Harbour of Saldanha Bay
Composite type breakwater

Stability of breakwater and fore-shore

report on modelinvestigations

M 1310

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1 Introduction

1.1 Terms of reference

On request of the companies Ballast Nedam and Havenwerken BV, Amsterdam, the Delft Hydraulics Laboratory has carried out some tests on a composite type breakwater for the harbour of Saldanha Bay, South-Africa, as an alternative of the original design of a spending beach. The tests served to establish a preliminary design of a breakwater cross-section, that meets the design-requirements. However, for the final design some additional tests and calculations may be necessary. The tests have been carried out in the 2-meter wide wind-wave flume of the Laboratory at Delft (Fig. 1) in October 1974 by R.J. van Dijk, who also drew up this Report.

1.2 Conclusions and recommendations

1. If a breakwater is built on top of an artificial sand mound, the breakwater may in principle be constructed with materials which can resist attack only from waves that are found in a uniform water depth equal to the remaining water depth in front of the breakwater. The requirement then is, that the dimensions of the sand mound (fore-shore) are such, that the waves in front of the breakwater have reached a state of equilibrium which belongs to that particular water depth.

2. A width of the fore-shore of 75 m at MSL -7 m level is insufficient to provide the required adjustment of the incident waves to the water depth in front of the breakwater. This results into serious damage of the breakwater at wave conditions below the design conditions (tests 1 and 2).

3. A width of a fore-shore of 300 m at MSL -7.5 m to -6 m, connected with a 1:25 slope to "deep" water (at MSL -20 m or -25 m) is sufficient to provide the required adjustment of the wave conditions to a water depth of + 8 m in front of the breakwater (tests 3 and 4).

4. Because of the requirements for the stability of the breakwater and the unpredictable behaviour of the sand in front of the breakwater it is necessary to fix the toe of the fore-shore (300 m from the breakwater) by means of a toe-wall of stones.
5. The stability of the toe-wall is ensured, as long as the sand at the sea-side of the toe-wall is still above the top-level of the toe-wall. According to the results of the tests scouring at the seaside of the toe-wall may be expected, when the sand in front of the toe-wall shows a 1:25 slope down to the initial sea bed.

6. It will not be necessary to apply a sea bed protection of the fore-shore at MSL -7.5 to -6 m, other than the toe-wall and the protection in front of the breakwater (tests 3, 4). This might lead to changes of the fore-shore profile under wave action. The effect of such changes has to be investigated further. The changes have not been reproduced correctly in the model, as the sand was reproduced by stones 10-40 kg in the model.

7. It will be necessary to carry out regular soundings in the vicinity of the toe-wall after completion of the breakwater in order to check the stability of the toe-wall. If this is not possible, it is recommended to apply a protection with stones 0.5-1 t of the 1:25 slope at the seaside of the toe-wall down to the -15 m level.

A toe-wall, constructed as the one applied in tests 1 and 2, does not require such a protection of the 1:25 slope.

8. A protective layer of cubes at the harbour-side slope (test 1) is required with respect to the support of the crest elements. Cubes of 8 t will meet the stability requirements, provided a fore-shore as described before will be applied.

9. During the construction stage of the breakwater no serious damage of the breakwater has to be expected at wave heights below $H_s = 2.7$ m.

10. During the construction stage initial damage is created at $H_s = 2.7$ m. Some of the unprotected core materials at the mean water level are removed. Serious damage occurs at wave heights $H_s$ exceeding 3.3 m.

11. If a crane is used on top of the core during the construction it should be removed from the unprotected part of the core at $H_s > 2.5$ m and it should be removed from the breakwater at $H_s > 3.0$ m.

12. Serious damage over a large length of the breakwater during the construction can be avoided by completing the breakwater as soon as possible after the construction of the core. This completion should then be carried out including the placing of the crest elements.
2 Design of the breakwater

The aim of the investigations was to assess a design for a composite-type of breakwater consisting of a sand mound, from the original sea bed (at MSL -20 m) till MSL -6 m, and a rubble mound superstructure above the MSL -6 m level.

The sand mound had to be designed in such a way, that wave attack on the rubble superstructure is limited by the local depth in front of the rubble mound.

This limitation of wave attack can be obtained by applying a horizontal fore-shore at the sea side of the breakwater, provided that the width will be big enough. The width of the fore-shore had to be determined during the model tests.

From both field and laboratory measurements it is known that the maximum significant wave height \( H_s \) is limited by the local water depth \( d \) according to: \( H_s (\text{max}) = 0.4 \times d \), provided that the wave conditions have reached a state of equilibrium. The relation is not valid for sudden reductions of depths or steep bed slopes.

Assuming that the wave conditions near the rubble mound are limited by the local depth (after a suitable design of the fore-shore) the maximum wave height near the rubble mound will be:

\[
H_s = 3.0 - 3.75 \text{ m for } d_{\text{min}} = (6 + 1.5) = 7.5 \text{ m},
\]

\[
H_s = 3.6 - 4.5 \text{ m for } d_{\text{max}} = (7.5 + 1.5) = 9 \text{ m},
\]

in which it is assumed that the fore-shore shows a gentle slope (not steeper than 1:40) from -6 to -7.5 m.

For the tentative design calculations of the rubble mound \( H_s = 4.0 \text{ m} \) has been chosen.

The weight of armour units can be approximated with the use of Hudson's stability formula:

\[
W = \frac{\gamma H^3}{K_D \cdot \Delta^3 \cdot \cotg \alpha}
\]

in which: \( W = \) weight of the single armour unit (tf)

\( \gamma = \) specific weight of the armour units (tf/m\(^3\))

\( H = \) wave height (m)
\[ K_D = \text{factor, depending on type of armour units, etc.} \]
\[ \Delta = \frac{y - y_w}{y_w} = \text{specific weight of armour unit under water in relation to the specific weight of water} \]
\[ \alpha = \text{breakwater slope.} \]

Generally speaking the wave height which is applied in this formula will be the wave height at which initial damage (about 1%) is to be expected.

Some generally accepted values of \( K_D \):
- Quarry stones: \( K_D = 3 - 4 \)
- Cubes: \( K_D = 7 - 8 \)
- Tetrapods: \( K_D = 7.5 - 8.5 \).

If concrete cubes are applied at the seaside slope the required unit weight will be: \( (\gamma = 2.3 \text{ t/m}^3; K_D = 7.5; \cotg \alpha = 1.5) \)

\[ w = \frac{2.3 \times 4^3}{7.5 \times 1.275^3 \times 1.5} = 6.3 \text{ t} \]

During the model tests cubes of 8 t have been applied; the design of the breakwater is shown on Figures 4 and 5.

In comparison with the design of the armour-layers the design of the core is somewhat less critical. Instead of stones 0.5-2 t as used during the tests one could apply quarry run covered with a layer of stones 0.5-2 t.

In this case protection of the quarry run of the core should be effected as soon as possible, in order to reduce damage to the core.
3 Arrangements of the model and execution of the model tests

3.1 Model setup

The situation of the model in the 2 m wide wind-wave flume is shown on Figure 3. The flume was separated into three smaller flumes; in the centremost the model was placed and in the other two the incident waves were recorded. The sea bed in front of the breakwater was situated at MSL -20 m (tests 1 and 2) and MSL -25 m (tests 3, 4 and 5).

The length scale of the model, \( n_L \), was 1:50; accordingly the time scale and the velocity scale (\( n_T \) and \( n_v \)) were \( 1: \sqrt[50]{50} = 1:7.07 \).

3.2 Wave generation and wave registration

The waves, applied during the model tests, were generated by a programmed wave generator (Fig. 2) which is able to generate irregular sea waves with the same statistical properties as can be expected in prototype conditions.

During the tests of the completed breakwater the wave conditions have been registrated at two places at the -20 m (-25 m) level (see Figure 3). In this way a relation between the incident wave height at "deep" water and the corresponding damage of the breakwater has been obtained.

The waves are characterized by the wave-energy spectrum and the wave-height distribution, examples of which have been shown on Figures 6 and 7. The energy spectrum is characterized by the period with maximum energy density (\( T_0 \)). The wave-height distribution is characterized by the so called significant wave height (\( H_s \)). In the tests the wave height which is exceeded by 15% of the number of waves has been used as an approximation for \( H_s \).

3.3 Execution of the model tests

The design condition of the breakwater has been accepted to be \( H_s = 8 \) m and \( T_0 = 16.6 \) s.
During the tests the wave height $H_8$ was increased in 4 or 5 steps from a low value, at which no damage was expected, till a condition equal to or superior to the design condition. During each step the wave period $T_o$ was chosen in accordance with the wave height.

Each step lasted 1.5 hours corresponding to about 10 hours in the prototype, which is considered to be the average maximum duration of a severe storm. During the steps the damage was not repaired, and so a relation was obtained between the wave heights and the corresponding increase of damage.

The damage after each step was registered by means of photographs and a description, from which the reports of damage (Tables 1-5) have been derived.
4 Results of the model tests

4.1 Stability tests of the completed breakwater (tests 1-4)

Figures 4 and 5 show the various breakwater sections, which have been tested. The wave heights have been measured at the MSL -20 m level (tests 1 and 2) and at the MSL -25 m level (tests 3 and 4).

Test 1

In test 1 the preliminary design of a breakwater protected by a 75 m fore-shore at MSL -6 till MSL -7.5 (slope 1:40, covered with stones 0.3-1 t) was tested (see Figure 4). Below the end of the protected fore-shore a toe-wall of stones 2-4 t was built.

A slope of 1:6 from MSL -7.5 m to MSL -15 m represents a situation after appreciable scouring by waves and currents has taken place. The ultimate bottom profile was modelled with stones of 10-40 kg, the smallest size of materials which could be applied during the model tests.

The results of this test are shown on Table 1. Already at the first step ($H = 4.75$ m) all the crest elements have been removed due to lack of support of these elements against horizontal wave forces. Cubes of the seaside slope have been removed as well.

The stones of 10-40 kg at the sea side of the toe-wall showed appreciable movement and some erosion and transport onto the 1:40 slope.

Test 2

In test 2 an improved breakwater section (protected by cubes on both slopes) and the same 75 m fore-shore at -6 till -7.5 m was tested (Fig. 4).

The results of the test have been shown on Table 2.

The crest elements show only little movement, but a lot of damage at the protective layers of cubes has been produced, although the maximum wave conditions at deep water are still below the design condition.

In front of the toe-wall most of the 10-40 kg stones were removed till MSL -12 m level; a lot of stones 0.3-1 t from the top of the toe-wall at MSL -7.5 m have been removed as well. The breakwater was covered with stones 10-40 kg and 0.3-1 t.
From this test it can be concluded that a fore-shore of only 75 m at MSL -7 m level is not capable to reduce the wave attack at the location of the breakwater to the one that corresponds to the attack that is found at a water depth of + 8 m.

Test 3

In test 3 the width of the fore-shore at MSL -6 to -7.5 m was 300 m, the slope being 1:200 (Fig. 5). In order to fix the toe of the structure at the MSL -7.5 m level a toe-wall of stones 2-4 t down till MSL -10 m was built. One preferred not to protect the whole length of 300 m fore-shore, but in order to ensure the stability of the first part of the fore-shore, a stretch of 50 m, starting from the toe-wall, was protected by stones 0.5-1 t.

In front of the toe-wall the sea bed was lowered (with a 1:25 slope) to the MSL -25 m level. This will represent a situation in which severe scour has taken place already, as at present in prototype a sea bed of +600 m width at a depth of about MSL -5 m has been constructed already. The sea bed was again modelled with stones 10-40 kg.

Table 3 shows the report of damage during this test. Especially in front of the toe-wall a lot of stones 10-40 kg have been removed, all these materials are deposited on the toe-wall. The stones 0.5-2 t at the MSL -7.5 m level and the stones 10-40 kg of the fore-shore show no damage.

The damage of the breakwater section (10 cubes removed from harbour side slope) during the design conditions can be considered to be acceptable.

Test 4

The section has been shown in Figure 5. On the basis of the results of test 3 it was decided that an additional test should be carried out in which no protection of the fore-shore, excluding the toe-wall at the MSL -7.5 m level, would be applied. The widths of the berms of the breakwater were reduced from 7 m to 4 m.

The results of the test have been shown on Table 4.

The damage of the breakwater at the design condition is still acceptable, especially if one takes into account that the packing of the cubes on the section was carried out on purpose in a very loose way.

Much damage occurs at the seaside of the toe-wall: a hole with a maximum
depth of about 3 m has been scoured, while all the materials were deposited on top of the toe-wall.

4.2 Test of breakwater during the construction stage (test 5)

One test of the breakwater during the construction stage has been carried out. From previous investigations it is known, that the part of the breakwater which is most liable to wave attack is the part at a level corresponding to the still water level. Table 5 shows the section which was tested. The core has been completed, and part of the protective layers of cubes has been placed. A schematically reproduced crane (with about the right weight and the right area exposed to wave attack) has been placed on top of the core. During this test the wave height was measured in front of the breakwater at the -7 m level.

Initial damage was produced at $H_s = 2.5$ m. At $H_s = 2.75$ m some cubes were removed from the top of the seaside layer and the core started to be flattened out. The crane was removed (turned over) because of settling and removing of stones.

The damage at the seaside layer of cubes and the flattening of the core continues when the wave height is increased to 3.3 m and 3.7 m. The crane was replaced on the core behind the seaside layer of cubes, and remained on the breakwater at $H_s = 3.3$ m, although some cubes hit the crane. At $H_s = 3.7$ m the crane was removed by the overtopping waves.

From this test it can be concluded, that no serious damage of the breakwater during the construction has to be expected at wave heights below $H_s = 2.7$ m. The crane should not remain on the crest of the core (unprotected part) at $H_s > 2.5$ m and be removed from the breakwater at $H_s > 3.0$ m.

It is clear that serious damage over a large length of the breakwater during the construction can be avoided by completing the breakwater as soon as possible after the construction of the core. This completion should then be carried out including the placing of the crest elements.

It should be emphasized that this test has been carried out with a two-dimensional section. Some differences in the prototype can be found, especially in the vicinity of the end of the partially completed breakwater (effect of the breakwater head) as 3-dimensional phenomena will play a role there.
SITUATION OF THE MODEL IN THE 2m WINDWAVE FLUME

DELFT HYDRAULICS LABORATORY
WAVE ENERGY SPECTRA

EXAMPLES AS MEASURED DURING THE MODEL TESTS

DELFTHYDRAULICS LABORATORY
WAVE HEIGHT DISTRIBUTIONS

EXAMPLES AS MEASURED DURING THE MODEL TESTS

DELFt HYDRAULICS LABORATORY

M1310 FIG. 7
**Significant Wave Height**

\[ H_s \ (m) \]

---

**Damage**

**Test 1**

---

**Before**

---

**After**

\[ H_b = 5\, m \]

---

**Report of Damage**

**Test 1 - Table 1**

---

**Code of Damage**

- Stones or cubes of first layer removed
- Stones or cubes of both layers removed; no protection left for underlying layers
- Place where one can find the removed cubes
- CREST ELEMENTS
- STONES
- STONES 10 - 40 lb
- Movements of stones or cubes
- SETTLING OF STONES OR CUBES
- GAP FORMED DUE TO SETTLING
**REPORT OF DAMAGE**

**TEST 3**

**SECTION**

To 4.15

- **No Damage**
- Stones 10-40 tons on the 1-4 ft top wall
- Stones 0.5-21 on 1:5 slope
- Stones up to 40 tons on the 1-4 ft top wall
- Movement of stones on 1:5 slope

To 4.90

- **No Damage**
- Stones 10-40 tons on the 1-4 ft top wall
- Stones 0.5-21 on 1:5 slope
- Stones up to 40 tons on the 1-4 ft top wall
- Movement of stones on 1:5 slope

To 5.08

- **No Damage**
- Stones 10-40 tons on the 1-4 ft top wall
- Stones 0.5-21 on 1:5 slope
- Many stones 10-40 tons on the 1-4 ft top wall
- Displacements of stones on 1:5 slope

To 7.50

- **No Damage**
- Stones 10-40 tons on the 1-4 ft top wall
- Stones 0.5-21 on 1:5 slope
- Many stones 10-40 tons on the 1-4 ft top wall
- Removed from 1:5 slope

To 8.15

- **No Damage**
- Stones 10-40 tons on the 1-4 ft top wall
- Stones 0.5-21 on 1:5 slope
- Many stones 10-40 tons on the 1-4 ft top wall
- Removed from 1:5 slope

Code of Damage: See Table 1

**TABLE 3**
### Significant Wave Height

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.90</td>
<td>No damage occasional overtopping of the core core at +3.39 m</td>
</tr>
<tr>
<td>2.15</td>
<td>Slight settling of core materials at seaside slope some more waves are overtopping the crest of the core; some waves hit the crane</td>
</tr>
<tr>
<td>2.75</td>
<td>The overtopping waves cause a flattening of the crest of the core; the crest level is lowered to +1.50 some cubes of the top of the sea side layer have been removed onto the crest the crane has been removed from the crest of the core because of setting of the core (not because of wave forces the crane) the crane has been replaced behind the cubes on the crest the upper part of the cubes of the sea side layer has been removed; some cubes hit the crane; some cubes removed to harbourside layer and harbour side berm. Many stones of the crest of the core have been removed; the top level is lowered to +2.00 m</td>
</tr>
<tr>
<td>3.30</td>
<td>The crest level has been lowered to the water level (at +5.00 m); many stones removed to harbourside berm many cubes removed from top of sea side layer; cubes can be found on top of harbourside layer and at harbourside berm some cubes removed from top of harbourside layer the crane has been removed on overtopping waves</td>
</tr>
<tr>
<td>3.70</td>
<td></td>
</tr>
</tbody>
</table>

**Code of Damage: See Table 5**
After $H_s = 4.75$ m

Test 1
Before Test 2

direction of waves

After $H_s = 5.95$ m

Test 2
After $H_s = 6.0 \text{ m}; T_o = 15.1 \text{ s}$

direction of waves

After $H_s = 8.15 \text{ m}; T_o = 16.6 \text{ s}$

Test 4