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New Cubipod armoured breakwater in Hanstholm, Denmark: design and construction

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The Port of Hanstholm, located on the exposed North Sea coast of Denmark was expanded during the years 2017–2020. The port expansion features a new large western breakwater in 9–13.5 m water depth with 15 t to 32.5 t Cubipods as its main protective armour. The breakwater head towards the harbour entrance is established by a large prefabricated concrete caisson (40 × 22 × 18 m). The breakwater was extensively tested in the 3D wave basin at Aalborg University, which revealed some remarkably benefits of Cubipods over other concrete armour units. Detailed design of the breakwater, the largest ever built in Denmark, was carried out by COWI A/S for the marine contractor Per Aarsleff A/S, who won the port expansion design–build contract. This paper presents the design consideration for the new western breakwater, including considerations of different breakwater concepts and details on why Cubipods in one layer was eventually chosen as the preferred armour for the breakwater. In addition, the paper describes the laboratory testing of the breakwater and its performance under severe wave conditions. The paper also touches on some of the innovative methods which were developed by the contractor to keep a high armour layer production rate under challenging wave conditions.

Keywords: buildings, structures & design/coastal engineering/laboratory tests/ports, docks & harbours

Notation

Cot(a)	slope of breakwater (1: x)
H_{m0}	spectral significant wave height
H_{s}, $H_{1/3}$	time domain significant wave height
H_{sb}	breaking significant wave height
K_{D}	breakwater stability number in the Hudson formula
K_{t}	wave transmission coefficient, $H_{\text{m0,t.}}/H_{\text{mo,i}}$
M_{50}	mass of rocks for which 50% is lighter
N_{d}	damage number, the number of armour units displaced in area considered
N_{od}	damage number, the number of displaced units per width, D_{n}, across armour face
P	permeability in the Van der Meer equation
q_{av}	time-average wave overtopping discharge per metre run of crest
S_{d}	damage number in the Van der Meer equation
s_{po}	wave steepness ratio of significant wave height (H_{s}) to offshore peak wavelength (L_{p})
T_{p}	spectral peak wave period
V	volume of concrete armour units (CAU)
W	mass or weight of CAU
γ_{t}	wave transmission coefficient, $\gamma_{\text{t}} = H_{\text{m0,trans}}/H_{\text{m0,incoming}}$
Δ	relative density of the armour material $(\rho_{\text{c}} - \rho_{\text{w}})/\rho_{\text{w}}$
ρ_{c}	density of concrete
ρ_{w}	density of water

1. Introduction

In 2017–2018, the marine contractor Per Aarsleff A/S won the design–build contract for a new port expansion in the Port of Hanstholm, Denmark, together with dredging contractor Rohde Nielsen A/S and consultant engineer COWI A/S. The port expansion features a new 400 m long western breakwater in 9–13.5 m water depth, the largest ever built in Denmark.

The Port of Hanstholm is located on a headland on the west coast of Jutland (see Figure 1) and is subject to waves generated over a fetch reaching all the way to the Shetland Islands, Scotland, in the North Atlantic. Design (extreme) wave conditions reach more than H_{m0} = 8 m with peak wave periods of T_{p} = 14–16 s. Tides are mild, with a tidal range of approximately 0.35 m, but storms can cause up to 1.6 m storm surge.

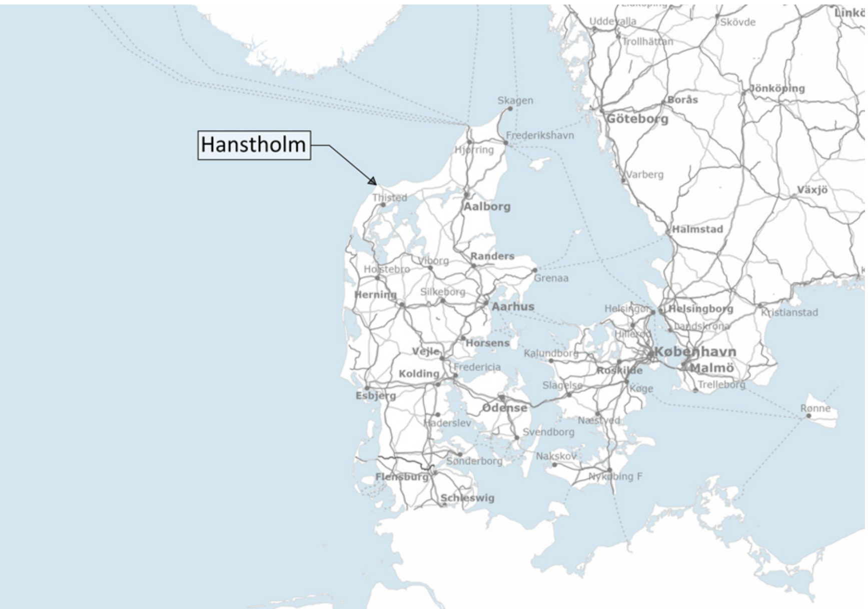


Figure 1. Location of the Port of Hanstholm

The port is subject to several storms every year, and on average wave heights exceeding $H_{m0} > 5$ m are experienced several times annually. Even during the summer season from April to September, wave heights reach $H_{m0} = 3.5–4.0$ m every year.

Construction of a breakwater at Hanstholm is further challenged due to coastal morphology, strong littoral currents and an estimated sediment transport towards the north-west in the order of 0.7 million cubic metres per year.

The Port of Hanstholm is famous in the field of coastal engineering due to its history. Construction of a port in Hanstholm began in 1917 and was led by engineer Jørgen Fibiger (1867–1936). The first breakwater consisted of large timber caissons, which were floated into position and filled with gravel and concrete. A superstructure of concrete was cast on top of the caissons. The first caisson was placed in Hanstholm in 1926, but construction was delayed due to bad weather, and in 1935 only the western breakwater was finished. During the German occupation of Denmark under the Second World War from 1940 to 1945, construction stood still and maintenance of the structure was neglected while the timber was attacked by shipworms. In addition, the breakwater suffered severe damage during a winter storm in 1942. After the war, repair works were initiated but never completed due to political disputes. The breakwater was eventually abandoned (Juhl, 1994).

In 1955, plans for a new port at Hanstholm were taken up again and led by Professor Helge Lundgren, who performed physical model testing of caisson breakwaters in the Netherlands and sediment transport studies at facilities near the Technical University of Denmark (DTU). The work of Professor Helge Lundgren led to the founding of the Institute for Coastal Engineering (*Vandbygningsinstituttet*) at DTU in 1964, which later became DHI. This work led to the construction of the port, which was opened in 1967 after 7 years of construction.

The Lundgren breakwater consisted of low-crested circular caissons with a diameter of 12.5 m and a wall thickness of only 0.25 m. The caissons were placed in succession using a large portal crane driving on top of the already installed caissons. During installation, the caissons did not have a bottom plate but were lowered onto the limestone underlayer and then filled with 1–2 m of concrete. The circular shape helped reduce the wave impact force, and further reduction was obtained by introducing an inclined superstructure (Lundgren and Juhl, 1995).

This paper presents the design consideration for the new western breakwater, including considerations of different breakwater concepts and information on why Cubipod in one layer was eventually chosen as the preferred armour for the breakwater. The paper focuses on the most exposed outer section of the breakwater excluding the breakwater roundhead. The authors have decided not to go into detail about the concrete caisson roundhead and the challenges of joining it with the mound, because they find it to be too extensive for one paper. The authors plan to cover these aspects in a separate paper to be published in the near future.

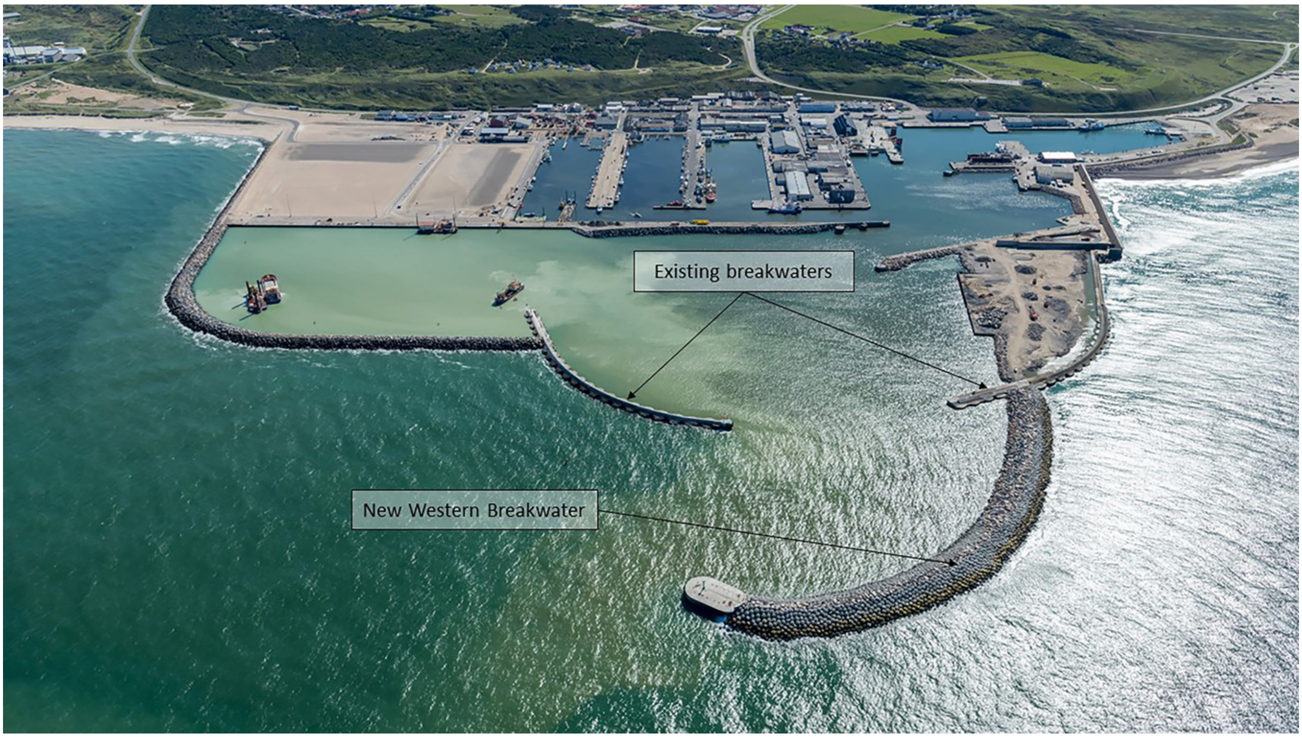


Figure 2. Port expansion project

It is noted that the port layout was defined by the owner and is not addressed further in the paper.

The port expansion project is shown in Figure 2 and comprises

- 400 m western breakwater
- 40 × 22 × 17 m caisson breakwater roundhead
- 1000 m eastern breakwater/revetment
- 350 m quay
- 130 000 m^3 additional hinterland
- increased water depths to 10.5 m.

2. Site conditions

The new western breakwater was constructed as a northward extension of the existing western caisson breakwater from the 1960s. As shown in Figure 3, water depths varied from −9 to −12.5 mCD along the breakwater and with bed slopes of up to 1:30 north-west of the most exposed outer part of the breakwater.

The strata consist of a surficial layer of limestone covered by post-glacial marine sand. The marine sand layer is very mobile with varying thicknesses of 0 to 4 m along the breakwater.

2.1 Design wave conditions

Design wave conditions defined by the owner are summarised in Tables 1 and 2. Notice in Table 2 that the outer section (Station 320–380) was to be designed according to tests in the wave laboratory in Aalborg University and that the design wave height was informed to be $H_{1/3} > 7.5$ m.

Due to severe wave breaking on the steep seabed in front of the breakwater, the wave spectrum is distorted by the reorganisation of wave energy. This was exemplified in the laboratory where a joint north sea wave observation project (JONSWAP) wave spectrum with a spectral significant wave height of H_{m0} = 8.2 m and a peak wave period of T_p = 16.5 s (from Table 1) was applied offshore in 19 m depth. This sea state generated a much larger time-domain significant wave height ($H_{1/3} = H_s$) of around $H_{1/3}$ = 8.5–9.0 m at the breakwater, which was located at seabed level of approximately −12.5 mCD. Therefore, the project distinguishes between the spectral significant wave height (H_{m0}) and the time domain significant wave height ($H_{1/3}$), as also noted in the design conditions provided by the owner in Table 2, which are all based on $H_{1/3}$.

Design water levels during extreme wave events were also defined by the owner as summarised in Table 3. The design water levels include the effect of surge, sea level rise and tide (which are mild).

2.1.1 Wave conditions during construction

Due to the rough wave conditions, breakwater construction was only possible during the summer seasons (April–September). In

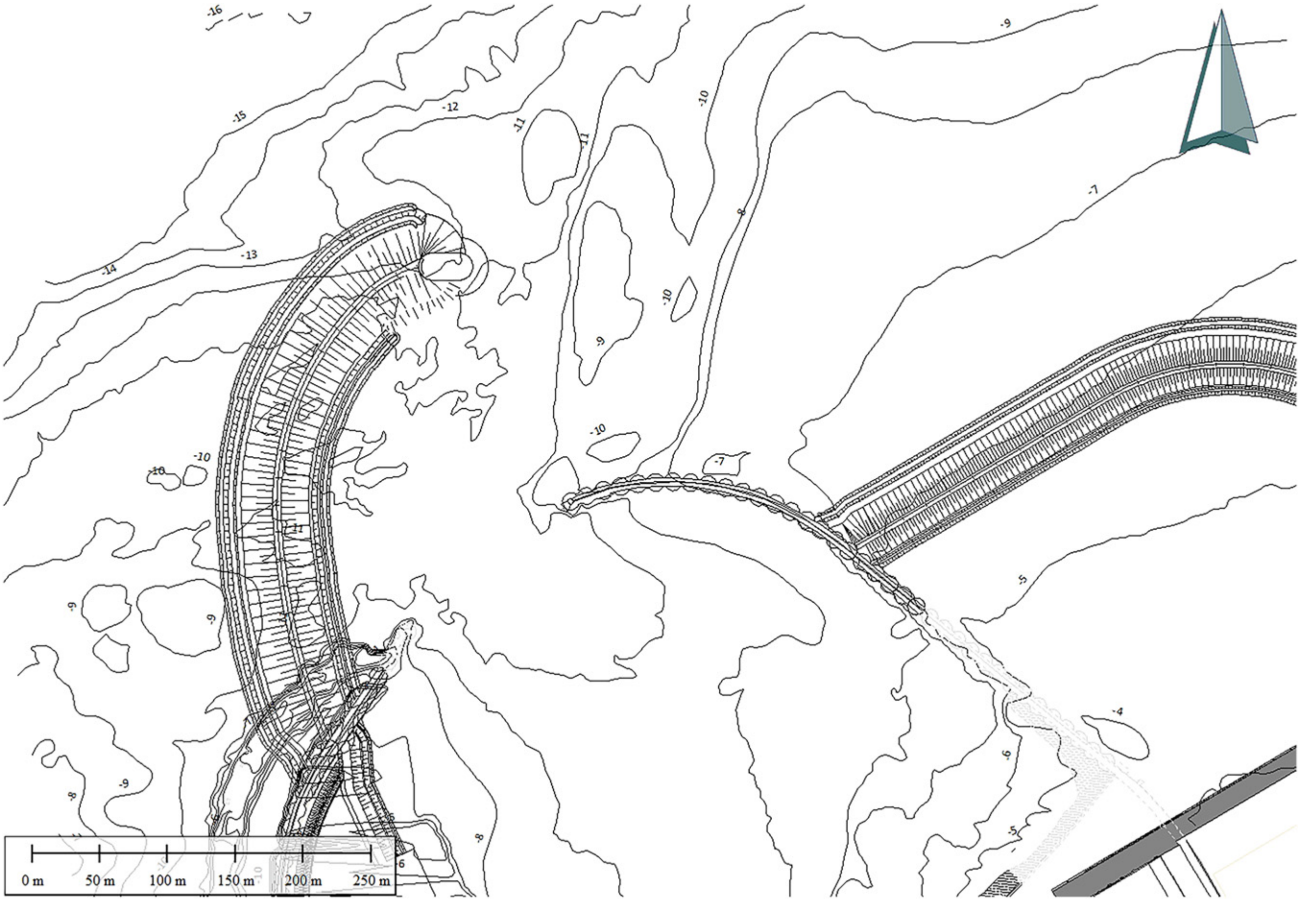


Figure 3. Bathymetry at western breakwater

Table 1. Offshore design waves in 19 m depth

	Wave direction		
	270°	300°	330°
Return period		H_{m0} (T_p)	
1 year	4.3 m (14.5 s)	5.2 m (15 s)	2.9 m (15 s)
10 years	5.2 m (14.5 s)	6.7 m (16 s)	4.4 m (15 s)
50 years	5.9 m (15 s)	7.8 m (16.5 s)	5.5 m (15.5 s)
100 years	6.1 m (15.5 s)	8.2 m (16.5 s)	6 m (15.5 s)

Table 2. Design wave conditions (100-year return period) along the western breakwater. Station 380 m is at the breakwater roundhead

Station	Wave direction	$H_{1/3}$	T_p
0–245 m	300°	7.2 m	16.5 s
245–320 m	300°	7.5 m	16.5 s
320–380 m	300°	>7.5[a] m	16.5 s

[a] Designed based on tests in wave laboratory

Table 3. Design water levels along breakwaters associated to the wave heights summarised in Table 2. Water levels are relative to CD

Return period	Design low water	Design high water[a]
1	−0.5 m	+1.3 m
100	−0.5 m	+1.7 m

[a] Including the effect of sea level rise

this season, wave heights exceeding H_{m0} > 3 m are experienced monthly (see Figure 4) and on average there will be one to three summer storms with H_{m0} > 3.5 m every year. Summer extreme wave conditions are summarised in Table 4 and show that there would be a significant risk that the breakwater would be exposed to a summer storm with H_{m0} > 4.5 m during construction.

In fact, on the Friday, 21 September 2018, the western breakwater was struck by the summer storm ‘Knud’, which came during construction and with less than 24 h prior warning in weather forecasts (see Figure 5). The storm was announced while the Contractor Aarsleff was installing the armour layer (Cubipods)

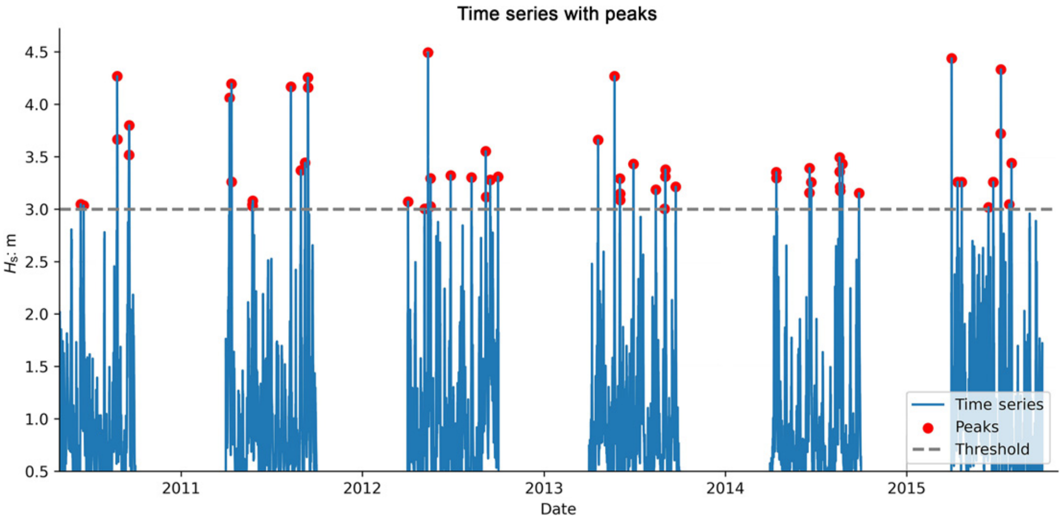


Figure 4. Events higher than a threshold of $H_{1/3}$ = 3 m in Hanstholm during the construction season from April to September

Table 4. Summer extreme waves in 19 mCD depth

	Wave direction		
	270°	300°	330°
Return period		H_{m0}	
1 years[a]	2.6 m	3.6 m	2.2 m
5 years[a]	2.9 m	4.6 m	2.9 m
10 years[a]	3.1 m	4.8 m	3.4 m

[a]Seasonal extremes – based on seasonal data April–September (1981–2015) in 19 m depth

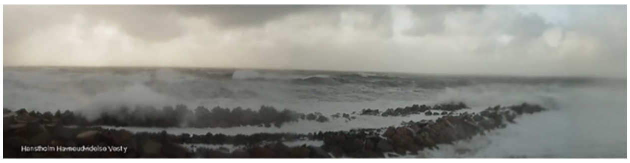


Figure 5. On 21 September 2018, the storm 'Knud' hit the partly constructed western breakwater

along the outer part of the partly constructed breakwater. There was no time to protect and cover the exposed lengths of the filter layer and exposed core before the storm hit and consequently the filter and core suffered quite significant damage and had to be repaired afterwards. Unfortunately, the wave gauge at Hanstholm had been relocated to a more sheltered position prior to the storm 'Knud' and therefore reliable measurements were not available. However, from other offshore measurements and the damage observed to the exposed filter layer, it is assessed that the wave height was in the order of H_{m0} = 4.5–5 m, categorising it as a 5–10 years summer event.

3. Design conditions and requirements

The design requirements for the western breakwater presented in Tables 1–3 were defined in the design basis prepared by Rambøll A/S for the Port of Hanstholm.

The most important contractual design requirements are listed in the following:

- overtopping, 1-year return period, $q_{av} \leq 20.0$ l/s/m
- wave transmission, 1-year return period, $K_t \leq 10.0\%$
- front slope damage level, 100-year return period, $S_d \leq 2.0$
- rear slope damage level, 100-year return period, $S_d \leq 4.0$
- toe stability number, 100-year return period, $N_{od} \leq 1.0$ or $N_d \leq 10\%$.

3.1 Constructability considerations

In addition to the contractual requirements, the following constructability considerations and constraints were defined by the contractor Aarsleff as the basis of the design.

- *Construction from land*: Reduce downtime by allowing construction with land-based equipment. Allow for a construction road on the breakwater in level approximately +1.5 mCD. Limited reach of excavators meant that the breakwater should not have a slope of more than approximately 1:2.
- *Stable filter rock during summer storm*: Use large filter rocks on a slope no steeper than 1:1.75, which will be stable in the event of a sudden summer storm where the filter has not been covered with armour units.
- *Limit unit weight*: Use units and rocks that are lighter than 15–17 t due to limiting lifting capacity (this was the limitation during tender stage – Aarsleff later acquired equipment capable of placing units of up to 25–30 t).

- *Limit rock weight*: The largest armour rock easily available to Aarsleff was individual rocks of up to approximately 25 t normal density and 15 t for high density. The corresponding median mass, M_{50}, would be considerably lower than 25 t and 15 t.

It is noted that some of the above constraints are contradictory. Bullet 1 contains a preference towards steep slopes due to excavator reach limitations and bullet 2 contains a preference towards milder slopes in order to reduce the risk of damage to the filter layer during construction.

4. Development of breakwater concept

During the tender stage, several breakwater concepts were evaluated during workshops held between Aarsleff and COWI. The concepts ranged from caissons to rock-armoured rubble mounds to various interlocking monolayer armour types.

4.1 Caissons

Professor Lundgren's breakwater from 1967 consisted of 12.5 m diameter caissons with a low and sloping wave wall. Caissons have certain advantages at Hanstholm due to the shallow and competent limestone at the site, which provides a robust, non-deformable and erosion-resistant base unsusceptible to scour and liquefaction. However, installation of caissons in Hanstholm is challenging due to the dynamic morphology and the rough wave conditions. The construction works of the existing breakwaters (which were actually performed from land) took 7 years to complete (from 1960 to 1967) and was delayed by 4 years due to rough wave conditions and continuous challenges caused by sedimentation. Moreover, caissons were not preferred by the Port of Hanstholm who had more than 50 years of experience with maintenance and repair of the existing caissons. Lastly, as opposed to large rubble mound breakwaters, construction of large caisson breakwaters is not a core competence of the contractor Aarsleff. Therefore, caissons were taken off the table at an early stage.

4.2 Rock-armoured rubble mound

Most breakwaters in Danish ports are rock-armoured rubble mounds. This is possible because wave conditions are relatively mild along the coasts facing the inner Danish waters and because high-quality rock is available in large quantities from quarries in Sweden and Norway.

However, due to the severe design wave conditions at Hanstholm, the rock armour would need to consist of substantial quantities of high-density basalt or norite in gradations of 20–30 t. And even with such rock, the breakwater would need a very mild slope of ∼1:2.75 to be hydraulically stable.

An alternative solution could be a berm breakwater, which would be allowed to reshape and adapt to the wave climate over time. However, such a concept would not meet the employer's requirements about minor front and rear armour damage during design events. Moreover, a berm breakwater would significantly increase the quantities of rocks to be transported – and (potentially) the price of the breakwater.

Hence, while rock-armoured rubble mounds were the preferred option of the contractor, it was decided to use concrete armour units (CAUs) and not rock as the primary armour layer on the seaward side and partly on the rear side as well.

4.3 Accropode, Core-loc or Xbloc armoured rubble mound

Solutions with CAU such as Accropode, Core-loc and Xbloc were also evaluated (see Figure 6) since these interlocking monolayer units have very high hydraulic stability according to the patentees of the various units.

However, the placement pattern and interlocking of the units require a relatively smooth underlayer and a steep slope no flatter than 3/2 (1:1.5). This was a particular challenge in Hanstholm because the filter layer would be exposed to severe wave conditions during construction.

It follows that choosing the optimal dimension of the CAU to match the design wave conditions would lead to a high risk of damage to the filter layer during construction. However, choosing the optimal dimension of the filter rock to reduce the risk of

Figure 6. Concrete armour units considered for the western breakwater in Hanstholm

damage during construction would lead to a significant increase in CAU dimension.

For example, as noted in Section 2.1.1, wave conditions during construction in Hanstholm would likely exceed H_s = 3.6 m and potentially H_s = 4.5–5 m. Therefore, the optimal filter rock dimensions should be stable for wave heights of up to H_s = 4 m and be repairable for waves up to H_s = 5 m. Using the Van der Meer equation, it is found that such stability is met with a 4.5–8.5 t filter rock gradation with M_{50} = 6.5 t on a 1:1.5 slope.

Using the latest design guidelines by Concrete Layer Innovations (CLI) for Accropode II (CLI, 2023) it is found that 4.5–8.5 t filter rock is allowed under Accropode II units of 20 m^3 and larger (for Xbloc the units would need be at least 16 m^3). Hence, as summarised in Table 5, the contractor would have to choose 20 m^3 Accropode II units to match with the optimal filter rock size in Hanstholm. The 20 m^3 units would need to have a minimum concrete density of ρ_c = 2400 kg/m^3 to match the design wave conditions, leading to a unit weight of W = 48 t.

If the lifting capacity of the contractor is limited (see Section 3.1), they will prefer lighter units. This may be achieved by shifting to Accropode II units with a higher concrete density of, for example, ρ_c = 2700 kg/m^3, in which case armour units of W = 30 t (11.1 m^3) match the design wave conditions. However, the lighter and smaller armour units of 11.1 m^3 cannot be installed on the 4.5–8.5 t filter rock gradation. The design guidelines for Accropode II (CLI, 2023) require that the filter gradation is reduced to 2.3–5 t (M_{50} = 3.6 t), which would suffer damage if unprotected and exposed to waves of more than H_s = 2.8 m (see Table 5).

Hence, while interlocking armour would in principle be adequate for the wave conditions in Hanstholm and would allow Aarsleff to build a steeper breakwater using lesser quantities of concrete, core and filter material, it would also increase the risk of damage to the unfinished breakwater during construction. Moreover, with a lifting capacity of only 30 t, Aarsleff would not be able to install the larger armour units required to reduce the risk of filter damage during construction. For these reasons, Accropode, Core-loc or Xbloc armour were not preferable.

Furthermore, the interlocking units are vulnerable to almost unavoidable rocking and settlements in the armour layer, which may cause the units to break and thereby reduce the safety of the breakwater. This aspect is further addressed in, for example, Juul Jensen *et al*. (2023).

4.4 Concrete cube armoured rubble mound

Solutions with rectangular or cubic concrete units have some advantages over interlocking CAU in Hanstholm, because there are fewer restrictions on the placement grid, steepness of slope and obtaining a smooth underlayer. Moreover, concrete cubes are capable of withstanding settlements and rocking without breaking, which make them more robust than the more slender interlocking units.

The drawback is that concrete cubes have a lower stability and roughness than the interlocking units, which lead to heavier units and increased overtopping (higher crest levels).

Using high-density concrete on a 1:2 sloped trunk section, the cubes would need to be 48 t and 29 t for a single-layer and double-layer configuration, respectively. Hence, with a lifting capacity of only 30 t, a

Table 5. Example of design to optimise dimension of CAU (Accropode II) for: option 1, unit weight (lifting capacity limitations) and concrete consumption, and option 2, filter rock stability when not protected and exposed to waves during construction. Filter layer stability calculated for: no. of waves, n = 1000; permeability, p = 3

Parameter	Unit	Option 1 Design optimised for unit weight and concrete consumption	Option 2 Design optimised for filter rock stability – if not protected during storm
Armour	—	—	—
H_s	m	9.0	
K_D	—	10.0	
ρ_c	kg/m^3	2700	2400
ρ_w	kg/m^3	1025	
Δ	—	1.63	1.34
Cot(a)	—	1.5	
V	m^3	11.1	20.0
W	tonne	30	48
Concrete consumption	m^3/m^2	1.38	1.66
Filter (largest gradation allowed)	—	—	—
M_{50}	tonne	3.6	6.5
H_s (start of damage)[a]	m	2.8	4.0
H_s (failure)[b]	m	3.8	5.0

[a] Threshold significant wave height corresponding to ‘start of damage’ on a 1:1.5 slope. Damage number S_d = 2

[b] Threshold significant wave height corresponding to ‘failure’ on a 1:1.5 slope. Damage number S_d = 8

concrete cube solution would need to have two layers, which meant a significant increase in concrete quantities over other alternatives.

Therefore, while the concrete cubes/rectangular blocks have significant technical advantages, it would be a very expensive solution compared with other alternatives.

4.5 Cubipod armoured rubble mound

As shown in Figure 6, Cubipods only distinguish themselves from cubes by the ‘pods’ on each side, which act as distance keepers and maintain armour layer porosity.

The units obtain their stability mainly through gravity and share many advantages with traditional concrete cubes in terms of robustness to rocking and settlements and because they do not require the same steep armour slope and smooth underlayer as interlocking units.

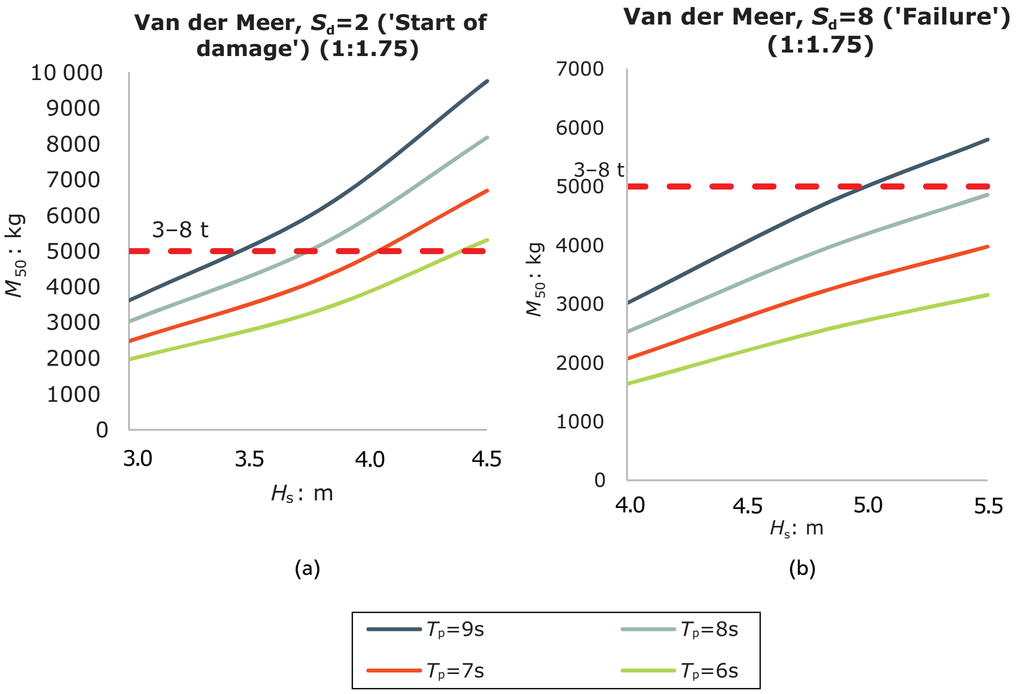
However, the Cubipod has a lower theoretical stability coefficient of around $K_D = 12$ (single layer) (Medina and Gómez-Martín, 2016) compared with patentee reported $K_D = 15$–16 for Accropode, Core-loc and Xbloc. The stability is, however, significantly higher than concrete cubes, which have a stability coefficient of around $K_D = 4$–5 for single layer placement.

COWI developed a design with 22 t Cubipods in a single layer on a 1:1.75 slope. The Cubipod had previously been used in a single layer, for example, in Punta Langosteira (Corredor *et al*., 2014), and with this design, both lifting and reach capacity of the contractor (Aarsleff) was met.

Accurate placement of CAU generally requires a quite smooth underlayer. However, the Cubipod units are less sensitive to underlayer smoothness than other more interlocking monolayer units and SATO (licence holder for the international Cubipod patent through Universidad Politecnica de Valencia (UPV)) allows filter layer rocks of up to $M_{50} = W/3$, where W is the weight of the Cubipod units. With Cubipods of down to 15 t on the less exposed inner part of the breakwater, it would thus be possible to use a filter rock gradation of 3–8 t (with a median mass of $M_{50} = 5$ t).

As shown in Figure 7(a), a 3–8 t filter layer placed on a 1:1.75 slope met the constructability requirement of being stable for wave heights of up to $H_s = 4$ m (depending on the wave period) and being repairable for waves up to $H_s = 5$ m. The risk of damage to the unfinished breakwater during construction was thus regarded as acceptable to Aarsleff. Moreover, the large rocks in the underlayer increased the permeability and thus the stability of the Cubipod front armour layer.

The Cubipod solution was subsequently selected as the preferred option of COWI and Aarsleff and was developed further during detailed design. The final trunk section of the breakwater is shown in Figure 8.



(a)

(b)

Figure 7. Exceedance of (a) ‘start of damage’ and (b) ‘failure’ criterion for 3–8 t filter rocks exposed to a summer storm. No. of waves, $n = 1000$; permeability, $p = 3$

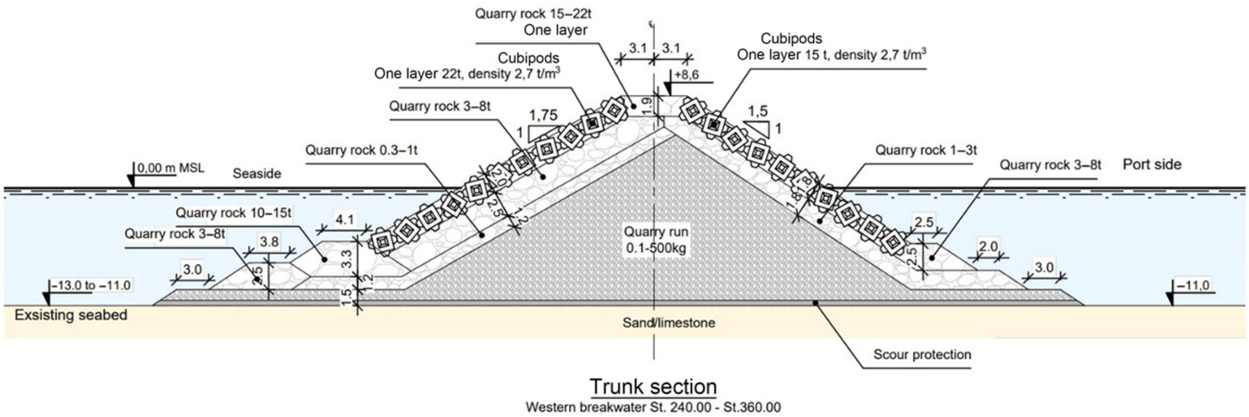


Figure 8. Final breakwater section BI (most exposed trunk section)

The final breakwater section has a single layer of 22 t Cubipod as front armour and 15 t Cubipod as rear armour. The crest consists of a single layer of densely placed 15–22 t quarry rock.

A relatively wide stepped toe of 10–15 t and 3–8 t rock forms the foundation for the Cubipod armour layer, which is placed on the aforementioned 3–8 t filter layer. In order to meet filter requirements, a secondary filter layer of 0.3–1 t is placed between the primary filter layer and the core of the breakwater, which consists of widely graded quarry run (0.1–500 kg).

5. Physical model tests

Three-dimensional physical model tests were performed at the coastal engineering basin at Aalborg University in Denmark in September 2018 (see Lykke Andersen *et al*., 2019).

To generate accurately the depth-limited waves in the basin, significantly larger depth was needed at the wavemaker followed by a 1:10 slope and then the actual bathymetry with slopes of up to around 1:30, as shown in Figure 9.

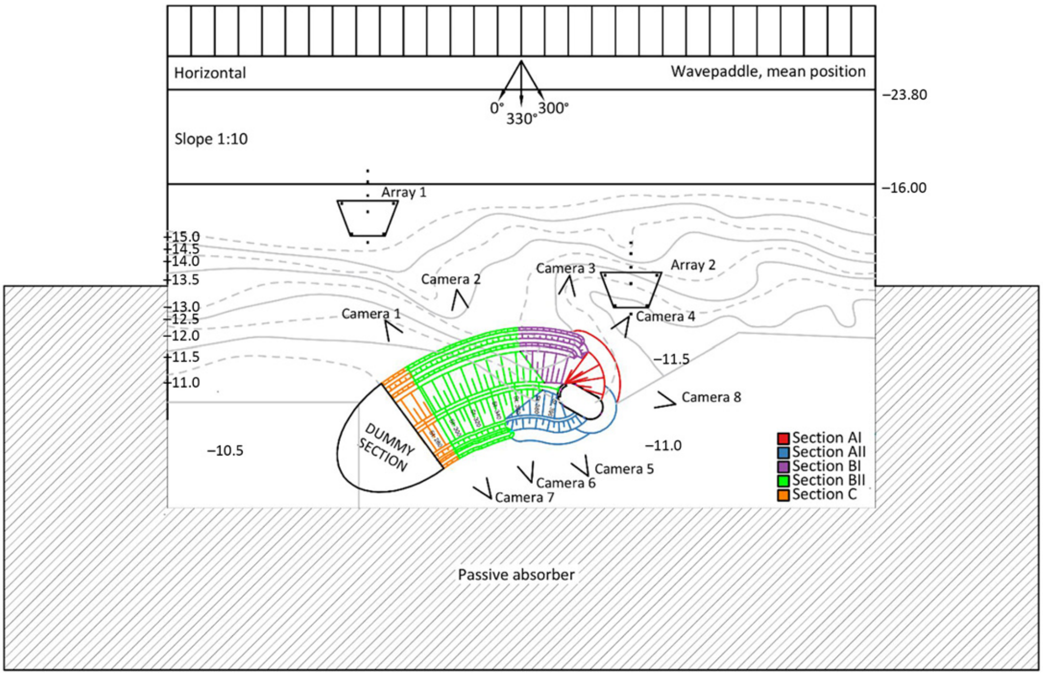


Figure 9. Model test set-up. Seabed contour levels in metres relative to CD (prototype scale)

For calibration, a wave gauge was placed in −19 mCD on the 1:10 slope and in addition wave gauge arrays were placed in two locations close to the structures (arrays 1 and 2 in −16 mCD and −12.9 mCD).

5.1 Wave height calibration

The breakwater was exposed to a set of test conditions as provided in Table 6, which include 1-, 10-, 50- and 100-year wave conditions and two overload tests with 10% increase of the 100-year design wave height.

Extracts from the calibration tests are shown in Table 7, and showed a remarkable difference between the spectral significant wave height (H_{m0}) and the time domain significant wave height ($H_{1/3}$) at arrays 1 and 2.

Moreover, at arrays 1 and 2, the time domain significant wave height of $H_{1/3}$ = 9.2–9.5 m was up to 15% larger than the incoming spectral significant wave height of H_{m0} = 8.2 m.

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 shoaling and wave breaking, which is dependent on, for instance, the seabed slope. These observations correspond very well to the findings by Allsop and Durand (1998) and Kamphuis (1996).

Allsop and Durand (1998) showed that the significant wave height ($H_{1/3}$) with a wave steepness of s_{po} = 0.018 such as the 100-year waves in Hanstholm can shoal by more than 20% on a steep slope, which agrees well with the measurements presented in Table 7.

Table 6. Test conditions (–19 mCD) for the most critical wave direction 300°N

Return period (T_r)	H_{m0}: m	T_p: s	Design high water: mCD	Design low water: mCD
1 years	5.2	15	1.3	—
10 years	6.7	16	1.5	—
50 years	7.8	16.5	1.7	—
100 years	8.2	16.5	1.7	−0.5
Overload 1	9	17	1.7	—
Overload 2	9	17	1.9	—

Table 7. Wave calibration in 3D laboratory with Lykke Andersen *et al.* (2019). WL = +1.7 mCD

Sea state return period		T_r	100 years
−19.0 mCD		H_{m0}	8.2 m
		T_P	16.5 s
Array 1, −16.0 mCD	Measured	H_{m0}	8.3 m
	—	$H_{1/3}$	9.5 m
Array 2, −12.9 mCD	Measured	H_{m0}	7.9 m
	—	$H_{1/3}$	9.2 m

From the point of breaking, the non-linear transformation of broken waves on a steep slope can be expressed with formulae by Allsop and Durand (1998), Goda (1974), Kamphuis (1996) and others. Kamphuis (1996) proposed a relatively simple expression that was found to fit well to the observations in Hanstholm:

$$H_{1/3} = 0.56(d_s + 0.1H_{sb})\exp[3.5\tan\theta]$$

where H_{sb} is the breaking significant wave height; d_s is the depth at the structure; $\tan\theta$ is the bed slope.

Using the expression of Kamphuis (1996) on the conditions at Hanstholm, one obtains a significant wave height ($H_{1/3}$) at the depth of wave array 2 of $H_{1/3}$ = 9.3 m for a bed slope of 1:50 (see Table 8), which compares very well with the measured wave height of $H_{1/3}$ = 9.2 m (see Table 7).

Using the same formula at the toe of the breakwater (d_s = 14.2 m) leads to a 100-year design significant wave height of $H_{1/3,100\ \text{years}}$ = 9.0 m (overload: $H_{1/3,\ \text{Overload 2}}$ = 9.2 m).

5.2 Cubipod performance

The Cubipods were placed in the diamond pattern dictated by the Cubipod manual (Medina and Gómez-Martín, 2016) and under the guidance and supervision of SATO. The placement of the units proved difficult on the rough underlayer of 3–8 t quarry rock, and as shown in Figure 10, the diagonal lines of Cubipods became somewhat ragged.

Tests were performed with both 15 t and 22 t Cubipods at the most exposed trunk section.

With design wave heights of approximately $H_{1/3,100\ \text{years}}$ = 9.0 m at the toe of the breakwater, this corresponds to a stability of K_D = 11.7 for 22 t units and K_D = 17.2 for 15 t units.

As shown in Figure 10, the 22 t Cubipod armour proved very stable with no extractions of units, even during the overload with

Table 8. Design significant wave height ($H_{1/3}$) at the Hanstholm western breakwater based on Kamphuis (1996) for different slopes 1:30 to 1:200

Tan θ	1:30	1:50	1:100	1:200
T_p: s	16.5			
$H_{m0,o}$: m	8.2			
Depth: mCD	−12.9			
Design high water: m	1.7			
d_s: m	14.6			
$H_{1/3}$: m	9.7	9.3	8.9	8.8
$H_{1/3}/d_s$	0.66	0.63	0.61	0.60

Figure 10. (Top) Trunk section before testing and (bottom) after two overload conditions

$H_{1/3,\ \mathrm{Overload}\ 2} = 9.2\ \mathrm{m}$ ($K_D = 12.5$). This exceeded expectations, especially due to the violent breaking of waves plunging directly onto the front slope, but also because the recommendation of Medina and Gómez-Martín (2016) $K_D = 12$ was exceeded.

The high stability of the 22 t Cubipod in the test could be influenced by the safety factor in the determination of K_D values and by the high porosity of 3–8 t rock filter. This last factor of increased filter layer porosity is believed to have a very positive influence on the high stability coefficient, K_D, observed in the laboratory. The authors suggest that this factor should be further analysed in future studies.

Following the test with 22 t Cubipods, a new test series was performed with 15 t Cubipods. During these tests, one unit was extracted during the 50-year event and three additional units during the 100-year event, while the overload test resolved in complete destruction of the 15 t Cubipod armour layer as shown in Figure 11. Thus, 22 t Cubipods were found to be adequate.

A key observation was how the Cubipod armour layer was able to self-arrange to cover irregularity or holes in the armour layer even after unit extractions. This meant that the armour layer was to some extent able to self-repair as long as the number of extracted

Figure 11. Trunk section with 15 t units after overload conditions

units was limited. Also, the bulky shape of the Cubipod will allow the units to rock and move without severe breakage and loss of mass, even in prototype scales. This ability makes a Cubipod armour more robust than most other single-layer units that tend to be more fragile and dependent on accurate and smooth placement.

5.3 Cubipod performance rear side

The rear side of the breakwater was severely battered by overtopping waves, and laboratory tests performed at an early stage of the project had shown that a 15–22 t rock gradation suffered unacceptable damage (exposed filter rocks) even after a 10-year event. After overload testing, the rear side and crest of the breakwater had completely failed, as shown in Figure 12.

Figure 12. Rear-side damage with 15–22 t rock after overload test

Test were subsequently performed with 15 t Cubipods on the rear side (see Figure 8). The 15 t units were placed on the same 3–8 t rock filter as was used under 22 t units, and as shown in Figure 13 the 15 t Cubipods on the rear side proved very stable, with no extraction of units even under the overload test.

5.4 Performance of rock crest

With different sizes of concrete units on the front and rear of the breakwater, it would be very difficult to install Cubipod units at the crest of the breakwater. The units would be likely not to meet the placement pattern and porosity requirements, and at the same time CAUs were not expected to archive a higher stability at the crest than quarry rock (of similar weight). A 15–22 t rock gradation, in contrast, could be placed to form a densely packed transition between the top row of Cubipods on the front and rear of the breakwater.

The stability of the rock-armoured crest was tested with both a crest level of +9.2 and +8.6 and found to behave satisfactorily with acceptable damage even at a level of +8.6, which was applied on the constructed breakwater (see Figure 8).

6. Construction

Aarsleff planned to construct the breakwater in the first two summer seasons (2018 and 2019) of the 3-year-long construction period. The 2020 season was thought of as a buffer in case of unforeseen challenges that would influence the production time schedule.

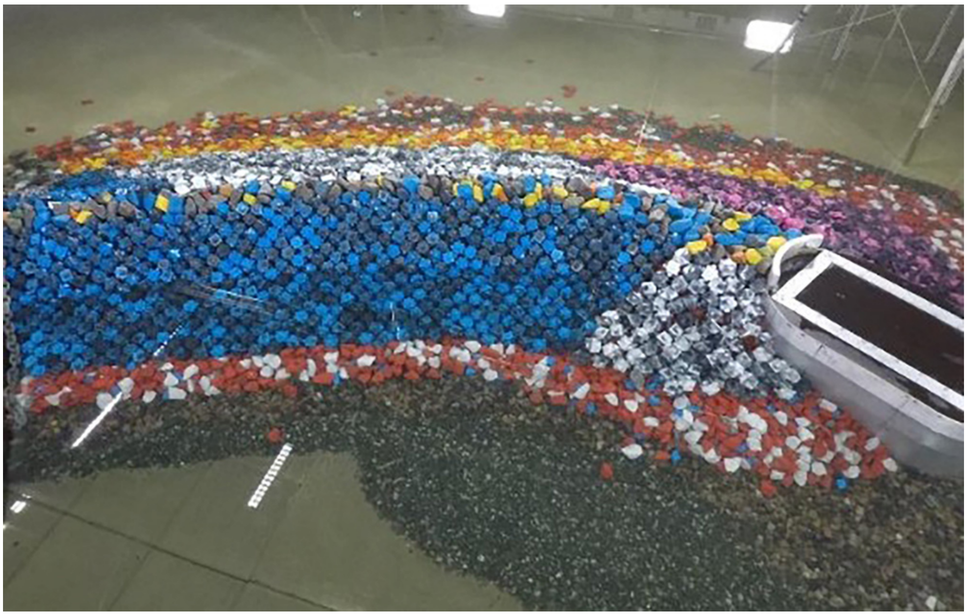


Figure 13. Rear-side damage with 15 t Cubipods after overload test

Such unforeseen challenges occurred, when the summer storm 'Knud' struck Hanstholm in September 2018 and caused a setback in the construction schedule. In addition, the owner commissioned a spur breakwater (submerged 'sand stopper') to be constructed, which meant that the third construction season was needed.

The breakwater was finished in August 2020, 4 months prior to the contractual deadline for the overall project.

6.1 Quantities

To construct the breakwater, the following quantities were delivered, handled and placed:

- 690 000 t of rock material of various gradings
- 1839 units of 15 t Cubipods
- 694 units of 22 t Cubipods
- 161 units of 30 t Cubipods
- 14 units of 32.5 t Cubipods.

The 30 t and 32.5 t Cubipods were placed in the roundhead of the breakwater.

The CAUs were produced at Aarsleff's production company Aarsleff BIZ in Świnoujście, Poland, from where they were shipped to Hanstholm on barges as shown in Figure 14.

6.2 Placement of Cubipods

Placement of the Cubipods was guided by a placement grid prepared by SATO. The relatively high hydraulic stability of Cubipod armours in the trunk is not generated by interlocking forces but is the result of a natural tendency (induced by gravity and the Cubipod geometry) to generate a homogeneous layer with low heterogeneous packing. This natural pattern of Cubipod armour favours the use of the recommended diamond-type placement grids, but the grid itself has status as a guiding tool rather than a requirement. The most important benchmark during construction was the porosity, which was monitored in panels based on units/m^2.

In existing Cubipod breakwaters, the units had been placed by wire crane with lifting tools like the one shown in Figure 15. This is a good tool when placing Cubipods above water since the Cubipods can be hooked quite fast and the placement can be visually checked.

Placing Cubipods under water with very limited sight is another matter.

Aarsleff favoured using hydraulic excavators for placing Cubipods, which had not been attempted before. Placing with excavators allowed for much faster and more accurate placement

Figure 14. Production and barge placement of Cubipod at Aarsleff Biz in Świnoujście, Poland

Figure 15. Placement of Cubipods with free-hanging lifting tool

because the operator would not only rely on GPS coordinates but was able to 'feel' the neighbouring Cubipods. As the grab was able to rotate, the operator could make sure the Cubipods had the correct support, which is crucial to single-layer armour units. For this operation, Aarsleff developed their own specially designed grab, as shown in Figure 16 (left).

The grab and the lifting tool were only designed to handle 15 t and 22 t Cubipods; therefore, to handle the largest units, wire grommets were cast into the units as shown in Figure 16 (right).

During fair weather conditions, Aarsleff managed to place an average of 85 units per 10 h shift.

Figure 16. (Left) Excavator equipped with special designed grab. (Right) 30 t Cubipod with wire grommet

7. Conclusion

The new western breakwater in Hanstholm (see Figure 17) was constructed from 2017 to 2020 in close collaboration between Aarsleff (contractor) and COWI (engineer) and SATO, who assisted in relation to the use of Cubipods in the project.

The breakwater was constructed at the most exposed location on the Danish west coast where the offshore wave conditions can have significant wave heights (H_{m0}) of more than 8 m with peak periods (T_p) of 14–16 s.

Even during the summer season from April to September, there will be six to seven events with wave heights exceeding H_{m0} = 3 m and during construction the breakwater experienced a storm ‘Knud’ with wave heights in the order of H_{m0} = 4.5–5 m.

To construct a breakwater in such a harsh environment required both a strong collaboration between designer and contractor and a robust design that can withstand the forces of nature during both construction and future storms.

- The new western breakwater was designed with a Cubipod armour layer consisting of 15 t and 22 t high-density (2.7 t/m^3) units and up to 32.5 t (3 t/m^3) at the roundhead.
- The Cubipod is a gravity unit developed for both single- and double-layer placement and with a very bulky geometry that allows it to rock and settle without the risk of breakage (unlike more slender interlocking units).
- The contractor (Aarsleff) developed innovative placement methods for the Cubipod, which together with simple placement guidelines from SATO allowed them to place 85 units per 10 h shift.
- The Cubipod armour does not require the same steep and smooth underlay as interlocking units (Accropode, Core-loc and Xbloc).
- Using Accropode guidelines, it would be possible to use a 2.3–5.0 t (M_{50} = 3.6 t) rock underlayer on a slope no flatter than 1:1.5. However, with Cubipods it was possible to use 3–8 t (M_{50} = 5 t) rock on a 1:1.75 slope capable of withstanding typical summer storms without cover.
- Physical model tests performed in Aalborg University showed that the Cubipod armour layer was very stable. In spite of violent breaking triggered by a very steep foreshore, there were no extractions of units, even during the overload testing with K_D = 12.5. The high stability coefficient, K_D, is most likely positively influenced using large filter rocks and having an increased filter layer porosity. The authors suggest that the effect of filter layer porosity on stability should be further analysed in future studies.
- Physical model tests also showed that the Cubipod armour layer had a remarkable ability to self-arrange to cover irregularity or holes and even to self-repair as long as the number of extracted units was limited.
- Placement of the Cubipod was guided by a placement grid prepared by SATO, but the grid itself had status as a guiding tool to make sure that the correct number of units were placed per m^2 (porosity) rather than a requirement. Even when placed out of grid the units had a natural tendency to settle into the recommended diamond grid (between two units in the row below).

Figure 17. Finished expansion of the Port of Hanstholm

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