APPENDIX B

Technical Note by
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SINTEF has carried out work along two avenues: 1. Measurements of wave forces on single armour units and 2. Test on a berm breakwater head in shallow water. This memo summarizes and give the preliminary results of the work.

1 Wave forces on single armour units.

1.1 Test set-up.

This work was carried out as a student thesis work by Kjetil Westeren, supervised by Alf Tørum. Westeren (1995).

We used the same test set-up as used by Tørum (1994) for work carried out in MAST I. The tests were carried out in a wave flume as shown in Figure 1 on a berm breakwater model as shown in Figure 2. The weight distribution of the stones in the berm is shown in Figure 3. The specific mass of the stone is \( \rho_s = 2700 \text{ kg/m}^3 \). The median nominal diameter is then \( D_{50} = (W_{50} \rho_s)^{0.333} = 0.0344 \text{ m} \).

![Wave paddle](image)

Figure 1. Wave flume.
Figure 2. Berm breakwater model cross section (measurements in cm).

Figure 3. Weight distribution of stones in berm cover layer.

The force transducer and the principles of placing the force transducer in the breakwater model are shown in Figure 4 and Figure 5 respectively. The force transducer was placed after the breakwater had been reshaped.

Figure 4. Force transducer: a) Stone from above; b) Facing flow; c From side.
Figure 5. Force transducer placed in breakwater, principle (measurements in cm).

The "force measuring stone" was placed in four positions A, B, C and D, Figure 6. The position of the top of the stone is shown in Table 1. Origo is at the intersect between the breakwater and the still water line.

Figure 6. Positions of the "force measuring stone".

<table>
<thead>
<tr>
<th>Point</th>
<th>Horizontal axis, cm</th>
<th>Vertical axis, cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-12.75</td>
<td>8.5</td>
</tr>
<tr>
<td>B</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>C</td>
<td>22</td>
<td>-5.5</td>
</tr>
<tr>
<td>D</td>
<td>65</td>
<td>-17</td>
</tr>
</tbody>
</table>

Table 1. Positions of the "force measuring stone".

The force measuring stone was placed at two elevation, except in point A: 1) embedded with the top at the elevation of the top of the neighbouring stones and 2) elevated one diameter with the lowest point at the elevation of the top of the neighbouring stones. This latter position simulates to some degree the position of a "rolling" stone.

The waves were measured in the middle of the flume at four locations, Figure 7: In "deep" water B1, B2, B3 and on the breakwater B4. The waves measured in "deep" water has been used as reference waves.

All tests were carried out with irregular waves and the target spectrum was a JONSWAP spectrum with a \( \gamma \)-value of 3.3. After some introductory tests it was decided to concentrate the tests and analysis on \( T_p = 2.0\) s and \( 2.4\) s with waveheights \( H_s = 0.13\) m and \( 0.18\) m for both
periods. Table 2.

![Diagram of wave gauges](image)

Figure 7. Location of wave gauges.

<table>
<thead>
<tr>
<th>$T_p$, s</th>
<th>$H_p$, m</th>
<th>$H_s$, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>0.13</td>
<td>0.18</td>
</tr>
<tr>
<td>2.4</td>
<td>0.13</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 2. Test matrix.

The values of the parameter $H_p/(\rho_s/\rho_w - 1)D_{w50} = H_s/D_{w50}$ is then as shown in Table 3. $\rho_w$ = specific mass of water = 1000 kg/m$^3$.

<table>
<thead>
<tr>
<th>$H_s$, m</th>
<th>$H_s/D_{w50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.13</td>
<td>2.22</td>
</tr>
<tr>
<td>0.18</td>
<td>3.08</td>
</tr>
</tbody>
</table>

Table 3. $H_s$ and corresponding values of $H_s/D_{w50}$.

The first tests were run with the "force measuring stone" in position D. For these tests a sampling frequency of 20 Hz was used. Thes tests were run for a test period of 500 s. Then we run some introductory tests with the "force measuring stone" in position B. In this position the forces appeared to be to some extent impulse type force (we will revert to the implication of this later). It was then decided to record the forces as well as the waves with a sampling frequency of 500 Hz. However, due to data storage limitations the remaining tests series had then to be run for 180 s only (approximately 100 waves in each test series).

1.2 Analysis and results.

1.2.1 Characteristic time series of wave forces.

Figures 8 and 9 show characteristic time series of the forces in the measuring points for the
elevated and the embedded stone cases respectively. In Figure 10 are shown on an expanded time scale part of the same time series shown in Figure 8.

<table>
<thead>
<tr>
<th>Point B</th>
<th>$T_p = 2.0 \text{ s}, H = 0.18 \text{ m}$</th>
<th>$T_p = 2.4 \text{ s}, H = 0.18 \text{ m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel forces</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
</tr>
<tr>
<td>Normal forces</td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
</tr>
<tr>
<td>Point C</td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
<tr>
<td>Parallel forces</td>
<td><img src="image7" alt="Graph" /></td>
<td><img src="image8" alt="Graph" /></td>
</tr>
<tr>
<td>Normal forces</td>
<td><img src="image9" alt="Graph" /></td>
<td><img src="image10" alt="Graph" /></td>
</tr>
<tr>
<td>Point D</td>
<td><img src="image11" alt="Graph" /></td>
<td><img src="image12" alt="Graph" /></td>
</tr>
<tr>
<td>Parallel forces</td>
<td><img src="image13" alt="Graph" /></td>
<td><img src="image14" alt="Graph" /></td>
</tr>
<tr>
<td>Normal forces</td>
<td><img src="image15" alt="Graph" /></td>
<td><img src="image16" alt="Graph" /></td>
</tr>
</tbody>
</table>

Figure 8. Characteristic forces for the elevated stone case. Horizontal axis: Time in s. Vertical axis: Force in N.
Figure 9. Characteristic forces for the *embedded* stone case. Horizontal axis: Time in s. Vertical axis: Force in N.
Figure 10  Part of time series shown in Figure 9 (expanded time scale).
In Points A and B the forces appeared, as mentioned, to be to some extent impulsive type forces. This raise the question on possible dynamic amplification of the wave force in the measurement system.

In order to investigate this we carried out some pluck tests. Figure 11 shows part of a time series of measured parallel force in Point B (still water level) after the "force measuring stone" had been plucked. The natural frequency of oscillation is approximately 10 Hz. The time series shows also that there is a considerable damping in the measurement system.

![Time series of parallel force in Point B after plucking the "force measuring stone".](image)

If we consider the impulsive forces to be a half sinusoidal pulse force with an amplitude $F_0$, the dynamic amplification factor for a system of one degree freedom is shown in Figure 12 as a function of the ratio between the duration time $t_0$ and the natural period of oscillation $\tau$. The damping has in this case been assumed to be zero.

![Dynamic amplification factor for a system of one degree of freedom. Zero damping.](image)

From the pluck tests, Figure 11, we find the natural period of oscillations to be approximately $\tau = 0.11$ s. From Figure 10 we find that the response duration $t_0$ is approximately 0.15 to 0.20 s. If we assume the response duration to be approximately the same as the force duration, we find $t_0/\tau = 1.5$ to 2.0. According to Figure 12 we find then that there is a slight dynamic amplification in the response data.
1.2.2 Statistical analysis of the wave forces.

The wave force data on the “force measuring stone” have been analysed statistically using the analysis program PROMOT of the time series analysis program STARTIMES developed by SINTEF.

The forces parallel to the breakwater slope were defined as positive when they acted in the direction of wave travel. The forces normal to the breakwater slope was defined positive when they acted in an upward direction.

The force time series have to some extent small irregularities resembling noise. We are mainly interested in the larger peak forces in positive and negative direction. It was therefore necessary to set a lower bound of the force values of interest. This lower bound varied slightly from time series to time series and was to some extent subjectively set such that the number of “valid” peak forces in each time series corresponded approximately to the number of waves in the series. Another restriction set was that the number of data points between each peak value should at least be 250, corresponding to t=0.5 s.

A three parameter Weibull probability model was fitted to the data according to the method of moments. The three parameter cumulative Weibull distribution function is given by:

\[ P(F_{peak}) = 1 - \exp\left(-\frac{F_{peak} - F_0}{F_c}\right)^\gamma \]

where
- \( F_{peak} \) = the considered peak force
- \( F_0 \) = location factor
- \( F_c \) = scaling factor
- \( \gamma \) = shape factor

Figure 13 shows an example of fitting the Weibull three parameter distribution to the data. Figure 14 shows the histogram for the same data.

A Weibull three parameter distribution was fitted to all the force time series. For each fitting the 5% and 99% level forces, F90% and F99%, were taken as characteristic forces. These characteristic forces have been plotted for each stone condition, embedded or elevated, and for each wave condition and for all locations, Points A, B, C and D. These plots are shown in Figures 15 - 18.

The normal forces have been analysed in a similar way as the parallel forces. Figures 19 - 22 shows plots of the 90% and 99% forces in Points A, B, C and D for different wave conditions and for elevated and embedded stone.
Figure 13. Weibull cumulative distribution of the parallel forces in Point D, elevated stone. $T_p = 2.0$ s, $H_s = 0.18$ m.

Figure 14. Histogram of parallel forces in Point D, elevated stone. $T_p = 2.0$ s, $H_s = 0.18$ m.
Figure 16. 99% forces for embedded stone.
Figure 17

90% forces for elevated stone.

Elevated stone, parallel force, 90% force

- Tp=2.0s, Hs=0.13m (uprush)
- Tp=2.0s, Hs=0.18m (uprush)
- Tp=2.4s, Hs=0.13m (uprush)
- Tp=2.4s, Hs=0.18m (uprush)
- Tp=2.0s, Hs=0.13m (dow nn rush)
- Tp=2.0s, Hs=0.18m (dow nn rush)
- Tp=2.4s, Hs=0.13m (dow nn rush)
- Tp=2.4s, Hs=0.18m (dow nn rush)

Horizontal length from SW, [cm]
Figure 18. 99% forces for elevated stone.
Figure 19  90% normal force for embedded stone.
Figure 20.
99% normal force for embedded stone.
Figure 21  90% normal force for elevated stone.
Figure 22
99% normal force for elevated stone.
Although there are some sources of error in the measurements, which we will discuss at a later stage, it can be concluded that the wave forces on the stones are larger the higher on the slope the stone is located. The general trend is also that the forces have a more impulsive character the higher on the slope the stone is located. It seems that the mechanism for reshaping of a berm breakwater is that the stones of the upper part is “knocked” loose and then roll upwards and subsequently downwards in and “elevated” position.

2 Stability tests on a berm breakwater head in shallow water.

The stability tests on a berm breakwater head in shallow water are supplementary tests to the stability tests in deep water carried out at DHI. The SINTEF shallow water test set-up is shown in Figure 23. Two breakwater heads were tested:

1) A berm breakwater head. Breakwater head A “DHI”, Figure 24, that was almost a replica of the DHI tested berm breakwater head, except that the water depth was 0.25 m instead of 0.55 m during the DHI tests.

2) A breakwater head, Breakwater head B “Nordic type”, Figure 25, with the same water depth, cover stone and core material as in case 1), but with a narrower berm, with a lower crest and with some core material in the shoulder.

Figure 23. Stability tests on berm breakwater head in shallow water. Test set-up.
Figure 23. Breakwater head A "DHI". The same as for DHI tests, but with shallower water.

Figure 24. Breakwater head B. "Nordic type".
The nominal diameter distribution for 5 samples taken at different locations on the breakwater head is shown in Figure 25.

Figure 26 and Figure 27 shows the relation between the measured significant wave heights and the measured zero-upcrossing periods at water depths 0.70 and 0.25 m respectively. We have used the measured zero-upcrossing periods at water depth 0.70 m when calculating the wave steepness \( s = H_s / (gT_s^2 / 2\pi) \). During the tests the wave steepness was kept almost constant \( s = 0.05 \). Figures 28 shows \( H_{\text{max}} \) vs \( H_s \) and Figure 29 shows the ratio between \( H_{\text{max}} \) and \( H_p \), for wave period \( T_p = 1.77 \) s and for the water depths 0.70 m and 0.25 m. As expected the ratio \( H_{\text{max}} / H_p \) becomes less in shallow water than in deep water because of the heavy breaking of the larger waves in shallow water.

The tests were run for the following steps of the parameter \( H_o = H_s / (\Delta D_{50}) = 2.3 - 2.7 - 3.2 \) 3.8 - 4.0). The last value has some uncertainty because of extrapolation of the wave generator exentriscity setting.

The tests were run in the following way for the Breakwater head A “DHI”:

After building the breakwater model the wave parameter \( H_o \) was increased in the mentioned steps. 2000 waves were run for each step. Profiles were taken with a laser distance measurement system after each step. The distance between each profile was 0.10 m and the distance between each measurement point in a profile was 0.02 m.

After this reshaping process the steps were repeated again, but now with 1000 waves for each step. Since no “damaging” reshaping had occurred on the breakwater at the end of the planned test series, we continued to run 10,000 waves with \( H_o = 4.0 \). Figures 30 and 31 show oblique views of the of the Breakwater head A “DHI” before the tests started and after the completion of the tests (after 10,000 waves with \( H_o = 4.0 \)). Figure 32 shows one of the profiles before the tests started and after the completion of the tests.

A striking feature is that the in shallow water the reshaping in shallow water effects only a part of the berm width while in deep water (DHI tests) the whole width of the berm was effected. If we go back to the results of the force measurement results we see that the highest forces on an individual stone are in the force measurement point above the still water level. It seems the mechanism for reshaping is that the stones in the area above the still water level are “knocked loose” and roll down the slope. But since the maximum wave heights in shallow water are less than in deep water we will also expect that the ability to “knock loose” the stones is less in shallow water than in deep water. Hence the reshaping in shallow water stops “earlier” than in deep water.

Based on the findings from the etts on Breakwater head A “DHI” we decided to test a breakwater head with a narrower berm width, Breakwater head B “Nordic type”, Figure 24. At the same time we put some core material into the berm.

For Breakwater head B “Nordic type” the planned test program was the same as for Breakwater head A “DHI type”. However, when running the reshaping tests (2000 waves for each step of \( H_o \)) a slight damage on the rear side due to wave overtopping occurred for \( H_o = 5.8 \). For \( H_o = 4.0 \) the damage on the rear side became so heavy that we decided to terminate this test step after approximately 1600 waves.
Figures 33 and 34 show oblique views of the Breakwater head B "Nordic type" before the tests and after terminating the tests. It is interesting to note that there was no damaging reshaping at the termination of the tests on the front side and on the breakwater head.

Figure 25. Berm stone size distribution.
Figure 26. Measured zero-upcrossing periods $T_z$, peak periods $T_p$ and significant wave heights $H_s$. Waterdepths $d = 0.70$ m.

Figure 27. Measured zero-upcrossing periods $T_z$, peak periods $T_p$ and significant wave heights $H_s$. Waterdepths $d = 0.25$ m.
Figure 28. $H_{\text{max}}$ vs $H_s$ for wave period $T_p = 1.77$ s and for the water depths 0.70 and 0.25 m.

Figure 29. The ratio between $H_{\text{max}}$ and $H_s$, $H_{\text{max}}/H_s$, for wave period $T_p = 1.77$ s and for the water depths 0.70 and 0.25 m.
Figure 30. Oblique view of Breakwater head A (DHI) before testing.

Figure 31. Oblique view of Breakwater head A (DHI) after testing.
Figure 32. Breakwater head A (DHI): Cross sections at 1200 mm before and after testing.

Figure 33. Breakwater head B "Nordic type" before testing.
Figure 34. Breakwater head B "Nordic type" after terminating the tests. Damage on the rear side due to wave overtopping.

3 Scaling.

The results in the preceding chapters have been given in model values or in dimensionless parameters form. In order to give an indication of full scale values we have listed in Table 4 prototype values of water depth, berm stone mass, maximum significant wave height and corresponding peak period for different scale ratios and assuming Froude scaling. The scale ratio is defined as the ratio between length values in the prototype and corresponding length values in the model.

<table>
<thead>
<tr>
<th>Scale ratio</th>
<th>Water depth at breakwater location m</th>
<th>Berm stone mass (mean) kg</th>
<th>Max. significant wave height $H_s$ at breakwater location m</th>
<th>Peak period $T_p$ in “deep” water s</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>10.0</td>
<td>1750</td>
<td>6.0</td>
<td>11.6</td>
</tr>
<tr>
<td>50</td>
<td>12.5</td>
<td>3400</td>
<td>7.5</td>
<td>12.9</td>
</tr>
<tr>
<td>60</td>
<td>15.0</td>
<td>5800</td>
<td>9.0</td>
<td>14.2</td>
</tr>
</tbody>
</table>

Table 4. Water depth, berm stone weight, significant wave height and peak periods for different scale ratios.
References.
