

## CHAPTER 259

### DESIGN AND CONSTRUCTION OF AN EXTENDED BERM BREAKWATER AT PORT OF HAINA, DOMINICAN REPUBLIC

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#### ABSTRACT

This paper presents the design, model test, and construction to rehabilitate a deteriorated rubblemond breakwater at Port of Haina, south shore of Dominican Republic (Figure 1). The original structure, 350 meter east breakwater and 250 meter west breakwater of rock and concrete block construction, was badly damaged during the storms in 1979 as shown in Figures 2 and 3. After numerous temporary repairs and studies, a rehabilitation design plan was approved. Construction started in June 1993 and is expected to complete by the end of 1995.



Figure 1 Location Map

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## INTRODUCTION

The Port of Haina, a fully developed port complex with a wide spectrum of cargo handling capabilities, is located on the southern coast of the Dominican Republic at the mouth of the Haina River. The harbor is protected against incoming waves by two breakwaters (Figure 2). During the fall of 1979, the site was affected by two hurricanes, Frederick and David, which resulted in considerable damage to the breakwaters. 110 meters of the east breakwater and 65 meters of the west breakwater were essentially destroyed (Figure 3). Since that time, progressive deterioration of the structures warrants a full rehabilitation of the breakwaters. Frederic R. Harris, Inc. was appointed by the Ministry of Public Works of the Dominican Republic to supervise the design review and construction inspection of the breakwaters.

The original contract plan was prepared by TAMS in 1989. During the bid process, the Government of Dominican Republic invites contractors to propose alternative designs leading to lower construction cost and equal or better stability. The winning bid, presented by Conde and Van Oord ACZ (Reference 1), proposed an alternative design featuring extended berm and low, wide crest. This "Extended Berm Breakwater" is different from the conventional "Berm Breakwater" in design concept proposed by Baird et al (Reference 2).



Figure 2 Site Photo

This breakwater design underwent extensive three-dimensional hydraulic model test conducted by Delft Hydraulics and was certified by Danish Hydraulic Institute. The alternative berm breakwater was approved by the Government of Dominican Republic and is being constructed since June 1993.

The construction site is subject to an 8.2 meter significant design wave height with 14 second wave period. The toe of breakwater is located in up to 15 meter water depth. Due to the large design wave, the conventional design by TAMS called for an 80 ton concrete block armor unit. The alternative breakwater by Van Oord ACZ adopted an extended berm design with typical berm width of 20 meter and a wider crest width at 20 meter.

The concept of an extended berm is to pre-break the incident wave further out at the berm elevation. The armor stability is greatly enhanced due to wave breaking on a level surface instead on a slope. The runup height is also reduced by the extended berm. Due to the higher armor stability and lower runup, the alternative berm breakwater is able to adopt a smaller concrete armor unit (40 ton vs 80 ton) and a lower crest elevation (5.0 meter vs 8.2 meter) as shown in Figure 4.

Approximately 30% cost savings are accomplished due to reduced armor and stone sizes and quantities and cheaper handling and transportation costs. By reducing the size of concrete cube, reinforcement is no longer required which simplifies the casting process and increases the production rate.

#### DESIGN CONCEPT

The basic design concept is to respect the geometry and alignment of the existing destroyed breakwater as much as possible in order to reduce excavation and dredging in the old remains. Adopting such a design approach results in non-standard breakwater cross-sections for which no standard design rules have been developed yet. Similar concepts have proven to be economical in other breakwater rehabilitation projects such as Tripoli breakwaters, Libya; Arzew breakwater, Algeria; Ashdod, Israel and Sines, Portugal. In the first two projects a horizontal berm, similar as for Haina had been applied. For Ashdod a dynamic berm has been applied, for Sines a stepped sloping profile was used.

Prior to the development of an alternative design, the causes of damage, state of the damaged breakwaters

and the re-design prepared by TAMS have been studied carefully, as well as the constructibility of the designs. The following discuss background, design approach, and special features of the alternative berm breakwater.



Figure 3 Existing Condition

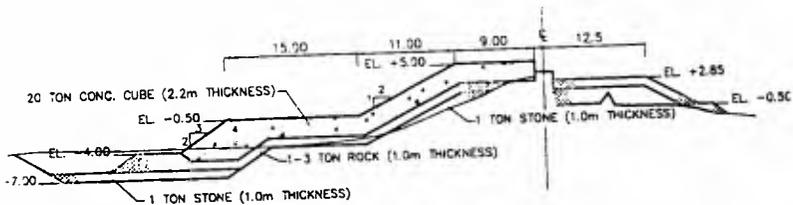


Figure 4 Typical Section

## Background

The existing breakwaters has been severely damaged by tropical hurricanes passing through the site. The condition of the breakwaters is described as severely damaged armor layers, displacement of concrete caps at the east head section, settlement and severe deterioration of concrete cap walls, and severe damage to the harbor side armor due to overtopping waves. Due to these instabilities the existing profiles, especially the east breakwater, show a flattened shape.

The head of the main breakwater (East Breakwater) situated at approximately -10 meter is located at the edge of a reef running down to approximately -40 meter on a slope of 1V:3H. This means that severe plunging almost deep water waves are directly hitting the breakwater head. The boundary and design conditions are summarized as follows:

Water Level Tides are small in the area and due to the large foreshore depth, the effect of wind setup can be neglected. The maximum still water level fluctuations to be considered are between + and -0.50 meter.

Currents Tidal currents are small in the region and are generally less than 1 meter/second.

Waves The maximum significant wave height offshore Port of Haina at a water depth of 40 meter is 8.7 meter, which is equivalent to a 250 year event. The corresponding wave period is about 16 second. The critical design wave direction is south-east.

Subsoil Conditions The subsoil conditions along the breakwater are characterized as cemented sand varying between dense and very dense.

## Design Criteria

The following design criteria have been adopted for the alternative design:

- During the maximum design wave condition ( $H_s=8.7m$ ) only small damage and displacements to the primary armor is accepted, after which the breakwater must remain able to fulfil its function.
- Hydraulic (dynamic) stable, damage criterium of the rock toe due to severe wave conditions is acceptable as long as it remains to fulfil its function of supporting the armor cubes.

- Displacement of armor units into the entrance channel is not allowed.
- For the underlayers and core material the filter requirements should be met such that no material can wash through the upper- and armor layers.
- No breakage of armor units due to severe wave attack is acceptable, so that the use of special shaped armor units (Dolosse, Tetrapods) will not be considered.
- Wave overtopping during moderate storms (recurrence interval of less than 10 years) must be negligible.

### **Extended Berm Breakwater Design**

Since the critical wave direction is from the southeast, the east breakwater was redesigned with a complete new concept of extended berm principle which would reduce the crest height and armor size while at the same time achieving more protection on wave overtopping and wave transmission. The berm has been designed along the following principles based on experiences on the previous designs such as Tripoli and Arzew and previous model tests:

- The horizontal berm elevation will be constant along the length of breakwater except for the head, where it will be raised in such a way that the amount of excavation of the old breakwater can be reduced to a minimum.
- The width of the berm may vary along the breakwater depending on the local water depth in such a way that the requirements for stability and overtopping are met along the total length of the breakwater.
- The armoring of berm consists of concrete cubes with varying weight, depending on the local wave attack, in such a way that the requirements for stability are met.
- The existing crest base will be improved in such a way that the breakwater will be accessible for trucks and/or cars.

After having developed this basic concept of berm breakwater it has been tested and optimized to an acceptable level of confidence in the 3-dimensional model of Delft Hydraulics Laboratory which tests have been inspected and evaluated by the Danish Hydraulic Institute.

### Geometrical Design

In general the berm level is -0.5 meter MSL, however, at the head section the elevation of the berm has been raised to +3.2 meter MSL, in order to reduce the amount of the excavation of the breakwater remains to a minimum. The width of the crest (approximately 70 meter) is such that wave overtopping over the head will still be negligible. The berm width is 25 meter in the deep sections but at shallow sections the berm width has been reduced to 10 meter for economic reasons. The width of the head section is approximately 70 meter.

Depending on the local wave attack the main armoring varies from 20 tons to 40 tons concrete cubes. The 40 tons concrete cubes are used for the most severely attacked sections, in a double layer for the 1:1.5 side slope of the head and in the transition from head to trunk and in a single layer for the toe of the head and the outer part of the crest of the transition. The 20 tons concrete cubes are used for the less severely attacked sections, such as the crest and the rear of the head, the rear and inner part of the crest of the transition, and the front and rear of the outer trunk section.

The toe of the armor layers is constructed in an excavated trench to ensure stability of the cubes at the toe and to support and protect the cubes on the slope against sliding during wave action. The trenched depth at the toe vary from -5.0 meter at the head to -3.0 meter at the trunk.

### MODEL TESTS

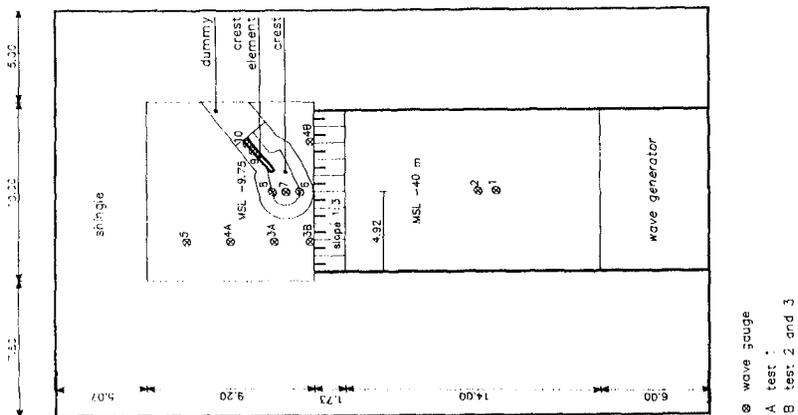
A three dimensional model investigations was carried out at Delft Hydraulics in a wave basin on a linear scale of 1:47. Three test series each consisting of 5 runs of 4 hours prototype were performed applying irregular waves of the JONSWAP type. Due to the exposed position of the west breakwater head particular attention was focused on this part. Especially the breakwater toe was in some occasions directly attacked by the impact of breaking waves. On those locations the toe had to be excavated into the seabed in order to avoid failure of the toe and thus to avoid consequential damage to the armor layer on the 1V:1.5H slope which was supported by the toe. Due to these measures the overall stability remained satisfactory in all test runs up to the maximum significant wave height of 8.7 meter. The following describe more details of the model test.

**Test Facility**

Tests were performed in a wave basin at the De Voorst Laboratory, which is shown in Figure 5. This wave basin has been provided with a random wave generator, capable of performing translatory motions. These motions are realized by means of a hydraulic actuator, programmed by a closed loop servo-system. The command signal of this loop is obtained from a computer disc, representing a random signal with a predetermined wave energy density spectrum. At this facility on line computer facilities are available to enable a direct computation of relevant characteristics. The significant wave height and mean wave period at the location of wave gauges 3, 4, and 5 are summarized in Table 1.

**Table 1 Incident Wave Condition**

test	incident wave		wave gauge 3		wave gauge 4		wave gauge 5	
	Hs [m]	Tm [sec]	Hs [m]	Tm [sec]	Hs [m]	Tm [sec]	Hs [m]	Tm [sec]
1A	3.89	10.5	4.34	9.6	3.57	9.2	2.96	9.3
1B	4.85	11.7	4.55	10.1	3.72	9.5	3.04	8.6
1C	6.06	12.6	4.97	11.0	3.97	10.0	3.10	8.5
1D	7.18	13.0	5.15	11.8	4.11	10.4	3.18	8.4
1E	8.57	13.6	5.66	13.0	4.37	11.5	3.50	9.4
2A	3.12	10.5	3.41	11.0	4.21	10.9	2.73	9.9
2B	4.51	11.3	5.61	11.9	6.68	12.2	3.13	9.2
2C	6.16	12.3	8.33	13.6	9.30	13.2	3.20	8.6
2D	7.27	12.8	9.72	14.0	10.24	13.4	3.29	8.7
2E	8.64	13.3	10.40	13.6	10.00	12.4	3.57	9.5
3A	3.07	10.4	3.50	10.8	4.17	11.0	2.76	9.8
3B	4.55	11.2	5.62	12.0	6.80	12.2	3.14	9.1
3C	6.15	12.4	8.28	13.6	9.37	13.1	3.20	8.6
3D	7.38	12.7	9.66	14.0	10.31	13.2	3.33	8.7
3E	8.60	13.2	10.53	13.8	10.08	12.2	3.60	9.6



**Figure 5 Test Setup**

### Model Scales

Tests have been performed, using a linear scale factor of  $n = 47$ . As gravity forces are predominant in case surface water waves are to be modeled, the Froude number  $v/\sqrt{gD}$  should be identical in model and nature. From this the scales follow for:

$$\begin{aligned} \text{velocity } n_v &= n \\ \text{time } n_t &= n \\ \text{mass } n_m &= n^3 \\ \text{mass density } n &= 1 \end{aligned}$$

For dynamic similarity in rubble-mound stability models the scale factor of the Reynolds number  $vD/\nu$  should also be 1. From these two equations  $n_{\text{Froude}} = n_{\text{Reynolds}} = 1$  it follows that  $n = n^{1.5}$ . Since in all tests water was used and thus  $n = 1$ , perfect dynamic similarity can not be obtained. However by selecting a linear scale factor of sufficient size the effects of viscous forces are minimized.

### Test Program

A total of 3 tests has been performed. Each test consists of 5 runs of 4 hours each. In the first test the amount of excavation was reduced as much as possible by using a conventional toe of 3-6 ton rock on the original seabed level. This test was not satisfactory regarding design conditions. Especially the toe of the head section showed considerable damage, resulting in a great number of displaced cubes. Besides the armor layer of 3-6 ton rock on the rear of the trunk did not remain stable.

To improve the stability, the toe structure and the rear of trunk were changed in the second test. In the second test the toe structure consists of a single layer of 40 ton cubes having a width of 4 blocks, while the top of the toe structure was at the original seabed level. The slope of the rear of the trunk was changed from 1:1.5 to 1:2. The armor layer of the 3-6 ton rock was replaced by 20 ton cubes. The second test was satisfactory.

For the third test the width of the 40 ton toe was reduced to 3 blocks, which also increased the distance from the toe to the steep foreshore. The third test was satisfactory except for the 40 ton blocks were displaced into the entrance channel, for which the second test the toe was stable. Therefore it can be stated that using the positive results of test 2 and test 3 will result in

a satisfactory design. The layout and cross sections of this recommended design are shown in Figures 6 and 7.

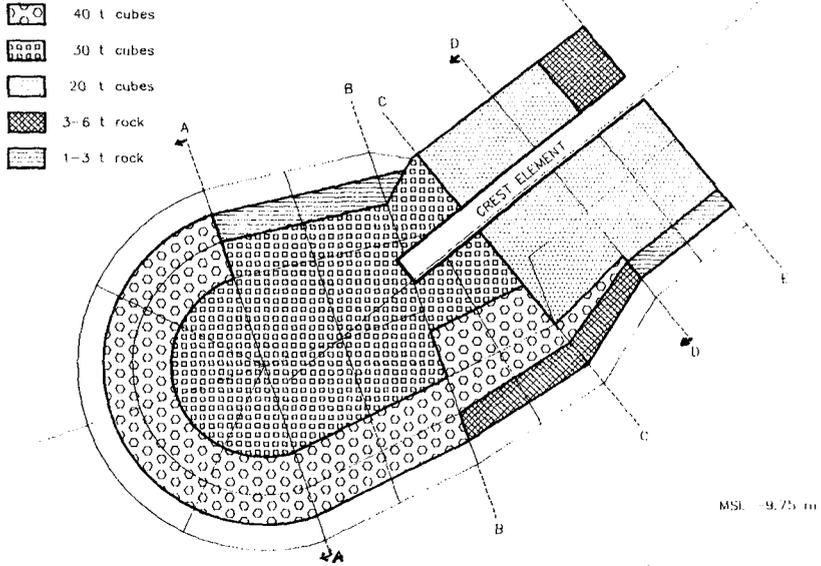


Figure 6 Layout of Recommended Design

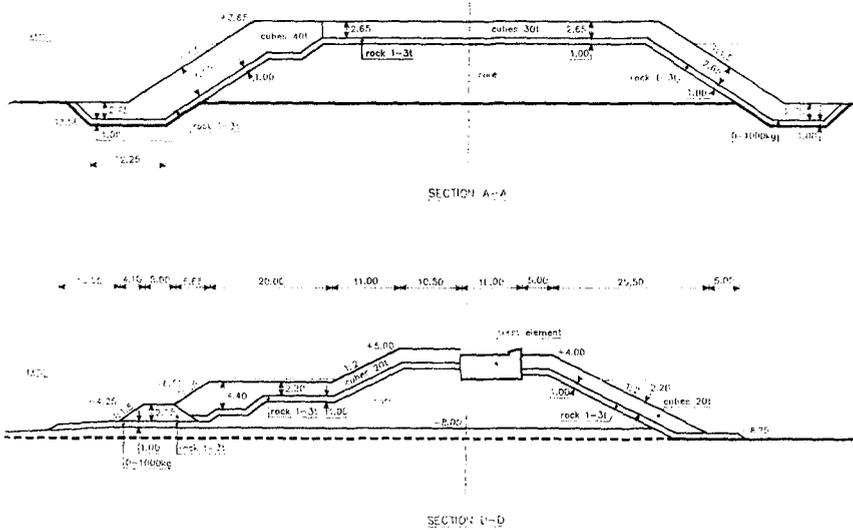


Figure 7 Typical Section of Recommended Design

## CONSTRUCTION

Construction of the breakwater began in June, 1993 with an estimated 18 month construction period. Special features of the construction work include the following:

### **Large quantities of concrete blocks**

Approximately 2,000 units of 40 ton and 6,000 units of 20 ton concrete blocks are used. Due to the large quantity, an on-site mixing plant is set up to deliver mixed concrete with trucks to the casting yard. Due to the limit of space, concrete units are stacked 3 units high. Each casted unit is inspected visually for cracks, spalling and defects. The rejected units are used at non-critical area or substitute for rock armor. The armors are not reinforced. Photos 1 and 2 presents the production of concrete blocks.

### **Quarry Production**

A large quantity of rocks has to be produced and delivered on site to keep up with the pace of construction. The approximate quantities of rock include:

0 - 1 ton of core stone:	101,600 Cubic Meter
1 - 3 ton of underlayer:	65,200 Cubic Meter
3 - 6 ton armor:	10,100 Cubic Meter

The quarry is located approximately 10 kilometer from site. The local quarry are predominantly limestone with good quality and durability in sea water. The average density of stone is 170 pounds per cubic foot. All rocks delivered to site are inspected for integrity and size, rejected armor are broken and used for underlayer or core material. Photo 3 shows typical rock armor used for west breakwater.

### **Toe Excavation and Concrete Block Placement**

Since deep trench has to be excavated for toe, underwater camera and diver are used to ensure the proper construction of the toe. Concrete blocks are placed randomly with specified density. The blocks are casted with slots on both sides and lifted with a special grip developed by the contractor as shown in Photos 4 and 5.

### **Dredging and Excavation**

Approximately 74,000 cubic meters of material are dredged at the toe, channel, and existing breakwater. During

construction of west breakwater, an extra 13,000 cubic meter of soft silt layer at the head and partial trunk section has to be dredged with hydraulic pump to ensure a firm foundation.

### ONCLUSIONS

The alternative design of berm breakwater is able to employ a smaller concrete block and a less toe trenching requirement. The design was aided with a physical model test and has been proved to be effective at various construction sites. Although several design changes were made, the construction is proceeding well and is expected to complete in 1996.

### REFERENCES

1. Van Oord ACZ, Marine Contractors, 2 Jan Blankenweg, P.O. Box 458, 4200 AL Gorinchem-Holland.
2. Baird, W.F. and Hall, K.R., "The Design of Breakwaters Using Quarried Stones", 19th International Conference on Coastal Engineering, Houston, Texas, 1984, pp.1024-1031.



Photo 1 Concrete Plant



**Photo 2 Block Production**



**Photo 3 Block Handling**



Photo 4 Block Placement



Photo 5 Rock Armor