and a clearly defined damage level.

The second part was concentrated on dynamic stability, i.e. the profile formation of rock slopes and gravel beaches under wave attack. About 150 tests were performed in this stage. The result was a computer program that can predict the profile for various wave conditions, including tides and storm surges.

All tests mentioned above were performed in small scale facilities with waves roughly between ten and twenty centimeters. It was stated at the beginning of the research program that the results derived in the small scale facilities should be verified on a larger scale in the Delta flume. These verification tests on scale effects are the subject of this paper.

For results on static stability one is referred to Van der Meer (1987a) and (1988a). Results on dynamic stability were presented by Van der Meer and Pilarczyk (1986) and Van der Meer (1987b). The complete research including set-up, analysis and data of tests was presented by Van der Meer (1988b).

MODEL FACILITY

The Delta flume has a length of 230 m, a width of 5 m and a depth of 7 m. The maximum significant wave height that can be generated is nearly two meters. The random wave generator was equiped with a system that measured and compensated for reflected waves from the structure. With this system standing waves and basin resonance were avoided.

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A surface profiler on a carriage was developed for the investigation. The profiler for the small scale facility was described in more detail by Van der Meer (1987a). The profiler of the Delta flume was constructed in the same way, but all length dimensions were increased by a factor 5.

RESEARCH TOPICS

Various aspects concerning rock slopes and gravel beaches were investigated in the Delta flume on a large scale. Static stability of rock slopes was investigated together in combination with run-up, run-down and reflection. A berm breakwater can be classified as initially dynamically stable and finally statically stable. A berm breakwater was tested with regard to profile formation and overtopping. Tests on dynamic stability of gravel beaches were divided into verification tests on profile formation and extrapolation tests with very small shingle.

STATIC STABILITY

The 300 tests on static stability of rock armour layers in the small scale facility resulted in two new practical stability formulae, given by:

$$H_s/\Delta D_{n50} * \sqrt{\xi_m} = 6.2\ P^{0.18}(S/\sqrt{N})^{0.2} \qquad (1)$$

for plunging waves ($\xi_m < \xi_m$ (transition)),

$$H_s/\Delta D_{n50} = 1.0\ P^{-0.13}(S/\sqrt{N})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^P \qquad (2)$$

for surging waves ($\xi_m > \xi_m$ (transition)), with:

$$\xi_m\ (\text{transition}) = \left(6.2\ P^{0.31}\ \sqrt{\tan\alpha}\right)^{1/(P+0.5)} \qquad (3)$$

where:

H_s = significant wave height
Δ = relative mass density = $\rho_a/\rho-1$
ρ_a = mass density of rock
ρ = mass density of water
D_{n50} = nominal diameter = $(W_{50}/\rho_a)^{1/3}$
W_{50} = 50% value of the mass distribution curve
ξ_m = surf similarity parameter = $\tan\alpha/\sqrt{s_m}$
α = slope angle
s_m = wave steepness = $2\pi H_s/gT_m^2$
T_m = mean wave period
P = permeability coefficient of structure:
- P = 0.1: impermeable core (lower limit)
- P = 0.4: most multi-layer breakwaters
- P = 0.5: permeable core
- P = 0.6: homogeneous structure (upper limit)

S = damage level = A/D_{n50}^2
S = 2-3: start of damage
S = 5-8: moderate damage
S = 8-15: filter layer visible (two layer system)
A = erosion area of cross-section
N = storm duration in number of waves

It was shown in the small scale tests that permeability of the structure had a large influence on stability. And especially the flow characteristics in a small scale model might be due to scale effects. Therefore tests in the Delta flume were concentrated on two different structures: a structure with a permeable core (P = 0.5) and a structure with an impermeable core (P = 0.1).

Scale effects in small scale tests on armour stability were discussed by various researchers. The effect of the Reynolds number on stability was investigated by Dai and Kamel (1969), Thomsen et al. (1972), Broderick and Ahrens (1982), Jensen and Klinting (1983), Sörensen and Jensen (1985), Shimada et al. (1986) and Burcharth and Frigaard (1987).
Although results are not througout consistent, lowest values for which no scale effects will be present are often set at $Re = \sqrt{gH_s}D_{n50}/\nu = 1.10^4 - 4.10^4$, with ν = kinematic viscosity. The range of Reynolds numbers used in the small scale tests was about $4.10^4 - 8.10^4$. Thomsen et al. (1972) found no scale effects for $Re > 2.10^5$. Shimada et al. (1986) suggest a value of $Re > 4.10^5$. The results of Thomsen et al. and Shimada et al. were both obtained in large wave flumes with monochromatic wave attack.

Eleven tests of the small scale series were repeated in the Delta flume and were scaled up according to Froude's law by a linear factor of 6.25. The stones had an average mass of W_{50} = 26.5 kg, a nominal diameter of D_{n50} = 0.214 m, a mass density of 2,700 kg/m³ and a grading of D_{85}/D_{15} = 1.38. The wave period was T_m = 4.4 s in all tests, the wave heights ranged from 0.7 - 1.2 m. The slope angle was 1 : 3. In total six test were performed on a permeable structure and five tests on an impermeable structure. The armour layer was rebuilt after each test which means that the data points in a wave height versus damage plot were independent.

The average of nine parallel profiles gives a clear picture of the structure and the damage. Figure 1 shows the profiles of the permeable structure and Figure 2 those of an impermeable structure, where the thin filter layer was placed on a concrete underlayer.

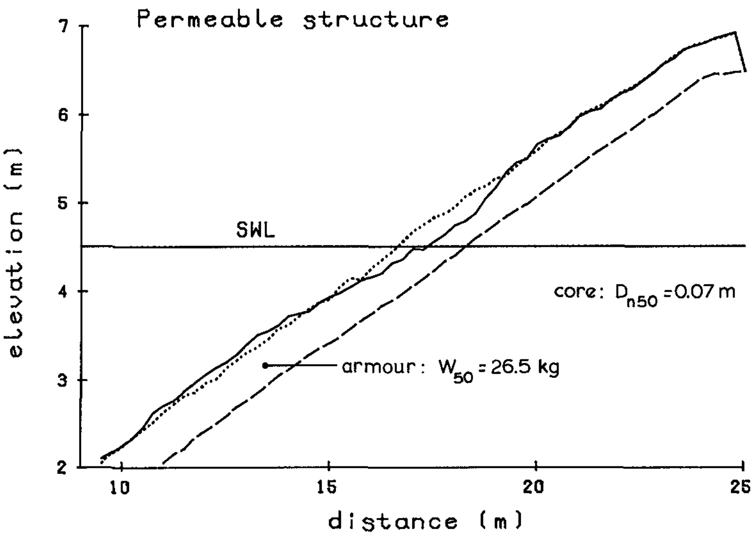


Figure 1 Cross-section with permeable structure in Delta flume

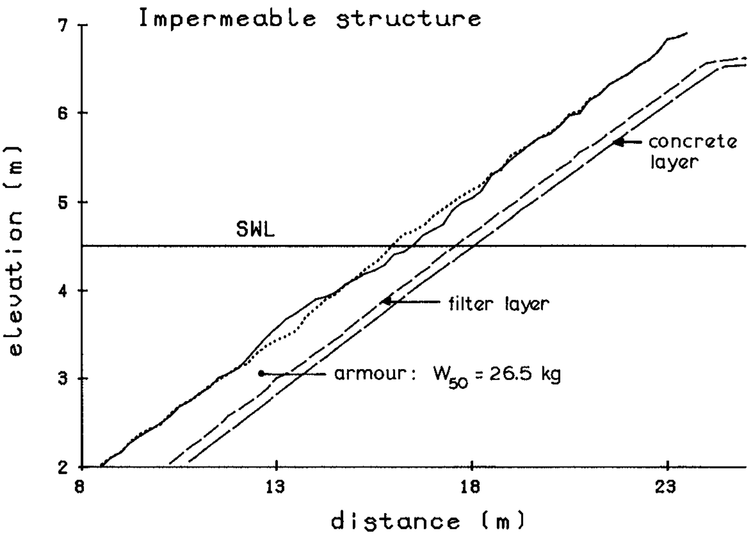


Figure 2 Cross-section with impermeable structure in Delta flume

Results of small and large scale tests can directly be compared in a dimensionless damage curve, where $H_s/\Delta D_{n50}$ is plotted versus the damage S. Figure 3 gives the results of the permeable core and Figure 4 the results of the impermeable core. Besides the different data points of the small and large scale tests, stability formula (1) was plotted in the figures (the curved line).

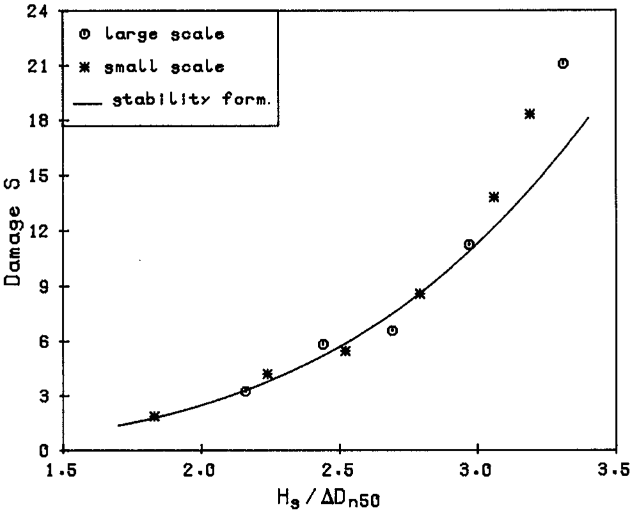


Figure 3 Results of permeable structure

From both Figures it can be concluded that the results of small and large scale tests are in close agreement. This confirms the validity of stability formulae (1) and (2). The stability curve fits well with the data, although some difference is found in Figure 3 for extreme damage levels, S > 12 (filter exposed).

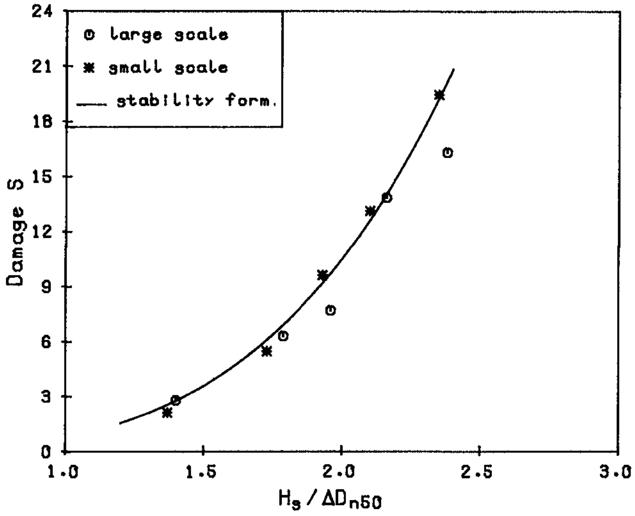


Figure 4 Results of impermeable structure

Comparison of reflection coefficients for small and large scale tests is shown in Figure 5 and coefficients are plotted versus the $H_s/\Delta D_{n50}$. The impermeable structure gives higher reflection coefficients in both cases than the permeable structure, where more energy is dissipated into the structure. The reflection for the impermeable structure is a little higher in the large scale tests. It is almost exactly the same for the permeable structure, the lower data points. In general the agreement is fair.

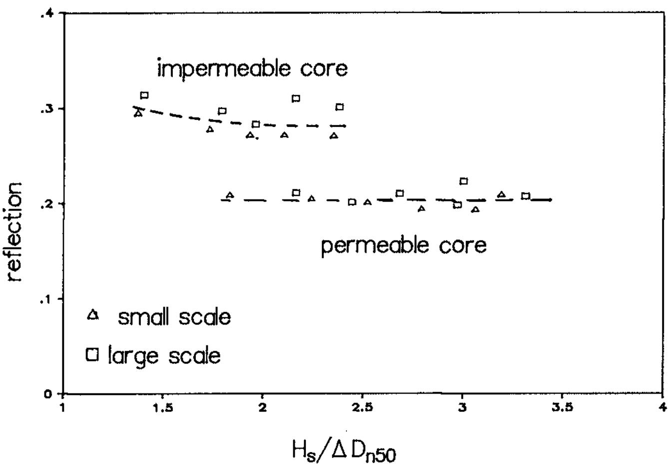


Figure 5 Reflection on rock slopes

The run-up was measured with a capacitance wire. Analysis showed, however, that the exceedance curves of the run-up gave a shift or nod about one meter above the still water level. Probably the wire was damaged at that point by a rolling stone. Therefore, run-up could not be compared with small scale tests.

The final conclusion of the large scale tests on static stability can be stated as follows: large scale model tests confirmed the validity of the small scale tests. The stability of an armour layer of rock was not influenced by the Reynolds number when Re was between 4.10^4 and 7.10^5. As these figures give the whole range of testing, the value of Re = 4.10^4 can only be regarded as an upper boundary for which scale effects on rock armour stability might start.

BERM BREAKWATER

The design concept of the berm breakwater is clearly presented by Baird and Hall (1984). A berm breakwater is a structure that behaves dynamically stable under the first storms and is statically stable further on. The berm breakwater of St. George in Alaska was extensively tested (Delft Hydraulics (1985)). The actual design was performed in a three-dimesional basin on a scale of 1 : 35. Tests on scale effects were performed before the design stage started. One test was performed in a flume on a scale of 1 : 35 and repeated in the Delta flume on a scale of 1 : 7.

The berm breakwater consisted of 2 - 10 tons rock (0.046 - 0.233 kg in the small scale wave flume and 5.8 - 29.2 kg in the Delta flume). The depth limited wave height at the structure was about H_s = 6 m (0.17 and 0.86 m, respectively). The deep water wave height was up to 11 m (0.31 m and 1.57 m). The test consisted of 8 steps with various wave height - wave period combinations, including long swell with peak periods up to 25 s.

Figure 6 gives one of the profiles of the Delta flume test, together with the corresponding profile of the small scale test, but scaled up with a factor 5. The profiles of both tests are very similar. They show the same amount of erosion and subsidence (due to lack of filter layer) at the berm and at the rear of the crest due to overtopping. Even the depth of the scour hole is the same, although shape and length of the scour hole are different.

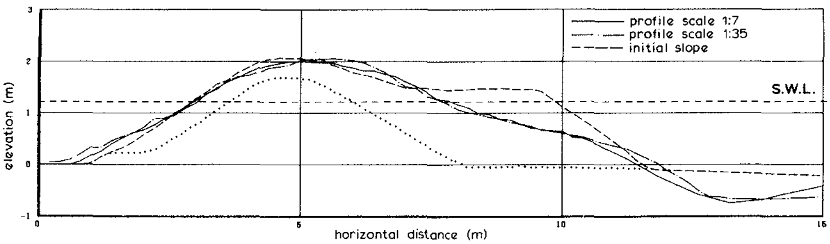


Figure 6 Profiles of berm breakwater in small and large wave flume measures on scale 1 : 7

The volume of erosion at the berm is given in Figure 7 versus the various steps of the test. The agreement between small and large scale test is very good up to step 5. In steps 6 to 8 the small scale test gives a little smaller amount of erosion.

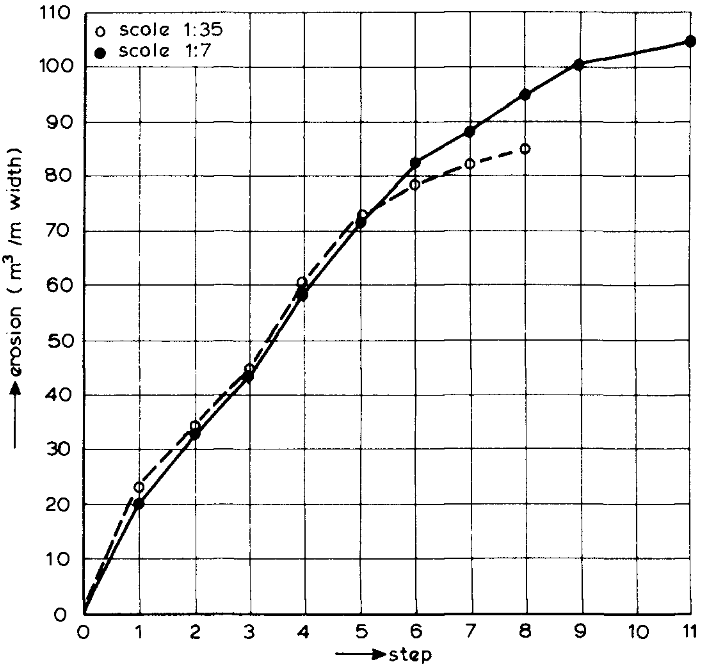
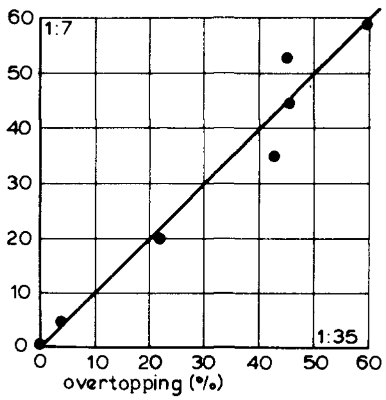


Figure 7 Erosion at berm developed during test

The comparison of overtopping in the berm breakwater tests (in prototype measures) is shown in Figure 8. The left plot shows the percentage of waves that reached the crest of the structure. The right plot gives the significant wave height behind the structure, generated by overtopping. The vertical axis gives the values of the Delta flume test and the horizontal axis the values of the small scale facility. The agreement of results is fair. Only the wave height behind the breakwater is consequently a little larger in the Delta flume.



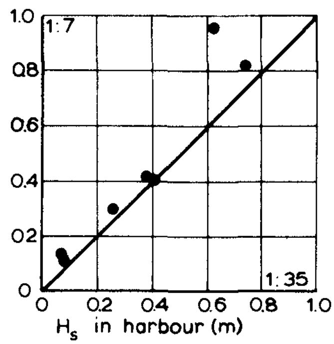


Figure 8 Results on overtopping

The complete analysis of the profiles, erosion and overtopping showed that no scale effects on stability were present in the small scale tests and it was proven that a scale of 1 : 35 for the three dimensional investigation would give reliable results.

DYNAMIC STABILITY

Dynamically stable structures are characterized by the forming of a profile under wave attack. In this case damage is not important, but the developed profile. Rock slopes can be classified as dynamically stable if the $H_s/\Delta D_{n50}$ value exceeds 3 - 4. Gravel or shingle beaches are described by $H_s/\Delta D_{n50}$ values in the order of 20 - 500.

Two topics were evaluated in the Delta flume. First the verification of some tests on gravel beaches, performed by Van Hijum and Pilarczyk (1982). Secondly the behaviour of very small shingle.

Two tests on gravel beaches were repeated on a 4.6 times larger scale. The small scale tests were presented by Van Hijum and Pilarczyk (1982 - tests 11 and 12). The diameter of the shingle was D_{n50} = 0.0187 m and the gradation D_{85}/D_{15} = 1.64. The initial slope was 1 : 5. The wave heights were respectively H_s = 0.77 and 1.00 m and the wave period was T_m = 4.3 s in both tests. This resulted in $H_s/\Delta D_{n50}$ values of respectively 26 and 33. The wave boundary conditions were not exactly the same in the small and large scale facility. Therefore it is very difficult to draw conclusions from a direct comparison of profiles. In stead of that the characteristic points of the profile, defined as dimensionless parameters by Van der Meer and Pilarczyk (1986), were compared.

Two of those dimensionless parameters are shown in Figures 9 and 10. The dimensionless crest height and step height (the point below the water level where the gentle upper slope changes into a steeper slope) are defined by respectively:

$h_c/D_{n50}N^{0.15}$ and $h_s/D_{n50}N^{0.07}$

The vertical axis of Figures 9 and 10 is the combined wave height - wave period parameter H_oT_o, defined by:

$$H_oT_o = H_s/\Delta D_{n50} * \sqrt{g/D_{n50}}\, T_m \qquad (4)$$

where:
$H_o = H_s/\Delta D_{n50}$ = dimensionless wave height parameter
$T_o = \sqrt{g/D_{n50}}\, T_m$ = dimensionless wave period parameter related to D_{n50}

Figures 9 and 10 show all test results for an initial slope of 1:5 for a range of H_oT_o = 100 - 4000. The two tests in small and large wave facility have different symbols. The vertical difference in the Figures gives the difference in wave boundary conditions, the horizontal difference is important for comparison. Figure 9 shows a good agreement of test results and established relationship (the curve). Figure 10 shows that most of the large scale tests have a little smaller value for h_s. But within the variation of the results it is acceptable. From the analysis of all profile parameters it followed that no scale effects could be found.

Finally the discussion of the tests on very small shingle. The shingle had a mean diameter of 4 mm which is almost at the transition to sand.

Such small diameters are difficult to scale for small scale tests as scale effects will definitely be present. Light weight material might be a solution, but will probably give scale effects above the water level (where the crest is formed) resulting in a too steep crest. Only almost prototype tests can give reliable information. The wave heights ranged from H_s = 0.7 to 1.7 m and the wave periods from T_m = 2.5 - 6 s. This resulted in $H_s/\Delta D_{n50}$ values of 90 - 260.

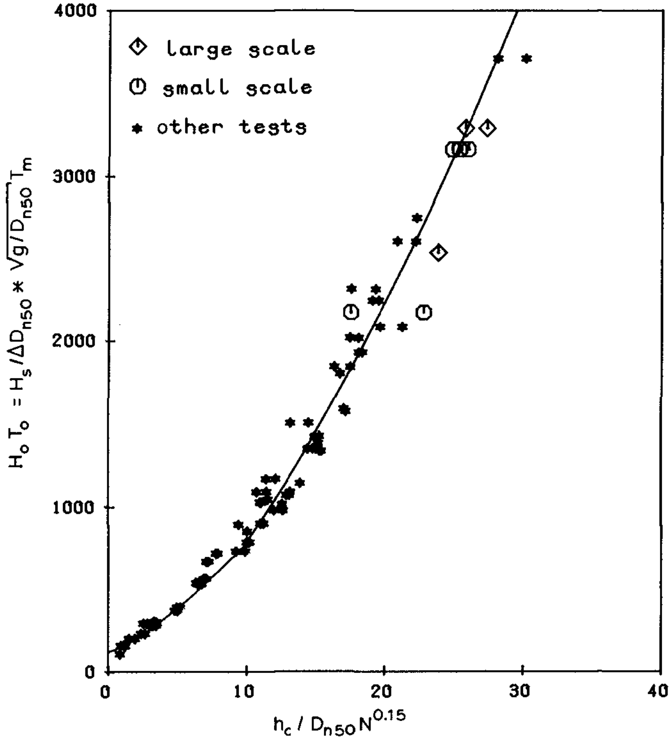


Figure 9 Evaluation of scale effects on crest height h_c

In total six tests were performed with this small shingle. The profiles were analyzed and used to develop a computer program which describes the profile formation of dynamically stable structures. Results were presented by Van der Meer (1986) and Van der Meer (1988b).

One phenomenon was never reported for shingle beach testing, although it is common for sand beaches. Namely the formation of ripples at the lower part of the slope, below the step. The authors are not aware of prototype measurements where ripple formation was reported. It will be difficult, however, to find ripples in prototype. First of all the sounding interval should be small enough to detect these ripples. Secondly, ripples are only formed for small shingle under high waves (storm conditions). They will be formed under the peak of the storm and probably with high water levels. The last part of the storm with lower water levels is able to flatten out the ripples formed before. And prototype profile measurements during the peak of the storm are rare.

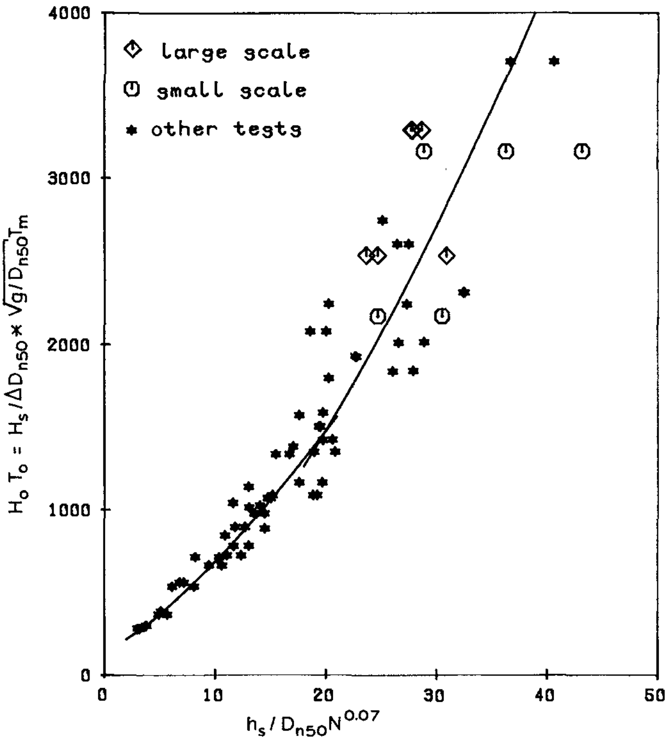


Figure 10 Evaluation of scale effects on step height h_s

The ripple phemomenon is very clearly shown in Figure 11 where 3 parallel profiles were plotted. The ripple height ranged between 10 and 40 centimeters and the ripple length ranged between 1 and 3 meters. As ripple formation has been studied for sand bottoms and slopes under wave attack and under flow conditions, it is interesting to compare the results for sand with very small shingle.

The best fit of the data was found with the results of Nielsen (1981). Nielsen defined the mobility number, ψ, which is a function of the water velocity at the bottom, u, calculated by linear wave theory:

$$\psi = u^2/\Delta g D \qquad (5)$$

Ripple crest height, η, and ripple crest length, λ, were related to the wave amplitude, a = 0.5 H. Nielsen gave the following relationships between crest height and length and the mobility number ψ.

ripple height:

$$\eta/a = 0.275 - 0.22\sqrt{\psi} \qquad (6)$$

ripple length:

$$\lambda/a = 2.2 - 0.345\psi^{0.34} \qquad (7)$$

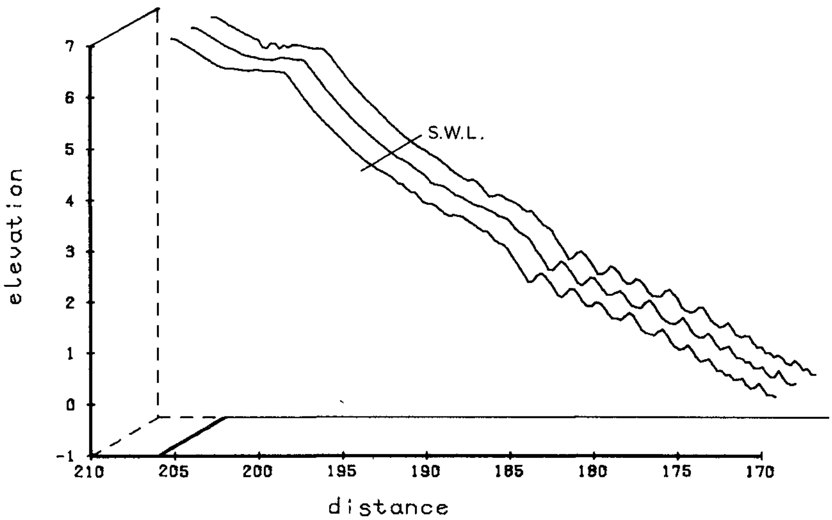


Figure 11 Ripple formation on 4 mm shingle

Equations 6 and 7 are shown in Figures 12 and 13 together with the results on shingle beaches. The general agreement is fair, although quite a lot of scatter is present. From the tests a boundary could be established for which ripple formation might start. This boundary is given by a $H_s/\Delta D_{n50}$ value in the order of 80 - 90 or a mobility number in the order or ψ = 10.

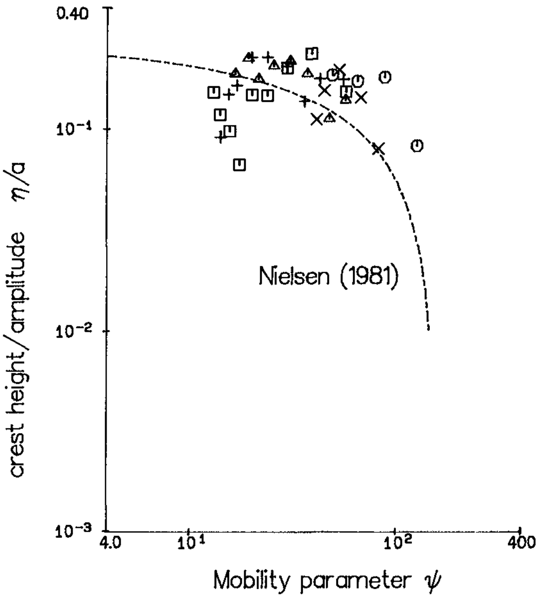


Figure 12 Ripple crest height of shingle versus mobility parameter ψ

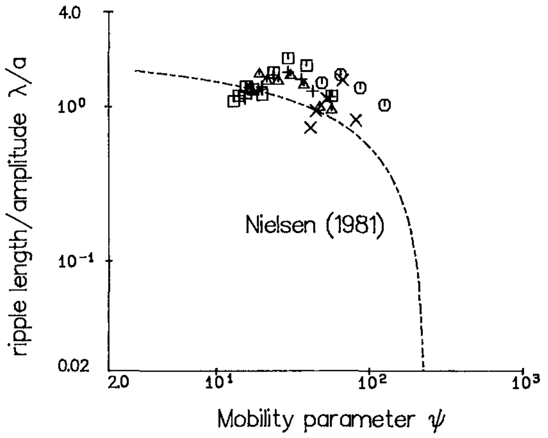


Figure 13 Ripple crest length of shingle beaches versus mobility parameter ψ

CONCLUSIONS

The conclusions derived from large scale testing of rock slopes and gravel beaches can be summarized as follows:

Small scale testing with wave heights higher than about 8 centimeters and for static stability with Reynolds numbers higher than 4.10^4, did not show scale effects for:
- rock slope stability
- berm breakwater stability
- reflection and overtopping
- profile formation.

A Reynolds number of 4.10^4 does not mean that lower values are not possible. Lower values were simply not used in the tests.

Finally, ripple formation could be described by sand ripple theory and a boundary could be established where ripple formation might start.

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