

TOE BERM DESIGN FOR RUBBLE-MOUND BREAKWATERS: EXAMPLE OF THE SAFI POWER PLANT PROJECT

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DAEWOO E&C (Engineering & Construction) is in charge constructing a new 1320 MW coal-fired power plant located approximately 15 km south-west of the city of Safi in Morocco. ARTELIA Eau & Environnement was appointed by the Contractor to perform the hydraulic design review of the rubble-mound breakwaters protecting the intake and outfall. The toe berm is a key design feature of rubble-mound breakwaters built in breaking conditions, since it helps to support the armour layer and protect the structure from potential scour-induced damage. The initial toe berm design was based on Van Der Meer's empirical formula (1998). Due to the very shallow water conditions, the toe design was verified through physical model tests (2D and 3D) in ARTELIA's hydraulics laboratory located in Pont-de-Claix, near Grenoble (France). The physical model tests demonstrated that the toe berm (6t rocks, 3:1 slope) was not stable at key singular locations, namely roundheads and roots, where direct impacts of breaking waves caused severe damage. Given the site conditions and the construction methods, the usual solutions consisting in increasing the rock size and/or placing the toe berm in a trench had to be ruled out. It was hence decided to reinforce the toe with artificial blocks and to use rectangular concrete blocks with holes. These blocks reduced the anti-stabilizing pressure difference between the top and bottom of the blocks (Tanimoto et al., 1996) and drag force due to the considerable current. They are more usually used at the toe of vertical caissons.

Keywords: rubble-mound breakwater; toe berm stability, very shallow water, physical model tests, Van Der Meer's formula

INTRODUCTION

Project location and background

GDF Suez, Nareva and Mitsui form the Safi Energy Company, which is in charge of building and operating the new Safi power plant. In September 2013, the Office National de L'Electricité et de l'Eau Potable (ONEE - the Moroccan national electricity and water utility) and Safi Energy Company entered into a 30-year "Power Purchase Agreement" for the Safi coal-fired power project. The project includes the construction and operation of a 2 x 693 MW ultra-supercritical coal-fired power plant in the coastal Safi region and the sale of electricity to ONEE for 30 years following completion of the plant. The estimated project cost is US\$1.7 billion.

The plant, located approximately 15 km south-west of the city of Safi in Morocco, is expected to be completed in 2018. The plant is being constructed by Daewoo Engineering & Construction, which has appointed ARTELIA to perform all the numerical and physical model studies for the intake and outfall breakwaters. This paper focuses on one very specific part of the study: the toe berm design for the intake rubble-mound breakwaters.

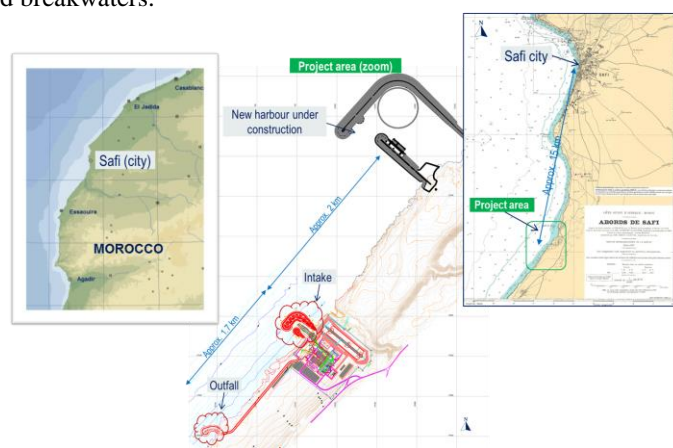


Figure 1. Site location

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Presentation of the intake outer breakwater

The next part of this paper describes the design of the CW intake breakwaters (see figures 2 and 3).

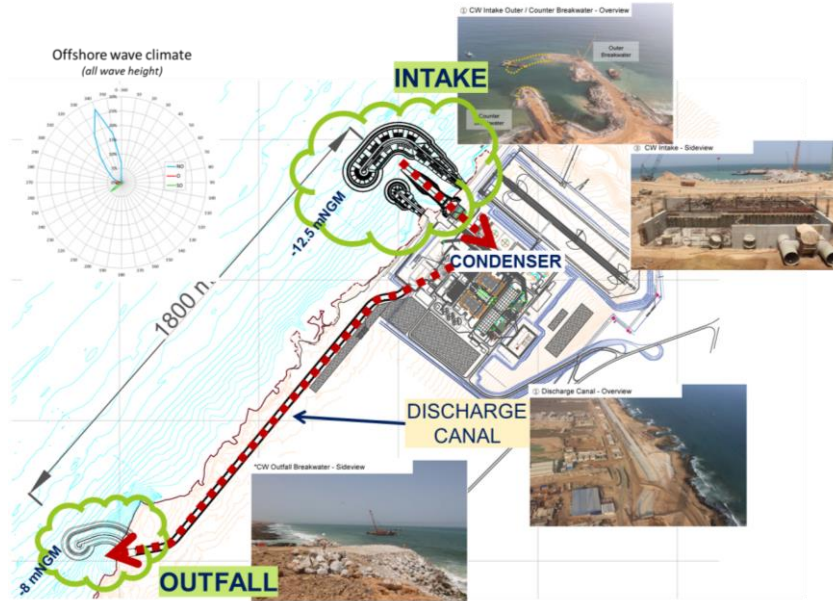


Figure 2. Diagram of Safi power plant operation

The CW intake breakwater system at the power plant has the function of delivering cooling water to the CW intake structure through the CW intake forebay. The CW intake breakwater system is composed of the 470.5m-long CW intake outer breakwater and the 121m-long CW intake counter-breakwater. These structures are rubble-mound breakwaters protected by 6m³, 12m³ and 20m³ concrete ACCROPODE™II units as shown in the figure below. The outer breakwater is located between the coast and -12m NGM. The design water levels defined by the Contractor are as follows: High water level (HWL) = +1.93 m NGM; Low water level (LWL) = -1.97 m NGM. The design wave is the 50-year return period wave.

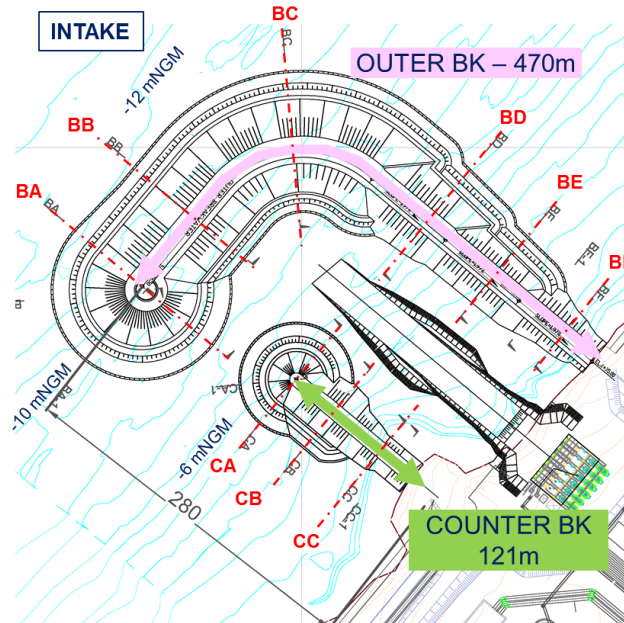


Figure 3. Layout of the CW intake breakwater system

INITIAL DESIGN OF THE OUTER BREAKWATER

The initial design undertaken by DAEWOO E&C was performed using the widely-used empirical Van Der Meer’s formula (1998):

$$\frac{H_s}{\Delta D_{n50}} = \left[2 + 6.2 \left(\frac{h_t}{h} \right)^{2.7} \right] N_{od}^{0.15} \quad (1)$$

Where:

- H_s = significant wave height (m)
- N_{od} = 1.0
- Δ = relative buoyant density of the stone
- D_{n50} = nominal diameter (m)
- h = water depth (m)
- h_t = water depth over the toe bund (m)

Initial toe design – focus on the breakwater roundhead (section BA)

The application of Van Der Meer’s formula for section BA led to a toe berm made of two layers of 6T rocks as shown in the cross-section below (see figure 4).

Section				BA and BB
Water level	WL	mNGM		1.93
Seabed level in front of the breakwater	z	mNGM		-12.5
Water depth in front of the breakwater	h	m		14.4
Toe berm level	Z _{toe}	mNGM		-8.7
Water depth over berm level	ht	m		10.6
Ratio ht/h	ht / h	-		0.74
Statistic Significant wave height	H _{1/3}	m		8.30
formula validity for 0.4 < ht/h < 0.9				0.74
formula validity for 3 < ht/Dn50 < 25				8.6
				Valid
Toe rock nominal diameter	D _{n50}	m		1.24
Armour unit mean weight	M ₅₀	kg		5070

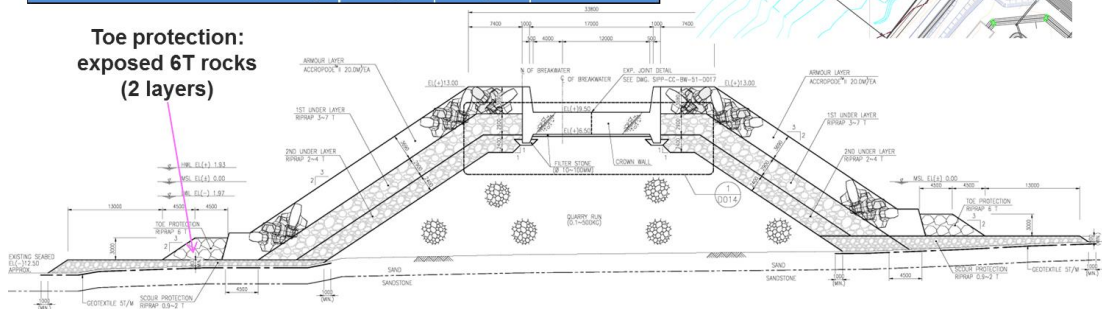
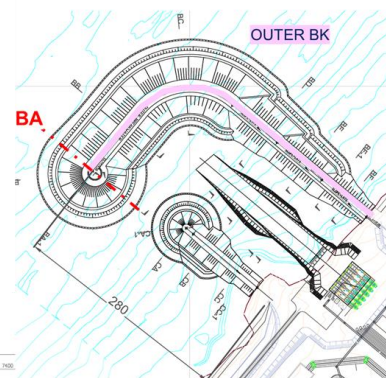


Figure 4. Breakwater roundhead (section BA) – initial design

Initial toe design – focus on the breakwater root (section BE)

DAEWOO E&C chose a similar approach for the breakwater root, where Van Der Meer’s formula was also used. The formula was found to be outside its range of validity since the ratio of ht over h is lower than 0.4.

Van Der Meer’s formula has limited applicability in very shallow waters. The toe design in projects such as the present one needs to be verified through physical model tests.

For the preliminary design, based on our experience of similar breakwaters we proposed to lower the toe into a trench reinforced with 4-6 t rocks. The alternative design was proposed for 2D model testing (see figure 5).

Section				BE
Water level	WL	mNGM		1.93
Seabed level in front of the breakwater	z	mNGM		-2.0
Water depth in front of the breakwater	h	m		3.9
Toe berm level	Z _{toe}	mNGM		0.8
Water depth over berm level	ht	m		1.1
Ratio ht/h	ht / h	-		0.29
Statistic Significant wave height	H _{1/3}	m		5.00
formula validity for 0.4 < ht/h < 0.9				0.29 NOT VALID
Toe rock nominal diameter	D _{n50}	m		
Armour unit mean weight	M ₅₀	kg		

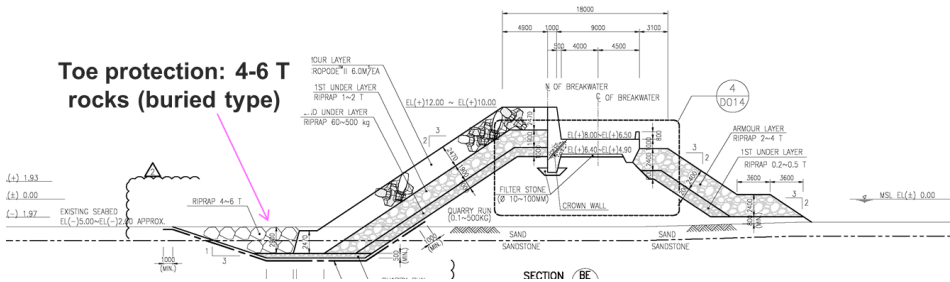
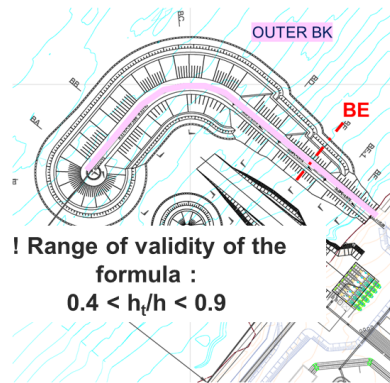


Figure 5. Breakwater root (section BE) – initial design

ASSESSMENT OF HYDRAULIC STABILITY OF THE TOE BY MEANS OF A 2D PHYSICAL MODEL STUDY

Introduction

2D physical model tests were carried out in ARTELIA’s hydraulics laboratory located in Pont-de-Claix, near Grenoble (France). The 2D physical model study was carried out in order to verify the toe berm design for the trunk sections of the breakwaters studied (see Figure 6).

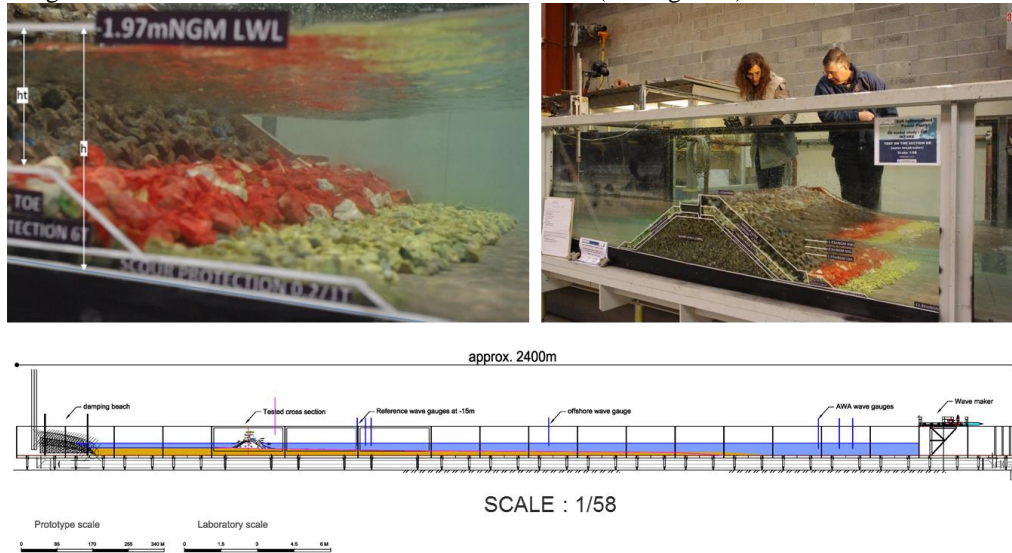


Figure 6. Layout of the 2D model (ARTELIA’s hydraulics laboratory)

Method statement

A series of 10 tests was performed by progressively increasing the wave height (from 1-year wave conditions to the design wave, i.e. a 50-year return period wave). An overload test with 120% of the 50-year return period was performed at the end of each test series, in order to evaluate the margin of safety with respect to the design criteria.

Results for the breakwater roundhead

The following figures show pictures of the section before (on the left) and after (on the right) the test series.



Figure 7. 2D physical model results: section BA before (on the left) and after (on the right) the test series.

At the end of the test series, the armour toe protection made of 6T rocks was flattened out: the percentage of displaced rocks was greater than 3% and the stability criterion was not fulfilled. Moreover, rocks had been displaced upwards to the first row of the ACCROPODE™ II armour layer, which could result in actual breakage of ACCROPODE™ II units (see figure 7). It was hence decided to improve the toe design by flattening the toe profile (from 3:2 to 3:1) as illustrated in figure 8.

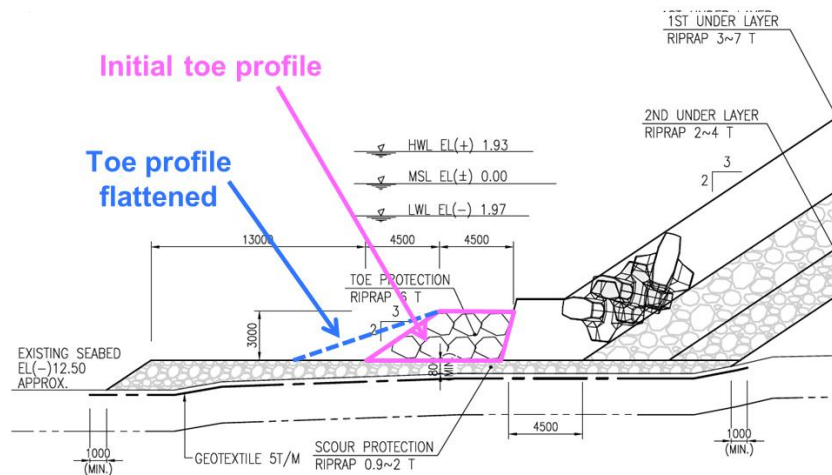


Figure 8. Modification of the toe design to obtain a flatter, more natural profile

Another test series was performed in order to validate the design of the flatter toe profile.

The results showed a percentage of displaced rocks of less than 3%. The stability criterion was hence fulfilled.

ASSESSMENT OF HYDRAULIC STABILITY OF THE TOE BY MEANS OF A 3D PHYSICAL MODEL STUDY

Introduction

As with the 2D physical modelling, the 3D model tests were carried out in ARTELIA’s hydraulics laboratory. The model layout is presented hereafter (figure 9).

In order to reproduce the outfall and the intake at the required scale on the same model, the outfall was located a shorter distance from the intake than in the prototype (namely about 700 m instead of 1700 m), as illustrated in the figure model layout.

This does not affect the study of the structures' stability, as the distance is sufficient to prevent any interference between the structures given the wave conditions reproduced.

The intake pumping and outfall discharge flow rates were reproduced on the 3D model.

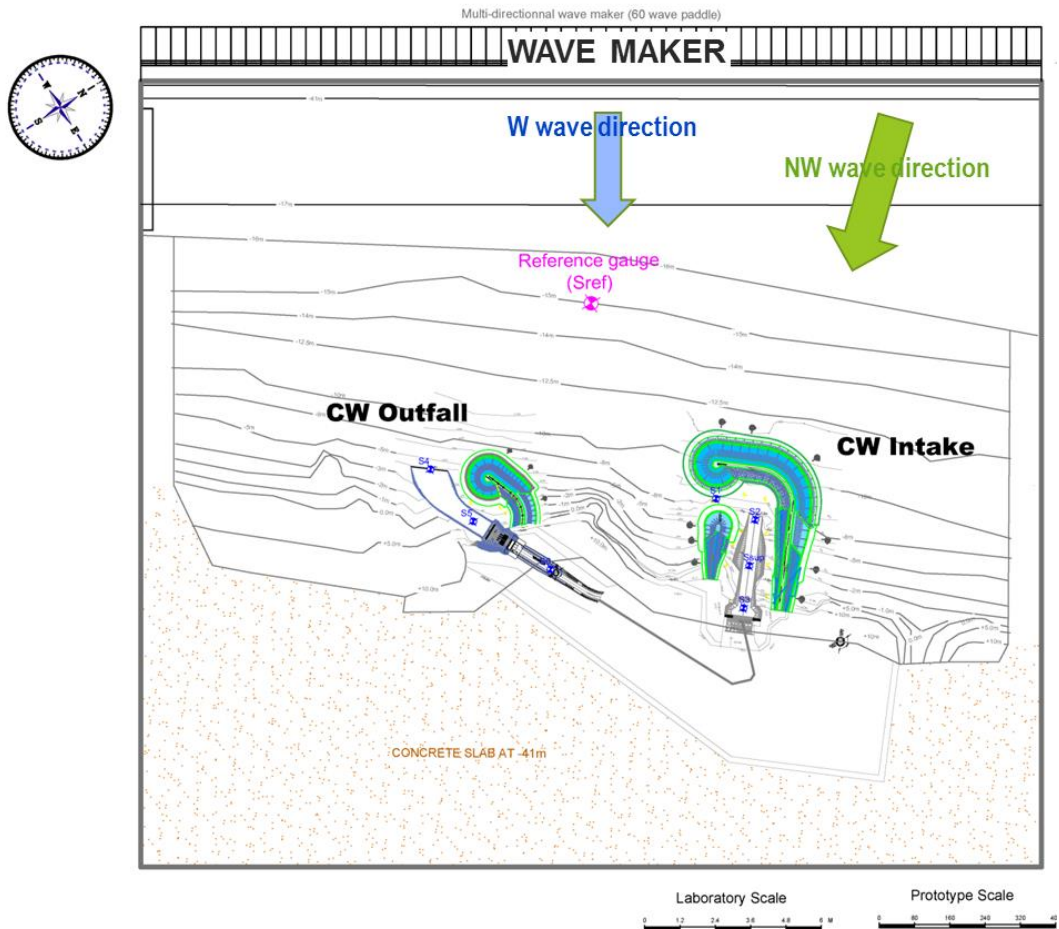


Figure 9. 3D physical model: layout

Method statement

The duration of the tests varied from 3 to 12 hours for the design wave conditions tested, in accordance with the prototype duration of the measured storms. In accordance with the wave propagation study, two main wave sectors were studied: W and NW. One test series corresponded to one incident wave direction.

Each test series consisted of 18 tests with varying wave heights, water levels and wave directions. As with the 2D model, a test with 120% of the 50-year return period design wave (overload test) was performed at the end of each series, in order to assess the risks and the margin of safety with respect to the design criteria.

The 3D tests demonstrated that the toe berm designed at the end of the 2D tests was not stable at key singular locations, namely the roundhead and the root of the breakwater, where direct impacts from breaking waves caused severe damage.

Results on the breakwater roundhead (section BA)

The toe berm made of 6t rocks was washed away (see figure 10) during the design wave conditions (high water level). This occurred due to the impact of waves breaking directly on the toe berm.

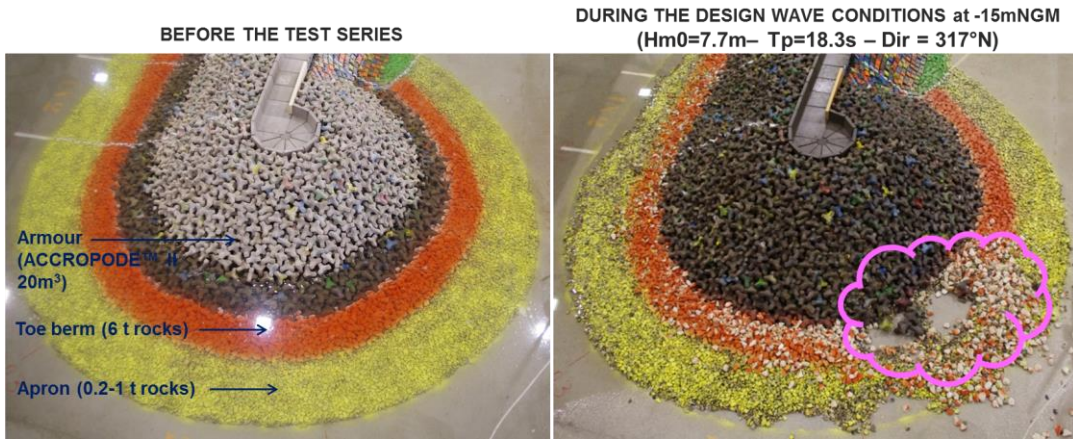


Figure 10. 3D physical model results: roundhead before (on the left) and after (on the right) the test series.

Considering that the project site is permanently exposed to wave attack and in view of the construction method, the usual solution consisting in increasing the rock size and/or placing the toe berm in a trench had to be ruled out.

It was then decided to use rectangular concrete blocks with holes (opening ratio of approximately 10%) in order to reduce the anti-stabilizing pressure difference between the top and bottom of the blocks (see figure 11).

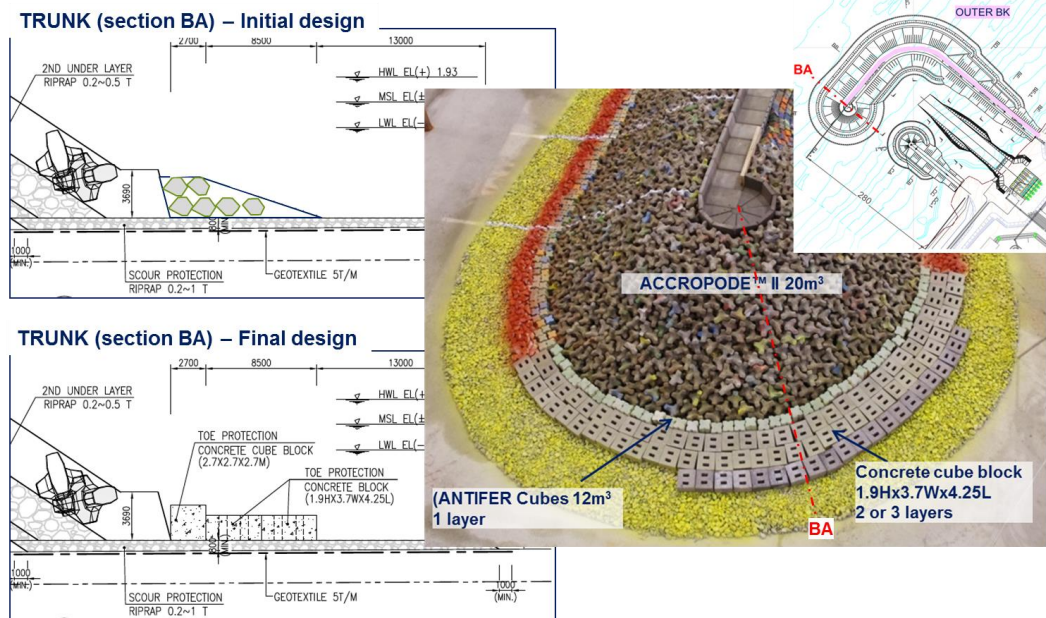


Figure 11. 3D physical model results: modification of the toe design

Another series of tests was performed in order to validate the modified design (see figure 12). The toe berm along the roundhead was observed to be stable at the end of the test series.



Figure 12. 3D physical model results: roundhead before (on the left) and after (on the right) the test series.

Results on the breakwater root (section BE)

This other typical cross-section (section BE) was tested as per section BA. The toe berm, made of 4-6T and 2-4T rocks, was eroded and even washed away in some areas along the root (see figure 13).

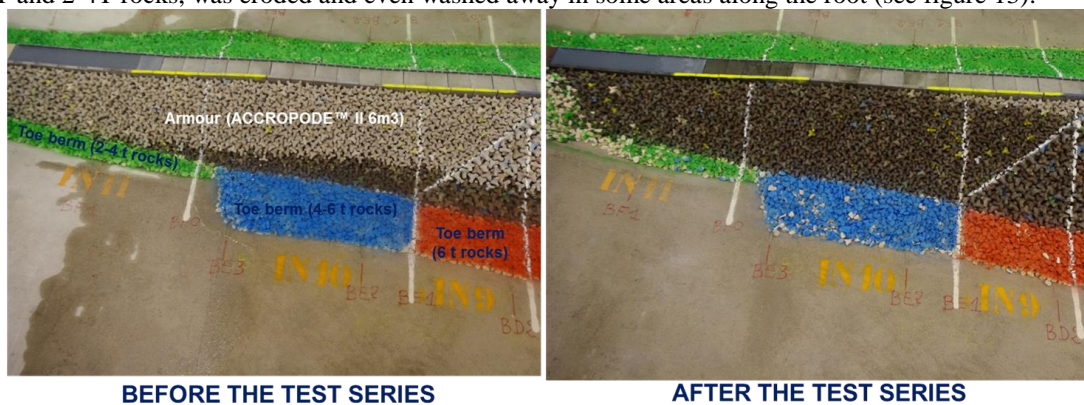


Figure 13. 3D physical model results: root before (on the left) and after (on the right) the test series.

For this part of the breakwater and taking into consideration the construction team's recommendations, it was also decided to use rectangular concrete blocks with holes, of a smaller size than those used for the roundhead.

Another series of tests was performed in order to validate this design.

Figure 14 shows pictures of the section before (on the left) and after (on the right) the test series. The toe berm along the trunk section is stable.

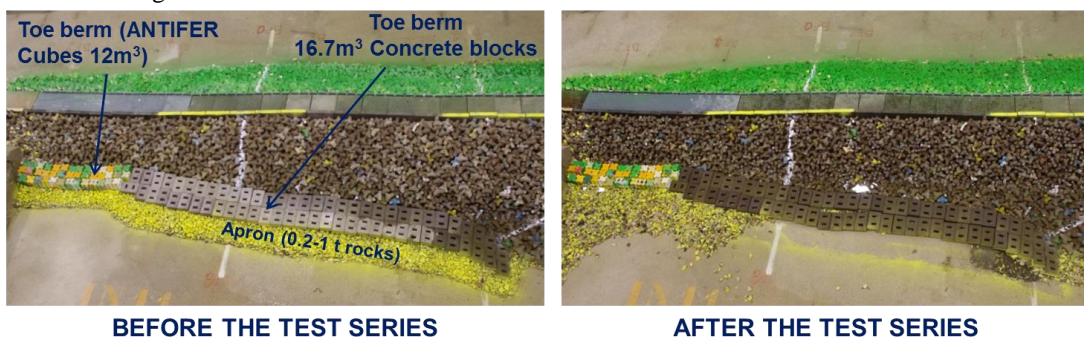


Figure 14. 3D physical model results: root before (on the left) and after (on the right) the test series.

MAIN CHARACTERISTICS OF THE RECTANGULAR BLOCKS WITH HOLES

The main geometrical characteristics of the rectangular concrete blocks with holes that are now being implemented along the roundhead and root of the CW intake outer breakwater are given in figure 15.

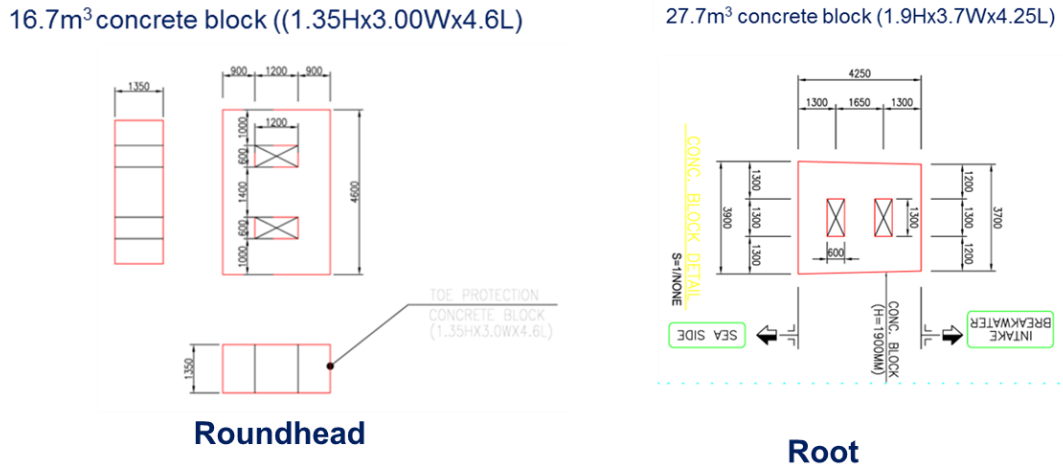


Figure 15. Main characteristics of the toe protection blocks

CONCLUSION

The blocks are currently being placed on the breakwater on site.



Figure 16. Pictures showing the construction and placement of the blocks along the breakwater root

REFERENCES

Aminti, P., Lamberti, A., 1996. Interaction between main armour and toe berm.

Van der Meer, J.W. 1998. Geometrical design of coastal structures, in: Dikes and revetments: Design, maintenance and safety assessment, Pilarczk, K.W. (ed.), Balkema, Rotterdam

Tanimoto, K, K. Furukawa and H. Nakamura (1996): sliding fluid resistance force of upright part of composite type breakwater and model of slide estimation, Proceeding of Coastal Eng, JSCE, Col. 43, pp. 846-850