SPENDING BEACH BREAKWATER AT SALDANHA BAY.

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## 1. ABSTRACT.

This paper describes the design and construction of a sand breakwater at Saldanha Bay exposed to the severe swell conditions of the south Atlantic.

The design of this spending beach breakwater was aimed to establish a seaward profile with sufficient "playroom" to incorporate profiles resulting from extreme wave conditions.
As far as the construction is concerned the emphasis is put on the sandlosses during execution of the works.
Finally the designed profile is compared with the actual established beach profile.

## 2. INTRODUCTION.

Saldanha Bay is situated some 110 km North West of Cape Town, South-Africa, on the Atlantic Coast. The bay has a wide entrance between protruding peninsulas which makes it one of the best natural harbours of the southern part of the African Continent. Through the centuries the bay has been used as a harbour of refuge by passing explorers and East Indiamen. Had it not been for the lack of fresh water, Saldanha Bay would have been developed as a major port long ago. Because of its proximity to deep water and the relatively few engineering works required, Saldanha Bay was chosen to be developed as harbour terminal of the iron ore export scheme, undertaken by the South African Iron & Steel Industrial Corporation Ltd. (Iscor).

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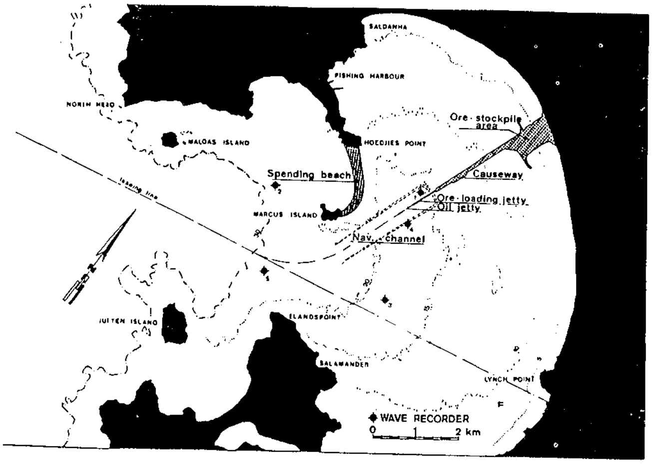
In order to enlarge the existing sheltered area of the inner bay, the northern peninsula Hoedjies Point was extended by a breakwater to Marcus Island situated in the middle of the entrance to Saldanha Bay (see illustration 1).
This breakwater was designed and constructed as an artificial sand beach on which the waves would spill and thus gradually dissipate their energy, hence the name "spending beach breakwater". This article describes the design and construction of the "spending beach".




ill. 1. General lay out of harbour configuration at Saldanha Bay.

## 3. BASIC INFORMATION.

### 3.1. Waves.

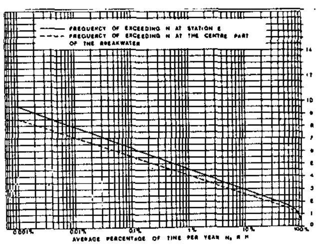
During a period of two years, preceeding the design period, wave heights were measured at five different locations in the bay (ill. 1). Simultaneously "deep sea" wave heights, directions and periods were measured. The measurements were executed by the Council of Scientific and Industrial Research (C.S.I.R.).

At measuring station 2 situated just seaward of the proposed breakwater, at a depth of approximately 25,00 m below C.D. the results show the following:

| Table nr. 1 | Station 2 | | | |
|---|---|---|---|---|
| Frequency exceeded % | $H_s$ m | $\bar{H}_s > H_s$ m | $T_s$ sec. | $\bar{T}_s$ sec. |
| 50 | 1,75 | - | 13,4 | - |
| 10 | 2,90 | 3,70 | | 15,3 |
| 5 | 3,40 | 4,10 | | 15,7 |
| 1 | 4,50 | 5,15 | | 16,6 |

The maximum significant wave height recorded at station 2, was $H_s$ = 4,90 m, corresponding with a deep sea wave height (significant) of $H_s$ = 8,50 m. The wave height probability curves given in illustration 2 show the frequency of exceeding of $H_s$ at station 2 and the centre part of the breakwater.

The predominant deep sea wave directions are south-south-west and south-west. Measurements obtained by means of clinometer-readings show SSW 43,9 % of the time, SW 37,0 % and S 10,1 %.



ill. 2. Wave height probability curves in station 2 and in the centre part of the breakwater.

Due to the configuration of the entrance of the outer bay, the combined effect of refraction and diffraction, forces the incoming deep sea swell to pass Marcus-Island with only a slight variation in wave direction.

A diffraction-refraction model built to acquire an insight into the wave reduction in the lee of the proposed breakwater, was also used to obtain photographs of the wave pattern of the area between Marcus Island and Hoedjies Point peninsula. Photographs were taken for different deep sea wave directions and periods. Aerial photographs taken of the actual wave crest patterns of know deep sea conditions, showed good correlation with the corresponding photographs of the model tests. An example hereof is given in illustration 3.

### 3.2. Wind.

Wind speed and direction were measured at Elandspoint, situated due south of Marcus Island, during the period March 1971 to February 1974.

The SSW-direction is the predominant wind direction with an average wind speed of 7 m/sec. The average windspeed during spring is 8,5 m/sec.

### 3.3. Tides.

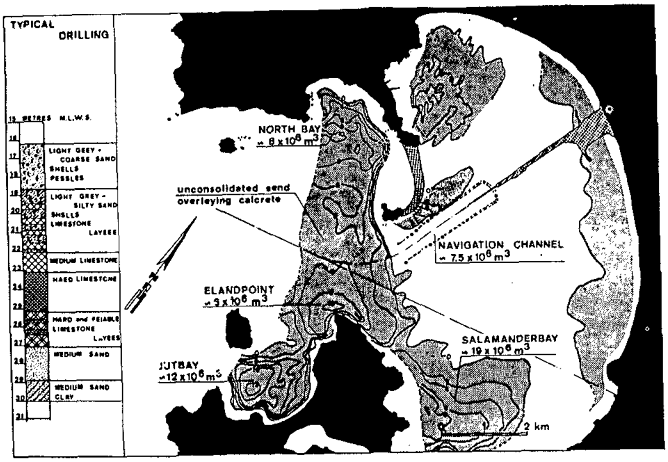
The tides are as follows: MHWS : 1,76 / MHWN : 1,26 m / MLWN : 0,76 m / MLWS : 0,26 m ; all related to C.D.

### 3.4. Currents.

As only a few current measurements were made, only approximate values of currents could be established. Currents at a depth of 10 m in the north channel went up to 0,15 m/sec. during ebb-tide and to 0,24 m/sec. during flood-tide. The direction of the tidal currents was almost perpendicular to the line Hoedjies Point - Marcus Island. Wave induced currents could not be distinguished.

### 3.5. Geology of the sea-bed.

A seismic survey related to some borings in the area of the proposed breakwater showed that the bottom generally consists of compacted sands on top and in between layers of calcareous material varying in hardness. At the lee side (north-east) of Marcus Island some loose sand layers could be distinguished. At some places (Blinkklip Rock) granite outcrops stick through the bottom layers. The shoreline of both Hoedjies Point and Marcus Island consists of huge outcrops of solid granite and boulders.




ill. 3. Loose sediments overlaying calcareous material, sandwinning areas.

### 3.6. Available building material.

As millions of m$^3$ of sand were required to build the spending beach breakwater, intensive soil investigations were carried out, comprising several sonar surveys, drilling/probing using a self elevating platform and a shallow water structure placed on the bottom, also washborings were made using a floating platform.

In the Saldanha Bay area the basement is formed by the so called "Cape Granite". Drilling results showed that the buried granite surface is highly irregular. In this basement the valleys are filled in with recent and subrecent material, gently sloping down in the seaward direction. The infill material consists mainly of loose and compacted sands with layers of cemented, sand and calcrete, differing in hardness.

The areas where loose sediment is overlaying calcareous layers, are indicated on illustration 3, being the outcome of an extensive sonar survey. As main sandwinning areas could now be distinguished:
- Salamander bay area of which approximately 19 x $10^6$ sand could be obtained and the
- navigation channel, with a total quantity of 7,5 x $10^6$ $m^3$ including large quantities of cemented sands and calcareous materials.

A typical drilling of the area of the navigation channel is also given in illustration 3. The medium grain size diameter of the numereous samples varied considerably. The Salamander bay area showed some small areas of decomposed granite (600 μ) excellently suitable as building material for the breakwater.

## 4. DESIGN.

### 4.1. General.

In an early stage of the design, after discussion with the Delft Hydraulic Laboratory, it was decided not to execute an extensive model study as this could not give sufficient reliable results within the available time, but to establish a design based on the knowledge obtained from literature and on the measurements of comparable existing beaches. In the design two main items can be distinguished:
a. the horizontal alignement and
b. the seaward profile.

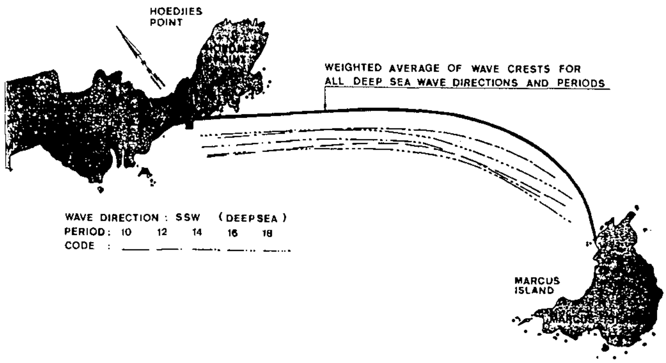
The horizontal alignement of a beach in general, must be designed in such a way that the predominant wave direction will always impinge perpendicular to the beach in order to prevent lateral transport of material.
It shall be clear from the geography of the bay that the location of Marcus Island in relation to Hoedjies Point offered a unigue possibility to build an artificial cove as a natural extension of North Bay beach (see illustration 1).
As other kidney-shaped beaches exist in the area, it was quite a challenge to try to design an artificial beach which could withstand the incoming ocean swell having its origin in the "roaring forties".

The seaward profile in general depends on the size of available sediment and the local wave conditions. Since wave conditions vary continuously, a single stable equilibrium profile cannot exist. The eguilibrium state will thus be of dynamic nature, continuously adapting itself to the momentary wave climate.

The design was aimed to establish a seaward profile, with sufficient "playroom" to incorporate profiles resulting from extreme wave conditions.

4.2. Horizontal alignment.

For different deep sea wave directions and wave periods, the wave crest patterns between Hoedjies Point and Marcus Island, as observed in the model test, are shown in illustration 4. The low-water line of the proposed breakwater was determined parallel to a fictive wave crest being the weighted average of the different wave directions and periods.
Taking into account the extended low water line of North Bay beach and the location of Marcus Island, the low water line of the proposed breakwater was now fixed, reflecting the kidney-shape like so many beaches in the area.
The width of the beach was determined by two factors. Firstly by the variation of the low water line being dependent on the wave direction and secondly by the variation of the seaward profile being dependent on the wave height.




ill. 4. Wave crest patterns according to model studies.

When a wave approaches the beach at an angle a longshore transport is generated. This breaking wave and the movement of sediment is caused by the component of the wave in the longshore direction and the longshore current generated by the breaking wave. Tidal currents in the area of the proposed breakwater could be considered negligible.
The longshore transport, being dependent on the wave energy and the angle of wave approach, should gradually decrease and finally come to a stop as the shoreline is shaped into line with the incoming wave-crests. Studies of a.o. Bascom, Inman, Rusnak showed large variations in beach-profile and large areal movements of sand in relative short periods of time. It was therefore assumed that the horizontal alignment should adapt itself rather rapidly to a change in wave direction.

The width of the beach (shore) should thus be able to "accomodate" all the low water lines corresponding with the different deep sea wave directions. For other kidney shaped beaches in the area, although no beach measurements were taken over long periods, the verbal opinion of local fishermen indicated only minor changes in the low water line.

### 4.3. Seaward profile.

#### 4.3.1. General.

The design problem was approached as being two-dimensional, i.e. the design of a cross-section which remains in equilibrium when subjected to local wave conditions, consistent with size and amount of available material.
This two-dimensional approach was considered acceptable due to the fact that tidal currents after the closure of the gap between Hoedjies Point and Marcus Island should be negligible, while generated longshore transport should always fade out the source of its origin, i.e. decrease the angle of wave approach by re-shaping the beach.

The main profile changes are limited to be inshore zone. A seaward profile is considered stable when in the long run the offshore transport equals the onshore transport of material. As the waves tend to change periodically, the profile adapts this change and in general a "summer" or step profile and a "winter" or bar profile can be distinguished. Under rough weather conditions a "winter" profile is formed with a shore line which may move landwards over a considerable distance. Large quantities of sand are then transported in the direction of the sea, where they are "stockpiled" in the form of one or more sand ridges or "bars". In calme weather the waves will gradually shift this sand back towards the shore.
There is accumulation of coarse sediment in zones of maximum wave energy dissipation and deposition of fine sediment in areas sheltered from wave action. This phenomenon is called sediment sorting.

#### 4.3.2. The offshore region.

Waves travelling towards the projected breakwater, eventually reach a depth where the water motion near the bottom begins to effect the sediment on the bottom.
The water motion immediately above the sediment bed then exerts enough shear to move sand particles. This depth is called the incipient depth and is determined by the sediment size and wave characteristics. Initially only very fine material oscillates in the direction of the wave and as the water depth decreases coarser sand particles will be moved back and forth in ripples.

In order to determine the incipient depth it is necessary to calculate both the orbital velocity immediately above the seabed and the threshold velocity at which erosion of the bed material takes place. The orbital velocity is calculated as follows:

$$U_z = 0{,}7\,\frac{\pi \times H}{T}\quad\frac{\cosh\,.\,\frac{2\,\pi\,(h\,-\,z)}{\lambda}}{\sinh\,.\,\frac{2\,\pi\,h}{\lambda}}$$

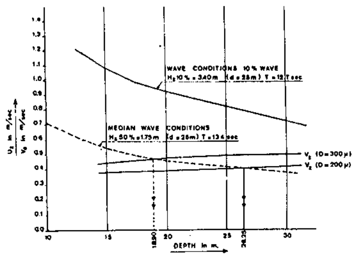
The threshold velocity at which movement of sand grains occurs, is computed as follows:

$$v_e = 4,5 \left[ \log \frac{12\ h}{4\ d_{50}} \right] \sqrt{d_{50}} \quad \text{while} \quad \frac{v_e}{v_z} = \frac{\log 32 - \log d_{50}}{\log 8\ (h-z) - \log d_{50}}$$

Since no sudden major sand transport takes place in the offshore region (no "storm" and "summer" profiles), it is assumed that the bottom configuration is determined by median wave conditions. Table nr. II gives the median wave conditions for the different water depths, as well as the $H_S$ 5 % wave conditions.

| Table nr. II | $H_S$ 50 % T, = 13,4 sec. | | $H_S$ 5 %, T = 15,7 sec. | |
|---|---|---|---|---|
| h in m | $H_S$ in m | λ in m | $H_S$ in m | λ in m |
| deep | 1,84 | 280 | 3,47 | 384,5 |
| 30 | 1,73 | 202 | 3,35 | 246 |
| 25 | 1,75 | 188 | 3,40 | 236 |
| 20 | 1,79 | 171 | 3,56 | 207,5 |
| 17,5 | 1,82 | 160 | 3,64 | 192,3 |
| 15,0 | 1,87 | 148 | 3,73 | 180,7 |
| 12,5 | 1,93 | 140 | 3,89 | 165,3 |
| 10 | 2,01 | 126 | 4,08 | 146,3 |

It is assumed that transport of bottom material takes place above the "apparent" bottom in a band of approximately 0,75 x the height of the ripples. The ripple height is approximately 0,10 x the water depth. The graph of illustration 5 shows that for median wave conditions 200 μ sand starts to move at a depth of 26,25 m and 300 μ sand at a depth of 18,90 m.




ill. 5. Critical velocity for D = 200 μ and D = 300 μ

The equilibrium slope developing in the offshore region can be estimated using the following equation:

$$\sin \alpha = \frac{k}{J} \cdot f_1 \left(\frac{h}{\lambda_0}\right), \text{ wherein } \frac{k}{J} = \frac{15,23\ H_0^2}{T.\ \lambda_0 \cdot d_{50}^2} \cdot \left(\frac{d^2_{50}}{T}\right) \cdot 0,57 \text{ and}$$

$$f_1 \left(\frac{h}{\lambda_0}\right) = \frac{\operatorname{cotgh}^2 \left(\frac{2\ \pi\ h}{\lambda}\right)}{\sinh^2 \frac{2\ \pi\ h}{\lambda} + \frac{2\ \pi\ h}{\lambda_0}}$$

The results of the calculation are given in table III.

| Table nr. III<br>waterdepth | $H_s$ . 50 %<br>$d_{50}$ = 300 μ | $H_s$ . 50 %<br>$d_{50}$ = 200 μ | $H_s$ . 5 %<br>$d_{50}$ = 300 μ |
|---|---|---|---|
| 30 m | | | 1 : 28 |
| 25 m | 1 : 76 | 1 : 54 | 1 : 17 |
| 20 m | 1 : 49 | 1 : 34 | 1 : 12 |
| 15 m | 1 : 29 | 1 : 20 | 1 : 7 |
| 12,5 m | 1 : 18,5 | 1 : 13 | 1 : 4 |

### 4.3.3. The breaking depth.

To estimate the depth at breaking, it is recommended by the Shore Protection Manual to use the empirical relationship between $d_b/H_b$ and $H_b/gT^2$ as derived from various beach slopes.

Station 2:

$\bar{H}_s > H_s$ 1 % = 5,15 m
$h$ = 25 m
$T_s$ = 16,6 sec.

$$\left.\begin{array}{l}\bar{H}_s > H_s \ 1\,\% = 5{,}15 \text{ m} \\ h = 25 \text{ m}\end{array}\right\} \ \lambda_0 = 430 \text{ m}$$

$$\left.\begin{array}{l} h = 25 \text{ m} \\ T_s = 16{,}6 \text{ sec.}\end{array}\right\} \ \frac{h}{\lambda_0} = \frac{25}{430} = 0{,}0581 \rightarrow \frac{H}{H_0^1} = 1{,}0 \rightarrow H_0^1 = 5{,}15 \text{ m}$$

$$\frac{H_0^1}{\lambda_0} = \frac{5{,}15}{430} = 0{,}012$$

| | | |
|---|---|---|
| slope 1 : 50 | $H_b$ = 7,11 m | <u>$d_b$ = 7,96 m</u> |
| slope 1 : 30 | $H_b$ = 7,62 m | $d_b$ = 8,08 m |
| slope 1 : 20 | $H_b$ = 8,14 m | $d_b$ = 7,81 m |

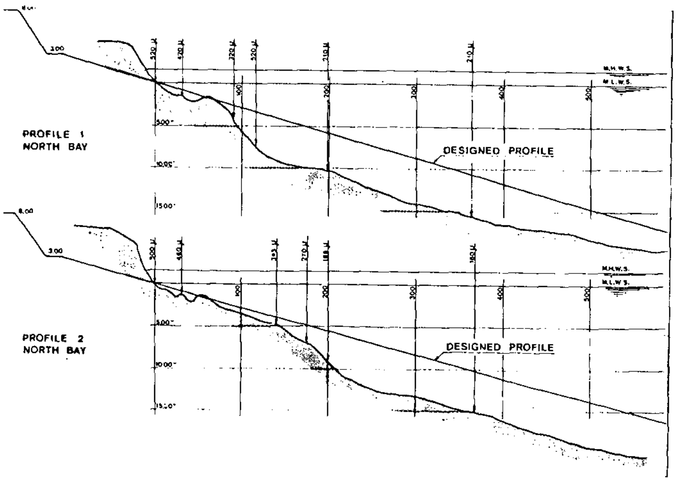
For $\bar{H}_s > H_s$ 50 % slope 1 : 50 <u>$d_b$ = 3,30 m</u>

### 4.3.4. The inshore region.

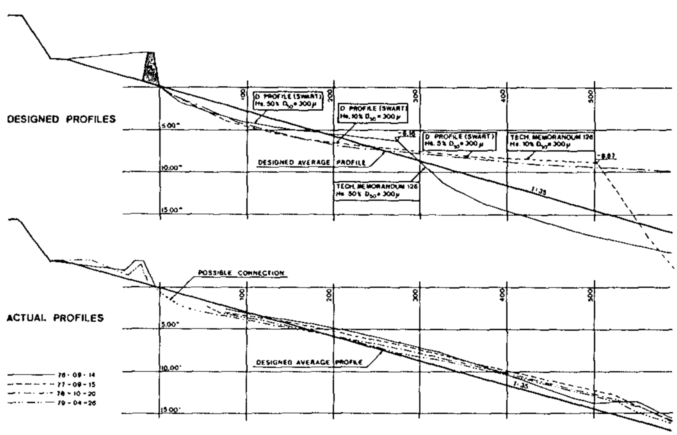
At the time of the initial design, "inshore" eguilibrium profiles could not be calculated theoretically. A great number of underwater profiles of existing beaches in the Saldanha Bay area and at other locations were compared with regard to wave attack and median sand grain diameter. This comparison indicated that an inshore profile slope of 1 : 35 with a sand of $d_{50}$ = 300 μ should be a safe estimate. In illustration 6 the designed profile is compared with the actual profiles of North Bay beach. The so called "sorting effect" of the sand grains along the profile should result in steeper slopes and thus increased safety. In 1974 mr. D.H. Swart published a method of calculating the eguilibrium profile of the so called D-profile (developping profile). In illustration 7 the theoretically calculated D-profiles (part below still water level) are given for $H_s$ 50 %, $H_s$ 10 % and $H_s$ 5 % wave conditions.

### 4.3.5. The foreshore (beach face).

Numerous field studies showed that there is a good correlation between median sand diameters and the slope of the foreshore. The foreshore (beach) slopes of the existing beaches around Saldanha Bay are corresponding closely with the data of these field studies. For an exposed beach and a medium grain size diameter of the sand of 300 μ, the slope of the foreshore shows <u>1 : 35</u>.




ill. 6. Comparison of designed profile with the actual profiles of North Bay beach.




ill. 7. Theoretically calculated D-profiles, spending beach design and actual profiles.

### 4.3.6. The "set-up".

"Set-up" due to wave action is that phenomenon whereby the average water level along the beach rises above that encountered under so called still water conditions. According to Bowen the set-up "z" depends on the breaker height $H_b$ and amounts to a maximum of $z = 0{,}25\ H_b$.

Assuming again the $\bar{H}_s > H_s$ . 1 % wave conditions:
$\bar{H}_s > H_s$ 1 % : 5,15 m (at station 2), $T_s = 16{,}6$ sec, $\lambda_0 = 430$ m.
$H_b = 7{,}62$ m.

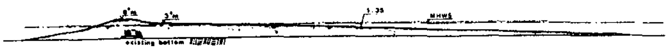
$$Z = 0{,}19\ [\ 1 - 2{,}82 \sqrt{\left(\frac{H_B}{gT^2}\right)}\ ]\ .\ H_B = 1{,}23 \text{ m}$$

Since a "storm" profile could develop in a very short space of time the breaker depth $d_b$ must be measured below C.D. ≘ M.L.W.S. The maximum height of the set-up "z" must be added to M.H.W.S. The maximum height of the wave run-up thus becomes 1,50 m (tide) + 1,23 m = 2,73 $m^+$ C.D. The design height of the berm crest was established at 3,00 $m^+$ C.D.

### 4.3.7. The back-shore.

Inland of the upper limit of wave set-up is called back-shore. The inland crest or berm crest could be overtopped by extremely high waves in which case the overtopping water flows down the so called deflation plane. Initially this plane was given a width of 20,00 m in order to take care of the changes in horizontal alignment. The final safety barrier (dike) was given a height of 8,00 $m^+$ C.D. (see ill. 8 ). As a result of the applied construction method, a stone wall was dumped at the proposed low water line up to a height of 4,00 $m^+$ C.D., the width at the crest being approximately 6,00 m. This relatively small stonewall consisted of quarry run 0-4000 kg. The stone crest was incorporated in the construction by filling the now 120 m wide plain between this stone crest and the root of the safety barrier up to a level of 3,10 $m^+$ C.D.

It shall be clear that this small stone wall prevented the foreshore (beach slope) to develop initially. It was assumed that the stone wall would finally disperse, due to wave action, below a level of M.H.W.S. with a seaward slope of approximately 1:4 - 1:6, thus offering an additional safety. The presence of this stone wall should also fix the horizontal alignment. Only the sand profile seaward of this wall could adapt itself to changes of incoming waves, resulting in varying depths just seaward of the wall. The final design profile is given on illustration 8.




ill. 8. Designed cross-section of spending beach breakwater.

## 5. CONSTRUCTION.

### 5.1. General.

For the construction of the spending beach breakwater about 20 million $m^3$ of sandfill was required.

The main volume of sand was obtained from the winning area due east of Salamander Bay (see illustration 3) while an additional amount of approximately 7 million $m^3$ became available during the dredging of the navigation channel.
The initial programme showed the construction time of the breakwater to be one year. However, during the first months of construction it appeared that the estimated production quantities in the winning areas were far too optimistic. This was mainly due to the fact that the sand grains were more cemented than anticipated which lowered the concentration of the sand-water mixture flow towards the suction pipes of the trailhoppers. As a consequence additional dredgers had to be mobilized in order to finalize the construction in time.

The breakwater was built in two stages. The first building stage comprized the dumping of sand in the profile by means of hopper dredgers up to a level of 6,6 m⁻ M.L.W.S. for which approximately 15 million $m^3$ of sand was required. The second building stage, being the closure of the upper part, was realized by the joint-effort of shallow draft hoppers and two big cutter-suction dredgers.

The closing started from Hoedjies Point in the lee of which the two cutter-suction dredgers operated initially. After the above water part protruded some 600 to 800 m from Hoedjies Point, one of the cutter-suction dredgers was moved in the lee of this above water part to recover sand from the underwater stockpile as well as the sand that was forced out of the profile due to wave action. It only took three months to close the "superstructure" of the spending beach breakwater, sometimes loosing some meters in the battle against a roaring Atlantic swell but always recovering quickly when the swell subsided.

## 5.2. The first building stage (see bottom up to 6,60 m⁻ M.L.W.S.)

The underwater mound was built of dredged material dumped into profile by hopper suction dredgers, the big hopper dredgers filling in the deeper parts and the smaller hoppers the shallower areas, always taking into account loaded draft and wave conditions. The local seabottom consisted of calcareous material interchanged with cemented sand layers and some loose sand. The original water depth varied from 16,00 m to 20,00 m below chart datum with steep granite slopes towards Marcus Island and Hoedjies Point. The dumping of sand by means of the different types of trailhoppers was carefully programmed and adjusted weekly in order to obtain a seaward under water profile closely approximating the designed profile.
On the 30th October 1975 the underwater mound was considered to be completed, having at that time an average height of 6,60 m⁻ M.L.W.S.
The quantities dumped into the underwater mound together with the spending beach stockpile, measured by shiploads, showed the following:

| | |
|---|---|
| . "Humber River" (big trailhopper) | 10,200,000 $m^3$ |
| . "Willemstad" (med. trailhopper) | 3,900,000 $m^3$ |
| . "Volvox Zelandia" (transmundum type) | 3,300,000 $m^3$ |
| . "Geopotes 12" (transmundum type) | 1,100,000 $m^3$ |
| . "Queen of Holland" (cutter-suction) *) | 1,900,000 $m^3$ |
| Total | 20,400,000 $m^3$ |

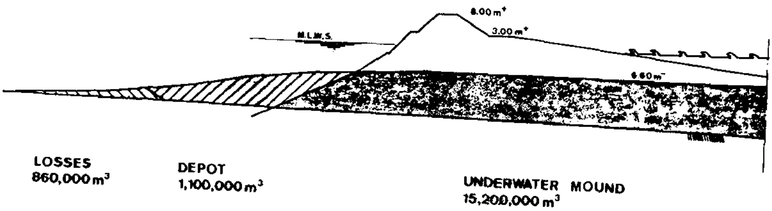
*) During the first vain attempt to close the spending beach, the cutter-suction dredger "Queen of Holland" pumped 1,900,000 $m^3$ into the spending beach, which quantity settled mainly below a level of 6,60 m minum M.L.W.S. and as such had to be added to the underwater mound.

The total quantity of 20,400,000 $m^3$ could be divided into 19,100,000 $m^3$ for the underwater mound and 1,300,000 $m^3$ for the spending beach underwater stockpile, situated at the lee side of the underwater mound along a distance of 800 m from Hoedjies Point.

The volumes of sand, including sandlosses, measured in situ as the difference of the initial soundings and the soundings at the day of completion of the underwater mound (30th October 1975) were as follows:

| | |
|---|---|
| . underwater mound | 15,200,000 $m^3$ |
| . stockpile spending beach | 1,100,000 $m^3$ |
| . sandlosses (measured outside profile) | 900,000 $m^3$ |
| Total | 17,200,000 $m^3$ |

In the cross section of illustration 9 the underwater mound, stockpile and sandlosses are indicated. The locations of the stockpile areas which were prepared during the first building stage and used for the closure of the upper part during.




ill. 9. Cross section of spending beach after completion of stage I, including sand stockpile and sandlosses.

The difference between the so called shipload quantities and the actual in situ measured quantities, totalling up to 3,200,000 $m^3$, can be attributed to the following:

a. Settlement of the subsoil as a result of the sandweight. Taking into account the results of the drilling in this area an average settlement of 0,15 m over an area of 1,800 x 400 m was considered as reasonable: 1,800 x 400 x 0,15 ≈ 100,000 $m^3$.
b. Too high estimates of shipload quantities as a result of:
   1) sand remaining in the ship holds during discharging (dumping).
   2) ratio between solid and loosely packed sand.
   3) loss of very fine material during the dumping process. This very fine material settled in a large area outside the beach profile in a thin layer on top of the existing seafloor not noticeable by echosounding.

The experience gathered at Saldanha Bay justified a reduction of 18 % for the bigger trailhopper dredgers and 12 % for the "transmundum" type of trailhopper dredgers. These reduction applied to the given quantities, give:
0,18 (10,200,000 + 3,900,000) + 0,12 (3,300,000 + 1,100,000) = 3,100,000 $m^3$.
Total: (100,000 + 3,100,000) = 3,200,000 $m^3$.

Sandlosses underwater mound.

According to the in- and out surveys of the construction area the total volume of sand which eventually arrived outside the construction limits was 900,000 $m^3$ which is 5 % of the total measured quantity. A more complete insight in the distribution of the volume of sandlosses as found after stage I of the spending beach was completed, can be obtained from illustration 10. This illustration shows the quantities of sand leeward of the underwater mound including losses and stockpiled sand per section of 100 meter across the length of the breakwater. The outer boundary of the transported sediment is also indicated; as an average the sand was transported upto ca. 500 m behind the stockpile area.
The biggest quantities of sandlosses were found near Hoedjies Point. This is explained by the fact that the construction of the underwater mound started from Hoedjies Point and from there was gradually extended towards Marcus Island.
It is interesting to note that these sandlosses did not contribute to the volume of sand in the stockpile area. The total quantities dumped in this area as measured from the shiploads and reduced with the above mentioned reduction factors were as follows:

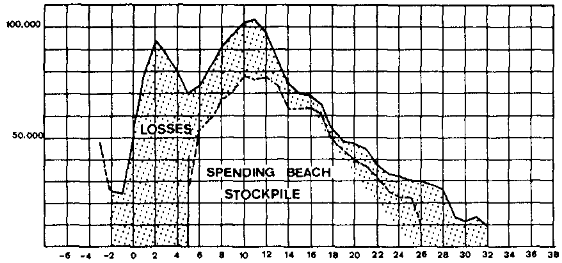
| | | |
|---|---|---|
| . "Humber River" | 1,200,000 x 0,82 = | 980,000 $m^3$ |
| . "Willemstad" | 100,000 x 0,82 = | 80,000 $m^3$ |
| . "Beachway" | 35,000 x 0,88 = | 30,000 $m^3$ |
| . "Geopotes 12" | 50,000 x 0,88 = | 40,000 $m^3$ |
| | | 1,000,000 $m^3$ |

which quantity approximately equals the measured quantity of 1,100,000 $m^3$ in the stockpile area.

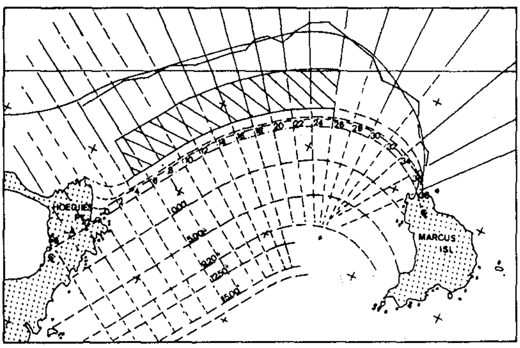
## 5.3. The second building area stage (6,60 $m^-$ C.D. up to 3,50 $m^+$ C.D.).

### 5.3.1. The temporary breakwater.

The first vain attempt to close the gap between Hoedjies Point and Marcus Island, resulted into a small beach around Hoedjies Point, shading off into part of the underwater mound.
This beach in conjunction with the depth contours of the underwater mound, existing during June-September 1974, caused a most unfavourable refraction-diffraction pattern, focusing the waves at the area of Smitwinkel Bay, in which the village of Saldanha and her different fish-factories are located. During the winter-period of 1974, storms caused therefore considerable damage to moored ships, quaywalls, small jetties etc. As the second building stage was scheduled for the period November 1974 - April 1976, a second winter-period had to be confronted. To secure the shelter of the existing fishing harbour area, it was therefore decided to build a so-called temporary breakwater. This breakwater, protruding from the lee side of Hoedjies Point, served at the same time as the outer limit of the sand stockpile nr. I behind Hoedjies Point (see illustrations 11 and 12).



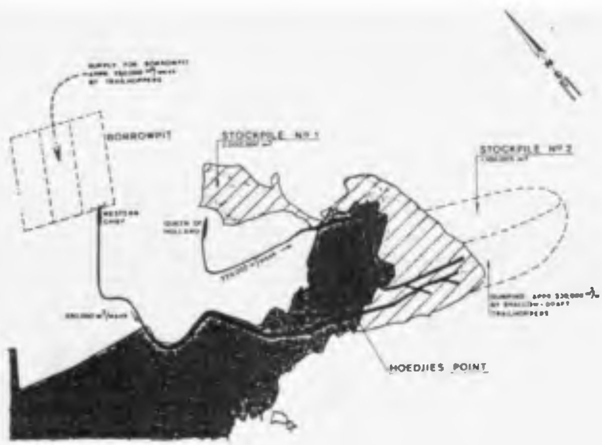

DISTRIBUTION OF SAND VOLUME LEEWARD OF UNDERWATERMOUND
BEFORE START OF CONSTRUCTION STAGE II




OUTERBOUNDARY OF TRANSPORTED SEDIMENT

ill. 10. Distribution of transported sediment.

ill. 11. Aerial photograph of Hoedjies Point at the beginning of the second building stage.




ill. 12. Operation of cutter suction dredgers at the beginning of the second building stage.

### 5.3.2. Working scheme and sand production.

The required amount of material for this second building stage comprized approximately 5,000,000 $m^3$.
To minimize the sandlosses a continuous high production was essential. For the "final attack", which started from Hoedjies Point two cutter-suction dredgers were used, having a joint production of approximately 500,000 $m^3$ per week, assisted by three transmundum type, shallow draft hoppers with a total weekly production of about 250,000 $m^3$.
Both cutter-suction dredgers operated initially in the lee of Hoedjies Point. One of these dredgers was operating in an earlier prepared stockpile (2 million $m^3$) while the other obtained his material from a borrowpit, continuously refilled by the bigger trailhopper dredgers. (See illustrations 11 and 12.) While the "above-water" part of the breakwater was striding along towards Marcus Island, the three transmundum type hoppers were used to dump immediately in front of the above-water part, taking full advantage of their horizontal sliding bottom doors.
A second underwater stockpile has been prepared behind the first 800 m to 900 m of the future breakwater, which stockpile was increased by the losses during the construction of the adjacent superstructure. As the construction of the superstructure progressed one of the cutter-suction dredgers, advanced from the stockpile no. 1 into the created shelter of the partly finalized breakwater. The complete sand transport scheme is given on illustration 12. The working scheme was adapted weekly in order to optimize the output of this expensive dredging fleet.

### 5.3.3. The superstructure.

The construction of the temporary breakwater in the lee of Hoedjies Point has learned that a stone wall located at the proposed low water line, with its crest at approximately 4,00 $m^+$, protruding seawards like a knife-edge, considerably restricted the amount of sand washed away from the head.
It proved that a relatively small stone wall (average 120 ton/$m^1$), combined with the pumping of sand at the seaside some 150 m distance behind this head, offered the best progress. The wave forces transported the sand along the seaward profile as well as longitudinally towards the "knife-edge" of the stone head.
Directly in the lee of the protruding stone-wall the sand settled and here bulldozers were used to push up the sand towards the backside of the wall. Sand accretion also took place in the deeper areas just in front of the head. As soon as the level here reached 0,00 m, stone was backtipped, shifting the spear head (knife-edge) forward again, as can be seen on illustration 13. This illustration also gives an impression of the enormous size of the breaking waves as compared to the size of the truck. Every 50 m a pipe branched off running through the small stone wall towards the seaside. It very quickly became clear that for the majority of the time, the seaward outlets had to be operated, making use of the waves to carry out the further transport of the spoil. Hydraulic fill directly in the centre part of the spending beach was restricted to a distance of 150 m, backwards from the head in order to minimize sandlosses. The cutter production system was gradually improved during the course of the construction i.e. the lay out of the discharge pipelines and rubble mound dam and also the progress of the rubble mound.

The "safety barrier" with its crest up to 8,00 $m^+$ C.D. was made in the orthodox way by hydraulic fill in between aand walls.

ill. 13. Protruding stone wall shifted forward and acting as knife-edge. Note the size of the breaking waves.

The above described construction method made it possible to realize the closure in 14 weeks with a minimum loss of sand and two months ahead of schedule. The progress is made visible in illustration 14.

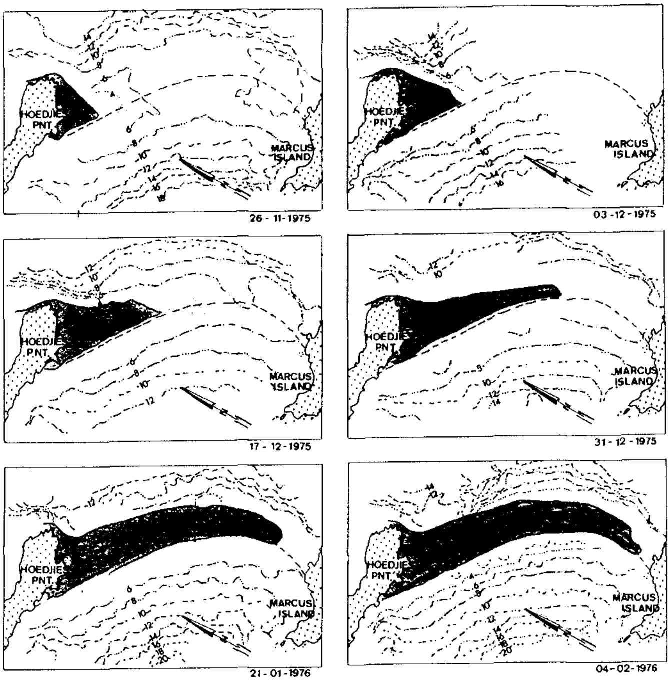
The maximum significant wave height encountered during construction was $H_s$ = 3,30 m (7-12-1975). At station no. 2, some 1,500 m seaward of the crest line of the spending beach, the Datawell wave rider buoy measured a maximum wave of $H_m$ = 5,05 m ($T_s$ = 14,5 sec) at that day.

The amount of stone per $m^1$ increased during periods of heavy wave attack. It only happened once that an earlier reclaimed area had to be returned to the sea.

On December 17th 1975, while about 800 m of the 1,900 m gap between Hoedjies Point and Marcus Island was brought above water, current measurements were executed over the remaining part of the underwater mound (average depth 6,60 $m^-$). Wave conditions were moderate $H_s$ = 1,75 m. Tide difference 1,30 m. Maximum current measured over a full tidal cyclus 0,30 m/sec.

Grain-size diameter.

During construction of the underwater mound a great number of bottom (grab) samples was taken mainly at depths between 7,00 $m^-$ and 8,00 $m^-$ C.D. The median grain-size diameter varied between 110 µ and 220 µ. The material of the superstructure however, proved to be much coarser. The average of six drillings made in the sand stockpile nr. 1 showed 53 % above 210 µ and 35 % above 420 µ. Samples taken from the cutter-suction dredger "Western Chief" gave an average of 370 µ for the median grain-size diameter. Unfortunately no bottom aamples on the seaward slope were taken after completion.




ill. 14. Progress of superstructure of spending beach breakwater.

As a result of the way of execution (pipe outlet on the seaside), the so called sorting effect started already during execution: the finer sand particles being washed away more easily by the waves and the bigger sand particles settling earlier. It can therefore be concluded that the median grain-size diameter of the seaward slope on average is in excess of 300 μ.

5.3.4. Sandlosses second building stage.

Sand losses of the superstructure were calculated up to 4-2-1976, being the data the connection with Marcus Island was realized. The quantities brought into the superstructure during the period from 30-10-1975 up to 4-2-1976 measured as follows:

| | |
|---|---|
| . Transmundum type of trailhoppers 2,220,000 $m^3$ | |
| Reduction factor to obtain solid $m^3$ x 0,88 | ~ 2,000,000 $m^3$ |
| . Cutter-suction dredgers | ~ 5,100,000 $m^3$ |
| Calculated solid $m^3$ | ~ 7,100,000 $m^3$ |

The volumes of sand measured in situ as the difference between the depth soundings of 30-10-1975 and 4-2-1976 ~ 5.600.000 $m^3$

The total losses comprise 21,1 % of the total sand production during the second building stage.

The losses were defined as quantities shifted outside the required profile by wave action, currents etc. The majority of the losses of the first 900 meters of spending beach accumulated in the stockpile area number II and were recovered by the cutter suction dredger "Queen of Holland" in building the second half of the superstructure.
As the construction of the superstructure proceeded the dredging process improved. This can be illustrated by comparing the sandlosses which occurred during the construction of the first 900 meters with the sandlosses which occurred during the construction of the remaining second half of the superstructure. They were respectively 29,7 % and 11,8 % of the corresponding sand productions.
This reduction in losses can be ascribed to
- less wave action during the construction of the second half of the superstructure
- improvement of the fill distribution in the reclaim area.

## 6. CONCLUSION.

After the construction was finished, echo soundings were performed at regular intervals during the period 1976-1979 in order to establish the seaside slope of the spending beach breakwater. An example of these actual measured profiles is given in illustration 7.

1) Comparison between the designed profile and the actual measured profiles show a good correlation.

2) This seaward profile up to a depth of approximately 6.00 m⁻ is very simular to Swarts D-profiles. Correlation with Technical Memorandum 126 is poor, specifically in the shallower areas.

3) None of the numerous echo soundings show an underwater bar which could be explained by the fact that the wave seasons have only minor differences.

4) The relatively small stonewall - used during construction to minimize sandlosses - has shaped into a protective layer around the area of low water and as such fixes the low water line of the seaward profile.

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