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Stability Assessment of the New Cubipod® Armoured Breakwater in Hanstholm

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Abstract: The present paper presents results from hydraulic model testing of the outer part of the new western breakwater for the Port of Hanstholm, Denmark. The design waves in Hanstholm are depth limited and breaking on a relatively steep, complex and dynamic foreshore. This causes the waves to shoal and focus on certain sections of the breakwater. The chosen armour layer design was high density Cubipods® in a single layer as both front and rear side armour. On the front side an underlayer of quite large rocks was used in order to secure stability during construction. Thus, the conditions deviate in several ways from previous projects where Cubipods® have been applied, i.e. high density concrete, coarser underlayer, breaking and depth limited waves on a complex bathymetry. For these reasons the design was tested and optimized in three-dimensional hydraulic model tests as presented in the present paper.

Keywords: Cubipod®, Concrete armour units, Armour stability, Depth limited waves, Rear side stability

1 Introduction

The Port of Hanstholm is the most exposed port in Denmark. It is located at a headland at the Danish West Coast and with long fetches towards west and north-westerly directions. The 100 year offshore significant wave heights exceed 8 m. The port is located in an area with significant longshore littoral drift (more than 500,000 m^3 annually in north-east direction per year). The bathymetry is highly dynamic and thus varies in time. The limestone surface is located at approximately level CD -14 m.

The first attempt to construct a port in Hanstholm was with timber caissons and converging breakwaters. Construction of the western breakwater started in 1919, but experienced many problems and the construction pace was very slow with only a few caissons per year. Significant sedimentation occurred as the eastern breakwater was never constructed. More than two decades later the project was abandoned as many of the wooden caissons had failed. In the 1960'ties the port construction was restarted with circular concrete caissons placed directly on the limestone. The layout of the converging breakwaters was further optimized and a narrow opening led to an exemplary example of a bypass port, see Fig 1. Hanstholm is today Denmark's leading port in the market for edible fish.

Taking into account todays large fishing vessels, the harbour entrance is too narrow and shallow, which makes navigation conditions difficult during the rough seas that frequently occur in Hanstholm. This together with the need for more hinterland were the main arguments for proposing a new western breakwater for Port of Hanstholm. Moreover, the water depth in the entrance will be increased from 9 to 11 m to be able to service larger vessels.

The port engaged Rambøll as their consultants for the new port expansion, and based on the above considerations, Rambøll established the layout of the port expansion and prepared tender documents for a design-build project including a large new western breakwater.

Fig. 1. Existing port layout (top right) and new port expansion (illustration by Per Aarsleff A/S).

2 Design Conditions

Offshore design conditions were established by Rambøll based on analysis of buoy data as well as hindcast wave data by numerical modelling. The wave buoy was located at CD -19 m and thus design conditions were given at that depth. Tab. 1 shows the design conditions established by Rambøll for the most critical wave direction (300°).

Tab. 1. Provided offshore (CD –19 m) design conditions for the most critical wave direction

Mean wave direction	300°					
Return period (T_r)	1	10	50	100	Overload 1	Overload 2
H_{m0} [m]	5.2	6.7	7.8	8.2	9.0	9.0
T_p [s]	15.0	16.0	16.5	16.5	17.0	17.0
HWL [m]	1.3	1.5	1.7	1.7	1.7	1.9
LWL [m]	-	-	-	-0.5	-	-

The provided wave periods were considered upper limits, which are most critical for overtopping and rear side stability. As front slope stability is not necessarily most critical for long waves also lower peak periods (12.5 s and 14.5 s) were tested for the 100 year design event. Waves from 330° and 0° (North) were also tested, but design wave conditions from these directions were much milder and never showed to be critical.

Due to the large variation in design wave height with direction, it was decided to verify the hydraulic stability for a mean wave direction of 310 degrees with the design wave parameters from 300 degrees, but this did not lead to significant differences in the results.

3 Desk design

After several negotiation rounds, Per Aarsleff A/S with COWI A/S as design consultants, got the design-build contract for the port expansion. After considering several alternatives, they decided to use high density Cubipod® armour units in a single layer for the design, with a K_D of around 10 for the trunk section based on e.g. (Jensen, 2013) used in the initial desk design. The Cubipod® is a bulky unit developed and patented by Universidad Politecnica of Valencia and licensed by SATO-OHL, see Fig. 2. The Cubipod® has previously been used in single layer for example in Punta Langosteira (Corredor et al., 2014).

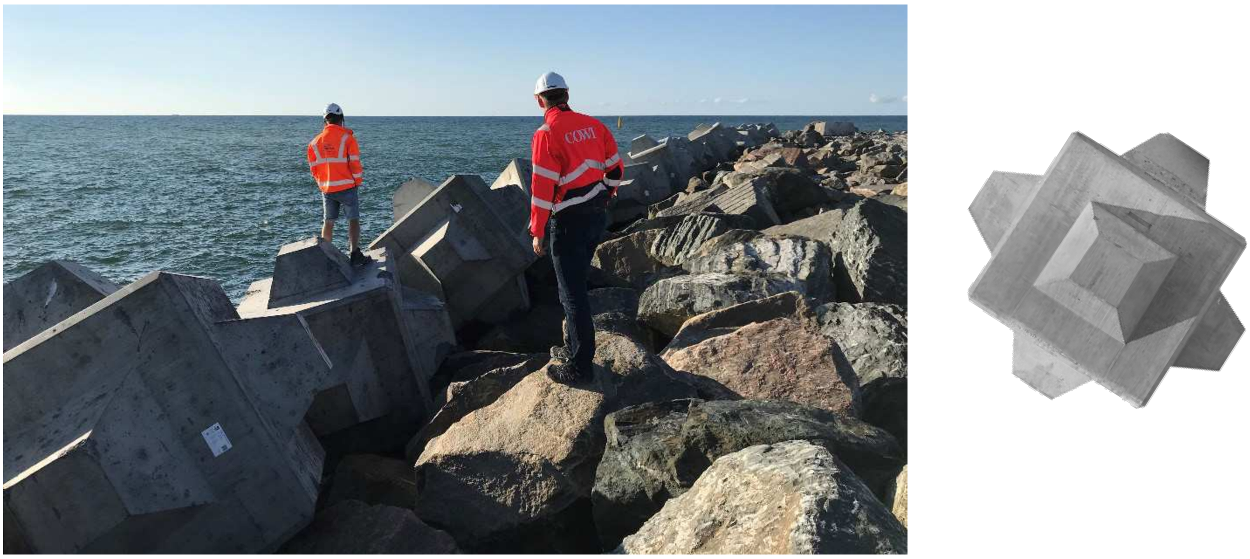

Fig. 2. Cubipod® armour unit placed on the new breakwater in Hanstholm (summer 2018).

COWI proposed a front slope of 1:1.75 with units of high density concrete (ρ = 2,700 kg/m3) placed on a quite coarse underlayer of 3-8 t rocks. The coarse underlayer was required in order to have sufficient hydraulic stability during construction, where the filter layer could be exposed to direct wave attack during temporal stages before Cubipod® armour placement. Calculations by the Van der Meer (1988) stability formulae showed that the rocks could resist significant wave heights of Hs = 3-4 m (corresponding to a typical summer storm) giving damage levels of Sd = 2-6. Moreover, the large rocks in the underlayer increase the permeability and thus the stability of the Cubipod® front armour layer. Accurate placement of concrete armour units generally require a quite smooth underlayer. However, the Cubipod® units are less sensitive to underlayer smoothness than other more interlocking monolayer units like Accropodes and X-blocks, and the possibility to use larger filter rock as well as a more gentle slope was some of the main reasons for choosing Cubipods® as main armour.

The Cubipod® manual (Medina and Gómez-Martin, 2016) recommends K_D = 12 for a trunk section with Cubipods® in a single layer. This was based on tests with non-breaking waves and non-overtopped 1:1.5 slopes with units of normal density concrete. Thus, the design for Port of Hanstholm is in several ways different from the basis for the Cubipod® manual as it both feature a coarser underlayer, milder slope, high density concrete, breaking waves on steep and complex foreshore and highly overtopped structure during design conditions.

As part of the tender a 1:30 bathymetry was specified. The tender design was validated on such bathymetry in 2-D and 3-D model tests performed in Aalborg University. However, during the initial construction phase a bathymetry with steeper and deeper sections and also with strong 3-D features were observed. Based on that it was clear that the previously tested 1:30 bathymetry might be unsafe along certain parts of the breakwater. Therefore, the tender design needed to be reinforced and tested again with the updated design bathymetry.

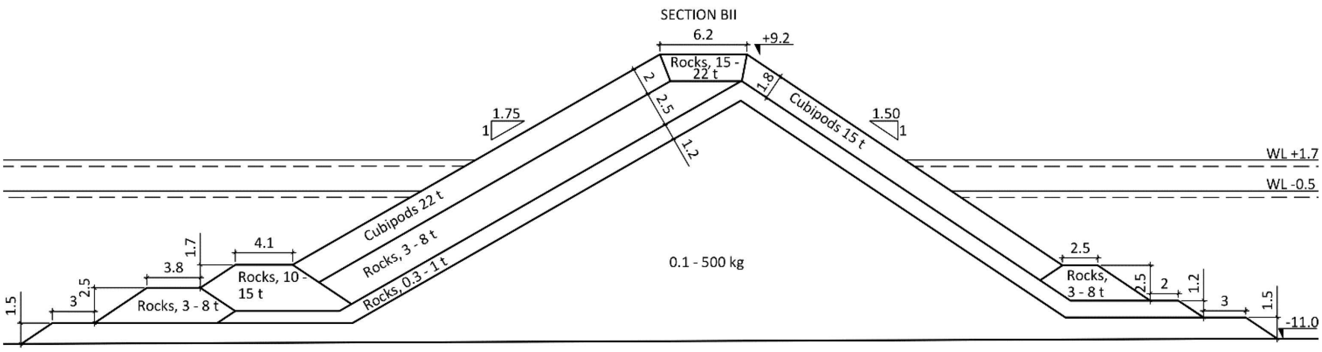

Fig. 3. Proposed cross-section for outer trunk (section BII in Fig. 5).

In the proposed reinforced design, the weight of the Cubipods® on the outer part of the trunk was increased from 15t to 22t. Moreover, on the outer part of the trunk the rear side was reinforced by replacing 15-22 t rocks with 15 t high density Cubipods®. The proposed cross-section for the most exposed part of the trunk is shown in Fig. 3. In the tender it was required that the roundhead should include a vertical caisson with fenders to increase safety for ships at the entrance, see Fig. 5. The front and rear roundhead design are shown in Fig. 4.

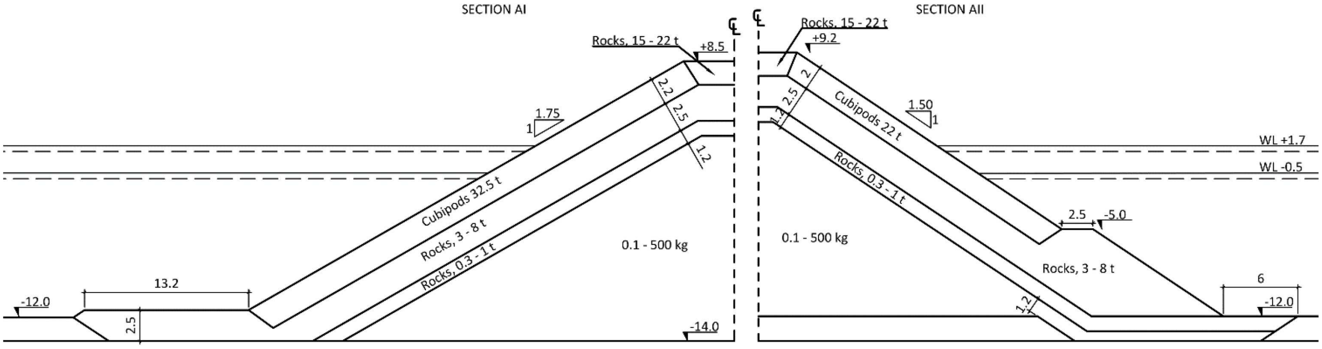

Fig. 4. Proposed cross-section for front side (left) and rear side of roundhead (right).

4 Model Test Setup and Methodology

The three-dimensional model tests were performed in scale 1:44.9 in the Ocean and Coastal Engineering basin at Aalborg University. The basin is equipped with a segmented wavemaker consisting of 30 pistons. The wavemaker is built with an unusual high discretization using vertically hinged paddles with segment width of 0.43 m. This leads to high quality oblique and short-crested waves with spurious waves of only small amplitude.

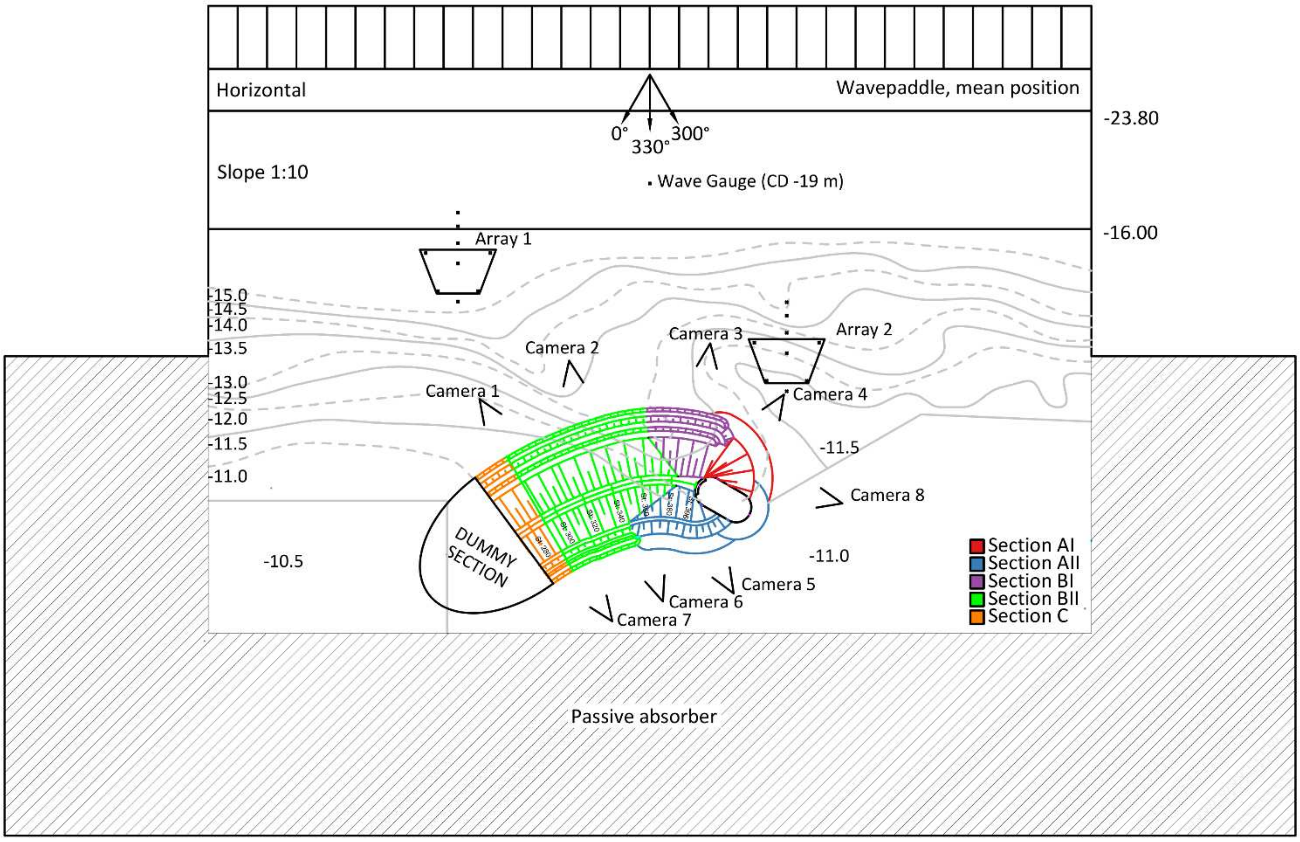

Fig. 5. Model test setup. Seabed contour levels in metres relative to CD (prototype scale).

In order to generate accurately the depth limited waves in the basin significantly larger depth is needed at the wavemaker. For this reason, a 1:10 slope was established in front of the wavemaker followed by the provided design bathymetry. The model was oriented so 330° corresponds to perpendicular to the wavemaker and thus the other two directions tested corresponds to ±30°. The model setup is shown in Fig. 5. The modelled part of the western breakwater is the outer approx.

210 m. Damages were measured by photo overlay techniques using several stationary cameras. Placement of the Cubipods® were done to achieve target porosities of 40-42% in order to match the as placed porosity at site.

Accumulated damage was measured by running a serious of tests ranging from 1 to 100 year return period from different wave directions with high water level (HWL) and low water level (LWL). If the structure did not fail in these tests the overload conditions were tested to confirm the reserve stability of the breakwater. The first construction tested was the initial desk design (Construction 1). Subsequently, different optimizations were tested in Constructions 2-7.

5 Wave Generation and Analysis

Due to the complex and steep bathymetry and the highly nonlinear and breaking short-crested waves it is difficult to both accurately generate and analyse the waves in the model tests. This required use of state-of-the-art wave generation and wave analysis methods.

5.1 Wave Generation Method

The first order spectrum was generated based on the InvFFT Random Phase method based on the JONSWAP spectrum with a peak enhancement parameter, $\gamma = 3.3$. The directional spreading function used was Longuet-Higgins et al. (1963) with a constant spreading parameter $s = 30$ for all frequencies.

The bound second order superharmonics were generated using Schäffer and Stenberg (2003) method with modifications by Eldrup and Lykke Andersen (2019a). Due to the limited stroke available it was decided not to generate the bound second order subharmonics correctly.

As shown in Fig. 5 the basin has partly reflecting and partly absorbing sidewalls. The reflecting sidewalls were used to extend the area in the basin with correct wave height using corner reflection compensation based on Dalrymple (1989). Active absorption based on digital filtering of signals from one wave gauge positioned on the paddle face was used (Lykke Andersen et al. (2016)). The waves were generated using the AwaSys 7 software (Aalborg University 2019a).

5.2 Wave Analysis Method

The data acquisition and wave analysis were performed by WaveLab software (Aalborg University 2019b). Surface elevation signals were sampled at 100 Hz and digital bandpass filtered with a passband of 0.05 to 5 Hz in model scale. This ensures that superharmonics are included in the analysis up to high order.

In order to overcome the difficulties in separating incident and reflected waves most of the tested sea states were calibrated without the model in place. Two wave gauge arrays were used to measure the waves both with and without the model in place, see Fig. 5. Additionally, for the calibration tests a wave gauge was placed on the 1:10 foreshore at the depth where the design sea states are provided (CD -19 m), cf. Fig. 5.

Waves were also measured with the model in place in order to verify that the incident waves were similar to those at the calibration tests. However, no accurate method exists for predicting incident wave parameters in nonlinear short-crested waves. The BDM method by Hashimoto (1997) was used to obtain the directional wave spectrum and thus the incident frequency domain parameters. However, as the method is based on linear wave theory, the results may not be very accurate for the present conditions. Moreover, the method does not account for phase-locking of reflected components, and therefore results are also questionable when the wave gauge array is placed close to reflective structures (Array 2).

A nonlinear wave separation method exists for long-crested irregular waves that account for phase-locking, but it assumes all wave components to be in-line with the wave gauge array. The reflected waves from the model will not be in the opposite direction of the incident waves, and thus this may influence the results especially with the model in place and at Array 2. Moreover, waves are short-crested and not long-crested. This method was applied to arrays of in-line wave gauges and is expected to provide reasonable results when reflected waves are small and because the incident directional spreading was not that high, cf. Eldrup and Lykke Andersen (2019b). To be able to apply this method the wave gauge arrays were rotated to be in-line with the mean wave direction. Even though the method is not valid for short-crested waves, it was applied to have reasonable estimates of the time domain and to check the frequency domain parameters provided by BDM. These results showed that at Array 1 the estimated incident waves with and without the model in place are almost identical, whereas at Array 2 larger differences are present. The incident wave parameters estimated by Eldrup and Lykke Andersen (2019b) are at Array 1 almost identical to the total waves without the model in place. However, when using the BDM method, the estimated incident spectral significant wave heights are highly underestimated in highly nonlinear seas. Above shows the relevance of calibrating the incident wave conditions without the model in place for the tested conditions.

5.3 Calibrated Sea States

The calibrated sea states are shown in Tab. 2. Note only the original sea states given in Table 2 were calibrated. The additional sea states with lower period were not calibrated. However, based on the calibrated sea states the incident waves can be reasonable estimated with the model in place at Array 1 and this was used for these tests.

Tab. 2. Calibrated sea states at 300° and HWL (SI units). * = Note that for highly nonlinear waves BDM seems to provide a significant underestimation of H_{m0}.

Sea state return period	CD-19.0 m					Array 1, CD-16.0 m							Array 2, CD-12.9 m		
	Target		Total measured			Total measured			Incident 2-D			BDM	Total measured		
T_r	H_{m0}	T_P	H_{m0}	$H_{1/3}$	H_{max}	H_{m0}	$H_{1/3}$	H_{max}	H_{m0}	$H_{1/3}$	H_{max}	H_{m0}	H_{m0}	$H_{1/3}$	H_{max}
1	5.2	15.0	5.6	5.6	11.3	5.4	5.7	10.6	5.4	5.4	10.8	5.3	5.5	6.0	11.2
10	6.7	16.0	6.9	7.1	13.1	6.8	7.4	13.0	6.8	7.1	12.0	6.6	6.6	7.4	12.2
50	7.8	16.5	8.0	8.2	14.0	7.9	8.8	13.8	8.0	8.5	13.0	7.5*	7.6	8.7	13.0
100	8.2	16.5	8.4	8.7	14.9	8.3	9.5	14.4	8.4	9.2	13.4	8.0*	7.9	9.2	13.3

Array 1 indicates that the total significant wave heights were more or less 0.1-0.2 m higher than the incident with 2-D and BDM separation. This was taken into account when comparing the data at CD-19m with the target values. Based on this it was concluded that the calibrated sea states were accurate or maybe slightly to the safe side.

The measured time domain significant wave height ($H_{1/3}$) is for the most severe sea states up to 15% larger than the spectral significant wave height (H_{m0}) at Array 1. At Array 2 the difference is also pronounced for the lower return periods. The difference between $H_{1/3}$ and H_{m0} in shallow water is well-known and a consequence of the altered wave height distribution due to nonlinear shoaling and wave breaking, which is dependent on for instance the seabed slope. Goda (2010) presented results from model and prototype that demonstrate a very significant difference in spectral and time domain significant wave heights for highly nonlinear waves. However, the bathymetry is not given in his data and this must influence the ratio significantly.

Furthermore, at Array 2 the maximum value of the breaker index $H_{1/3}/h$ is found to 0.63 when the total measured waves are considered and 0.60 when the incident waves using the 2-D method are considered. The same values for H_{max}/h are 0.91 and 0.86, with h being the water depth at the measurement location (Array 2). This shows the design waves are depth limited at the toe of the structure. The breaker index is very dependent on the seabed slope and the above results correspond very well to the findings by Allsop and Durand (1998) for a seabed slope of 1:30.

6 Trunk Stability

6.1 Initial desk design

No Cubipods® were extracted during the 100 year events provided in Tabs. 1 and 2 on the front trunk. During the overload conditions one 15 t Cubipod® was extracted from the front side. This opened for the possibility that 15t Cubipods® on the front side was maybe enough and this were investigated more detailed in following constructions as only a narrow section with 15t units was included in the model with the initial desk design. Moreover, lower wave periods were not tested in Construction 1. The 3 - 8 t rocks in the toe observe significant damage, but 10 – 15 t rocks in the toe were sufficiently stable to still support the first row of Cubipods®.

The 15t Cubipods® on the rear side was stable during 100 year events with only minor settlements. During the overload conditions one unit were extracted so the design seems optimal.

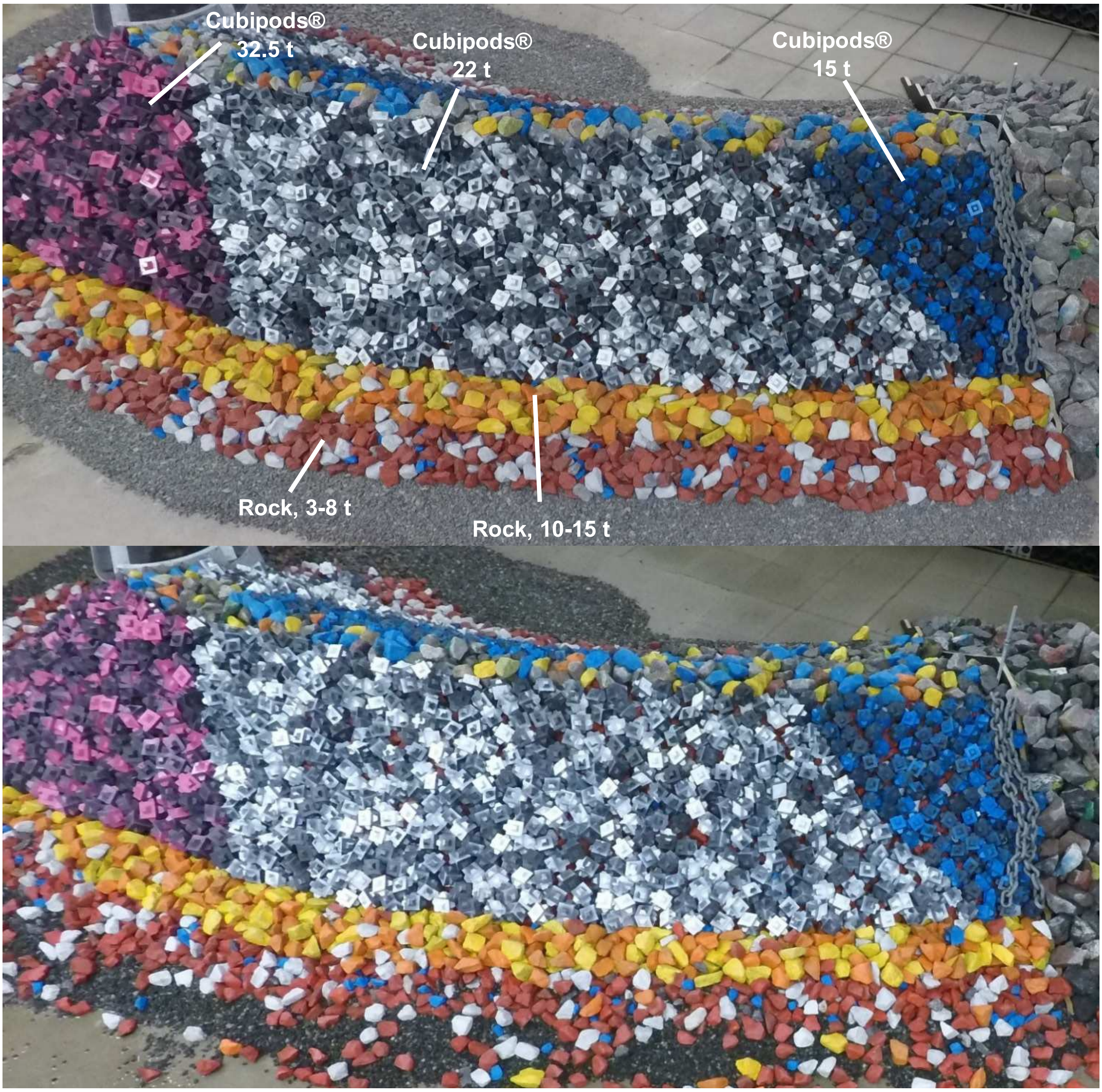

Fig. 6. Trunk section before testing (top) and after overload conditions (bottom).

6.2 Optimized design

In the optimizations a longer stretch with 15 t Cubipods® as front armour showed that one unit was extracted during the 50 year event. Three additional units were extracted when testing the 100 year event with a lower peak wave period (T_p = 12.5 s). This occurred even though the smaller period leads to more dissipation of energy at the foreshore and approximately 10% lower incident wave heights at Array 2 were measured ($H_{1/3}$ = 8.2 m and H_{max} = 11.9 m). This shows a significant influence of the peak period on the hydraulic stability for breaking waves and the shorter period waves were observed to break more violently on the front armour than the longer period waves. Continuing with the overload condition with long wave period showed complete destruction of the armour layer with 15 t units, see Fig. 7. Thus 22 t units are needed on the front side of the most exposed parts and the transition to 15t units should be selected carefully taking into account the results with lower wave periods.

For the 15 t units $K_D = 17.2$ is obtained for the 100 year event with long period (using $H_{1/3} = 9.0$ m and $T_p = 16.5$ s at the toe) and this significantly exceeds the recommended $K_D = 12$ in the Cubipod® manual. The 22t units leads to $K_D = 11.7$ for the 100 year event and is thus in-line with the recommendations.

Very recently it has been discovered by Eldrup and Lykke Andersen (2019c) that for rock slope stability the governing wave height parameter under nonlinear and breaking wave conditions is H_{m0} and not $H_{1/3}$ or $H_{2\%}$. If the same is the case for concrete armour units the K_D values are much lower (11.6 for 15t and 7.9 for 22t), which however based on the model results seem to be too optimistic and lead to a non-conservative design. The present stability results were also compared with the design formula of Gomez-Martin et al. (2019). However, due to the bathymetry the waves at the toe of the structure are much higher in the present tests than in their tests. Their formula leads to 9.5 t units for gentle foreshore slopes. The stability formula derived by Gomez-Martin et al. (2019) is thus not valid for the present case and shows the strong influence of the steep bathymetry, which causes nonlinear shoaling and higher breaker index and thus more severe wave conditions on the structure.

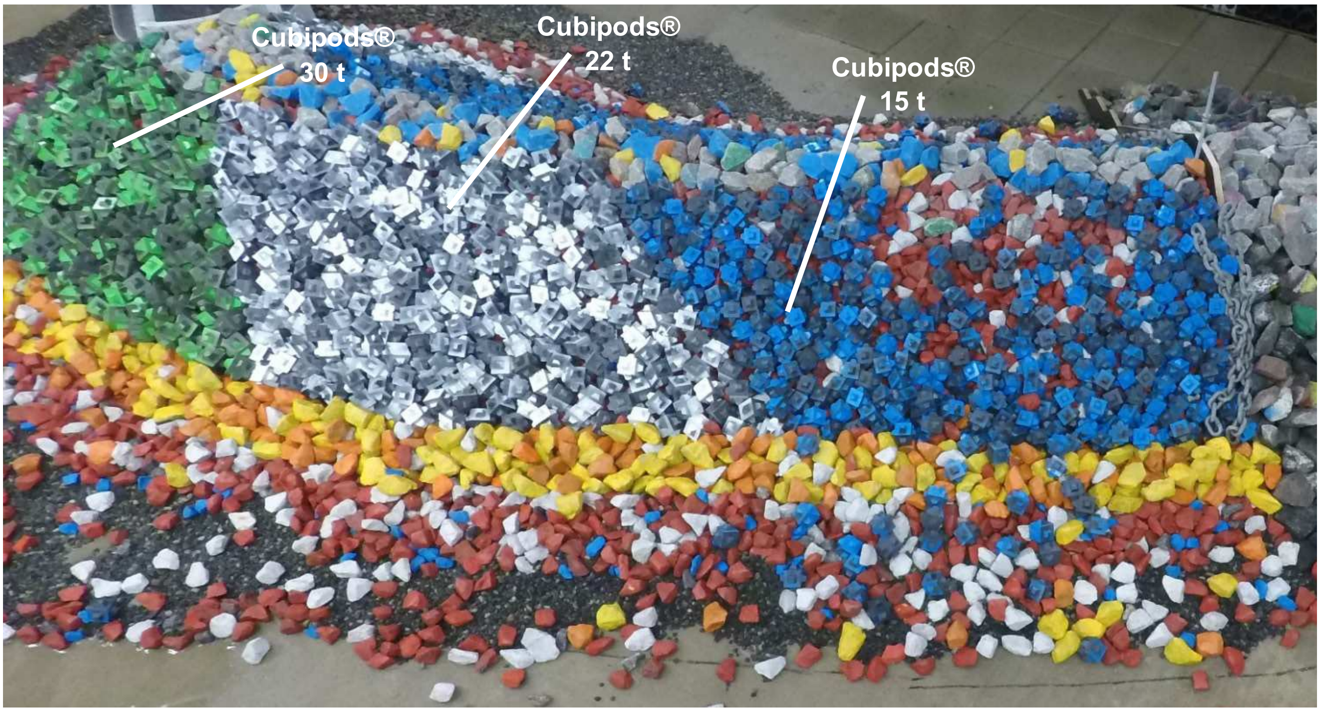

Fig. 7. Trunk section with 15 t units after overload conditions.

Furthermore, the tests with optimized designs showed that the 3-8t rocks underlayer on the rear side might be removed and still acceptable stability of the 15t Cubipods® was found. This leads to a cost reduction, but leads to less stability during construction against overtopping as 0.3-1t rocks are directly exposed until Cubipods® are placed. This design was also tested with a crest level of 8.6 m and the stability of the rear side was still acceptable, but some damage to the 15-22 t rocks on the crest must be expected. An alternative design with 15-22 t rocks on the rear side and a crest level of 9.2 m was also tested, but this showed far from sufficient stability. Thus Cubipods® in a single layer seems to be an attractive solution for rear side armour on highly overtopped structures.

7 Roundhead Stability

7.1 Original design

The original design showed insufficient stability of the 32.5 t high density Cubipods® along the caisson interface as two units were extracted during the 100 year design event. During the overload conditions one additional Cubipod® was extracted from the roundhead. The remaining Cubipods® in the roundhead were stable, but observed significant settlements. The 22t Cubipods® on the rear roundhead seems optimal as no damaged occurred during design events, but in two constructions units were extracted during overload. The toe on the rear roundhead observed damage but was still supporting the lower row of Cubipods®. The 0.3 – 1 t scour protection around the caisson observed significant damage/scour. Thus the interfaces with the caisson was the most significant focus point in Constructions 2-7.

Fig. 8. Front roundhead before testing (left) and after all tests including overload conditions (right).

7.2 Optimized design

A stable solution for the roundhead at the caisson interface was found by using one column of orderly placed 25t high density (ρ = 3,000 kg/m^3) rectangular blocks along the caisson. 17t high density (ρ = 2,830 kg/m^3) perforated rectangular toe protection blocks showed almost sufficient stability and slight increase in the density might be sufficient for these blocks also to be stable.

Moreover, a reduction in the weights of Cubipods® in the less exposed part of section AI to 30 t was tested. This solution is shown in Fig. 9 where 30t units are in green color. This solution showed acceptable results with no units extracted even in overload conditions, but significant settlements were observed with large gaps between units.

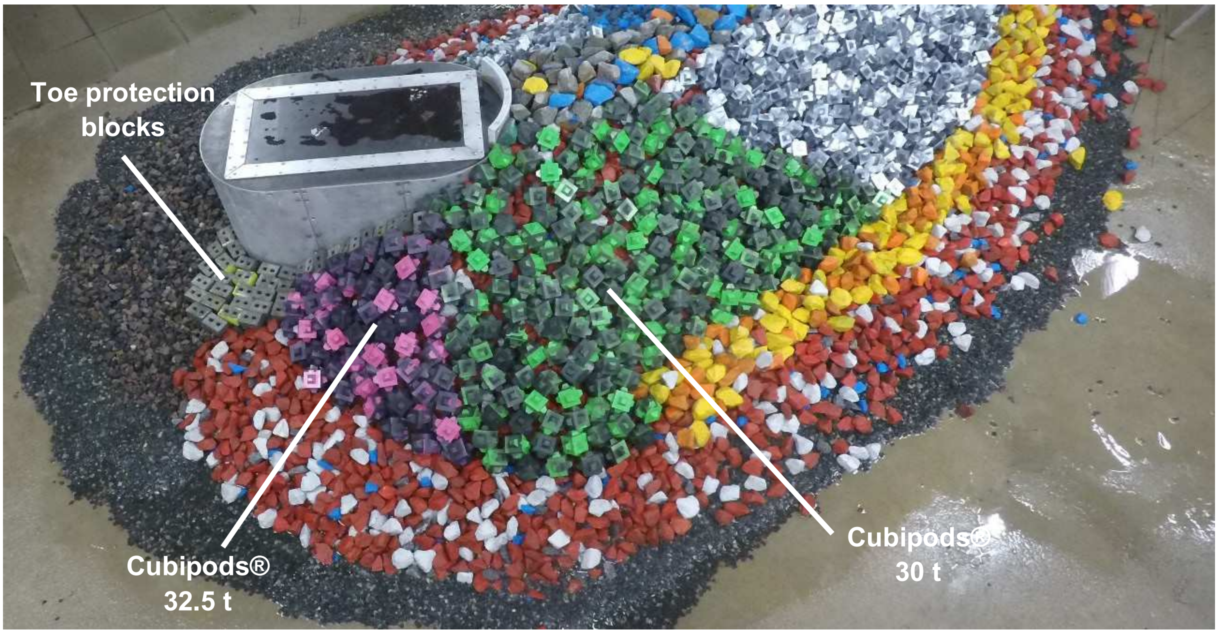

Fig. 9. Optimized roundhead and outer trunk solution after overload conditions.

The perforated toe-protection blocks were also proposed to reinforce the 0.3 – 1 t rock scour protection at the caisson, see Fig. 9. An alternative solution for the scour protection with 10 – 15 t rocks in an excavated trench also showed acceptable stability. As expected, the results showed that interfaces with the vertical caisson in the roundhead proved to be a critical area as the Cubipods® have no interlocking along the caisson and with the updated bathymetry the flow seems to be increased with a jet downwards along the caisson during wave impact from the most critical wave direction.

8 Conclusions

The present paper presented results from three-dimensional hydraulic model tests of the new Cubipod® armoured western breakwater for Port of Hanstholm. The design waves are characterized by highly nonlinear and breaking long waves on a complex and steep bathymetry.

Altogether, the presented example shows just how important it is to carry out 3-D physical model tests of breakwater stability when dealing with complex bathymetries and aggressive depth limited wave conditions, where current design formulae and best practice is insufficient to reach a safe design, especially when dealing with breakwaters with monolayer armour.

Several gaps seem to exist in the existing knowledge. A major lack is knowledge of the relation between spectral and time domain wave parameters including wave height distribution in breaking and highly nonlinear waves. The influence of long waves generated in the breaker zone, which also contributes to higher breaking wave heights in shallow water is also poorly understood. Moreover, the stability of armour units under these conditions seems not sufficiently studied. Another uncertainty contributing to this is that the present tests are with high density concrete units not commonly used and tested.

Stability of armour units and toe/scour protection next to caissons depends to a large degree on the wave conditions (especially the wave direction and seabed contours in front of the structure) as this significantly influence the downward generated jet along the caisson.

A concern in respect to the presented tests is that the bathymetry is highly dynamic and the bathymetry influences to a large degree the loading on the structure elements as it might cause focusing of waves in specific points along the breakwater. Therefore, the aim was to have some reserves in the final design, especially if the failure mode is not ductile and the worst-case bathymetry is uncertain.

The model tests confirmed the general robustness of the Cubipod® armour unit and its particular application for the new western breakwater in Hanstholm. Placing the armour units on a 1:1.75 slope and on a filter layer of large 3-8 t rock increase the overall robustness of the breakwater both during design events and in the construction stage. Even in the tests where the Cubipod® armour suffered damage above acceptable limits, the breakwater proved to have considerable reserve stability and was in no way near a total collapse – even for wave conditions well beyond the specified design condition.

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