

## CHAPTER 193

### BEACH IMPROVEMENT SCHEMES IN FALSE BAY

by

D H Swart\* and J S Schoonees\*\*

#### ABSTRACT

A physical sediment model was used to design two beach improvement schemes in False Bay near Cape Town. Different layouts of T, L and detached breakwaters were tested to determine their effect on the beach and how they improved bathing conditions. Although caution was exercised to eliminate scale effects, it was found to be necessary to minimize their influences. A method was devised to do so which proved successful. It was found that equilibrium shoreline positions were reached after 4 years (prototype time). Sand feeding after construction will be necessary to minimize the deleterious effects of the breakwaters on the adjacent beaches.

#### INTRODUCTION

False Bay which is a large, partly protected bay (ca 35 km x 35 km) is situated near Cape Town, South Africa (Figure 1). The study site covered about 8 km of coastline from Strandfontein to Kapteinsklip alongshore (Figure 1) and up to the -20 m to mean sea level (MSL) contour. False Bay provides beach recreational facilities for the nearby metropolitan area of Cape Town.

The CSIR was commissioned to undertake studies on recreational schemes proposed for the north shore of False Bay. A tidal pool was built at Strandfontein as Phase 1 of this project. Phase 2 consists of beach improvement schemes at Kapteinsklip and Middelbank. The aim of this latter phase is to provide safe bathing beaches using groynes. At present the major part of the coast is rather unsafe for bathing mainly because of steep

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\* Division director and \*\* Research engineer, CSIR, P O Box 320, 7599 Stellenbosch, South Africa.

beach profiles. The client laid down specific criteria to ensure safe bathing, e.g. a uniform flat beach slope and the absence of rock and strong rip currents. Further requirements were that a safe bathing beach forms soon after completion of the structures and that the adjacent coastline must not be adversely affected in the long term. This paper deals with the design of these beach improvements schemes using a physical sediment model.

Based on the expected number of visitors and bathers and assuming that 10 bathers per metre distance alongshore can be accommodated, it was stipulated that 1 200 m and 2 000 m of safe bathing beach must be provided at Kapteinsklip and Middelbank respectively.

Waves were recorded in about 20 m of water using two Waverider buoys. During the one year recording period, the mean significant wave height at the Waveriders was 1,2 m with a mean peak wave period of 11,1 s. The tides in the False Bay area are semi-diurnal with a mean spring tidal range of 1,6 m. From quarterly surveys it was found that the average median sand grain size on the beach is 0,45 mm. A best estimate of the gross longshore transport rate is 387 000 m<sup>3</sup>/year while the net longshore transport is eastbound and about 145 000 m<sup>3</sup>/year (CSIR, 1983). More details about the characteristics of the site can be found in Schoonees and Möller (1982) and CSIR (1983, 1988).

The mobile bed model which was about 100 m by 35 m, had a horizontal length scale of 1/20 and a vertical scale of 1/50 (giving a distortion of 2,4). The model sediment was sand with a median grain size ( $D_{50}$ ) of 0,315 mm. Calibration of shoreline evolution was carried out using results from aerial photography. This yielded a sedimentological time scale of 24 h of waves in the model being equivalent to 1 year in prototype. The wave climate was schematized into five cycles comprising quasi-irregular waves from both the westerly and easterly sectors. Schoonees and Möller (1982) and CSIR (1988) discusses the design and calibration of this model in detail.

The values of parameters given below are prototype values unless specified otherwise.

## TEST SERIES

### General

The authorities plan to implement the Kapteinsklip beach improvement scheme first followed by Middelbank some four years later. Because of the

time and cost involved, it was decided to run most of the tests for four years. This was found to be sufficient to reach an equilibrium coastline at the groynes as will be discussed later.

### **Programme**

The groyne layouts for the Kapteinsklip scheme, the A series, are shown in Figure 2a. Figure 2b illustrates the groynes tested in the Middelbank area. T, L and detached breakwaters were modelled. In addition, a zero test (Test A5) was conducted to determine the behaviour of the model without any structures. Having built in the 4 year shoreline condition (of an L groyne) at Kapteinsklip, another zero test (Test B3) was run; in this case for the Middelbank scheme. This test also served the purpose of showing shoreline response over 8 years at Kapteinsklip because the initial shoreline was the 4 year condition.

### **Scale Effects**

The effect of varying the number of cycles of westerly and easterly waves to make up one representative year was investigated. Both 5 and 10 cycles were tested. It was found that there was very little difference in coastline evolution between using 5 or 10 cycles. The easier and cheaper option of 5 cycles was therefore chosen.

Sediment samples were taken during Test B3 at different cross-shore and longshore locations to investigate the variation in  $D_{50}$  over time. No consistent tendencies and therefore scale effects due to sorting were found.

Rocky areas must be accurately represented in the model. Because they limit the availability of sand, rocky areas have a profound effect on coastline evolution if they are present. Care, therefore, should be exercised to survey them in detail so that they can be modelled correctly.

Three different scale effects were present in the results. These are diffraction effects causing the longshore transport to be incorrectly simulated, onshore sediment transport and the formation of cusps. For each of these, a factor was determined such that the shoreline response could be modified to account for these scale effects.

Because it was a distorted model, only refraction or diffraction could be reproduced correctly. Refraction was chosen to be of more importance and thus diffraction had to be corrected for. This was done as follows:

- Diffraction coefficients were determined from diffraction diagrams (US Army, Corps of Engineers, 1977) along a line in the lee of the groynes and parallel to the shore. This was done for the diffraction in the model and for the case where diffraction would have been correctly

simulated. From these, the incident wave characteristics and by assuming Snell's law, the breaker conditions were calculated.

- The longshore transport rates were then computed at different positions along the shore for both diffraction cases. The formulae by Engelund, Hansen and Swart and the SPM adapted by Swart (Swart, 1976a and b) were used. The Schoonees (1986) method was used to account for a gradient in breaker height.
- Volume changes per time were derived from the difference in longshore transport rates. These yielded the diffraction factor,  $k_{dif}$ . Following this procedure for westerly and easterly waves, weighted mean diffraction factors were determined along the beach. These varied around 1,0 (from 0,5 to 2,0). A factor of 1,0 means that the rate of shoreline change was predicted correctly in the model. These factors were almost always positive which means that the model predicted accretion and erosion to occur in the correct locations.

To account for onshore sand transport, mean beach profiles found in the model during Test A5 (a zero test) were computed after 1, 2, 3 and 4 years. A cross-shore sediment transport model (Swart, 1974) was calibrated for both model and prototype beaches. Thereafter the model was used to predict the onshore transport correction factor ( $k_{dwa}$ ) for different wave heights.

$$k_{dwa} = \text{(predicted distance that the coastline changed under model conditions, scaled to prototype size) / (predicted distance that the coastline changed in prototype conditions)}$$

An analysis of the cusps found in the zero tests (Figure 3) yielded a mean cusp factor ( $k_{str}$ ) of 2,0. The ratio of the distance to the apex of the cusp to the distance to the new mean shoreline position without the cusp is  $k_{str}$ . This factor was taken to be 2,0 if a cusp was present or 1,0 if not. This could be done because adaptations of the coastline position were only done at or close to the apex of cusps if they were present.

Adaptation of the coastline changes to account for these three scale effects were done as follows:

$$\begin{aligned} V_m &= V_{dwa,m} + V_{dif,m} \\ \text{and } V_p &= V_{dwa,p} + V_{dif,p} \\ \text{where } V &= \text{volume accreted or eroded} \end{aligned}$$

and subscripts dwa = onshore transport

dif = diffraction  
 m = model  
 and p = prototype

Furthermore:  $k_{dif} = V_{dif,m} / V_{dif,p}$   
 $k_{dwa} = V_{dwa,m} / V_{dwa,p}$

Substituting these equations it is possible to obtain  $V_p$  as a function of  $V_{dwa,m}$ ,  $V_m$ ,  $k_{dif}$  and  $k_{dwa}$ . Assuming that the distance over which the shoreline changes horizontally is directly proportional to  $V$  (which model results confirmed) and incorporating  $k_{str}$ , the result is:

$$a_p = a_{dwa,m} \left( \frac{1}{k_{dwa} \cdot k_{str}} - \frac{1}{k_{dif}} \right) + a_m / k_{dif}$$

This relationship was used to modify the measured model responses. It should be noted that the model was designed using preliminary (visually estimated) wave data. While the model was being constructed, the wave recordings were carried out. If the measured wave characteristics are used to compute the vertical scale in the same way as explained in Schoonees and Möller (1982), the vertical scale would have been 1/75 (instead of 1/50) giving a distortion of 1.6. This would definitely affect the cross-shore transport and possibly also the formation of cusps, thus influencing (hopefully reducing) the scale effects in the model.

### Test Results and Discussion

Figures 4a and b show the original model results of Test A4 and the model response of this test corrected for the above-mentioned scale effects respectively. Note that the adaptation of the results were only done in the lee of the L groyne and not to the right of the groyne. The corrected shoreline response of Test B3 which started with the 4 year condition of Test A4, is illustrated in Figure 4c. Shoreline variations over a period of 8 years were thus modelled. From these figures it can be seen that most shoreline changes occurred within 2 or 3 years after construction of the groynes. Equilibrium is reached after 4 years. The same was found of the coastline evolution in the other tests. This is in accordance with what has been found in the field in Israel (Nir, 1982), Japan (Toyoshima, 1976) and Brittain (Barber and Davies, 1985). Thereafter the shoreline fluctuates slightly around the equilibrium position (Figure 4c).

Detailed wave and current measurements were done in order to quantify the improvement in bathing conditions. Figures 5a and b show wave height contours and current velocities for regular westerly waves at the end of Test A6. The comfort of beach users will be improved by the schemes

because the beaches will be wider and will vary less in width over time. Bathing safety will be substantially enhanced because of the reduction in wave action and the associated reduction in current velocities. However, a varying wave climate will be provided along the beach from calm areas to enough wave action for the more adventurous swimmer. In addition, the beach slopes will be flatter and most of the rock will be covered by sand. Rock pinnacles which will still be exposed will have to be removed. To limit the return flow (out of gaps between groynes) due to wave overtopping, it was recommended that the crest height of the groynes be +2 m to MSL. An analysis of prototype schemes around the world and the tidal ranges occurring at those sites, also indicated this crest height to be realistic.

To minimize the effect of the schemes on the adjacent coastline, it was recommended that sand feeding be carried out with compatible sand. This should be done once and up to the equilibrium shoreline position after the construction of the groynes is complete.

Kamphuis (1975) presents an equation to calculate the sedimentological time scale. In this formula the factor  $M$  should be 1 for perfect agreement. By analysing a number of studies he showed  $M$  to vary between 0,35 and 14,6. For this study  $M = 1,48$ , which is in good agreement with the theoretical and empirical values.

### **Proposed Schemes**

Figure 6a presents the proposed scheme at Kapteinsklip. Together with the predicted equilibrium coastline position, areas are shown where additional beaches can be created using sand nourishment. In Figures 6b and c two alternatives are given for the Middelbank scheme. For both the schemes at Kapteinsklip and Middelbank, the required length of safe bathing beach will be obtained. Sand feeding is strongly recommended to minimize the effect on the adjacent beaches. Building each of these schemes in phases in order to limit the initial expenditure, was also considered and found to be viable (CSIR, 1989).

### **CONCLUSIONS**

A physical sediment model is an effective tool to apply to investigations of coastline evolution following the construction of groynes. Clients and designers can see what happens during tests. However, these models are expensive to build and operate.

Scale effects are normally present in studies using mobile bed models. Caution should be exercised to minimise these. In this study the important scale effects were found to be the result of diffraction effects, onshore sand transport and cusps. A method was devised and applied successfully to minimize these scale effects. To achieve this, it is essential to conduct (a)

zero test(s) by which the model behaviour without structures can be quantified. It was also found that rocky areas should be built in accurately because they can have a profound effect on beach evolution.

In accordance with field observations worldwide it was found that most shoreline changes occurred within the first 2 to 3 years after groyne construction. Equilibrium shoreline positions are reached after about 4 years. An independent check on the sedimentological time scale confirmed its accuracy.

The T, L and detached breakwaters proposed for the beach improvement schemes (Figures 6a - c) will improve bathing conditions significantly. Sand feeding after construction is necessary to minimize the deleterious effects of the groynes on the adjacent beaches.

### **ACKNOWLEDGEMENT**

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### **REFERENCES**

Barber, P C and Davies, C D (1985). Offshore breakwaters - Leasowe Bay. Proceedings of the Institution of Civil Engineers, Part 1, Volume 77: 85-109.

CSIR (1983). False Bay: Field data report. CSIR Report C/SEA 8219/1,2 and 3, Stellenbosch. (In Afrikaans).

CSIR (1988). False Bay: The hydraulic design of beach improvement schemes with a physical sediment model. CSIR Report EMA-C 8886, Stellenbosch (in Afrikaans).

CSIR (1989). Improved beach amenities along the False Bay coast between Strandfontein and Mhandi. CSIR Report EMA-C 8940, Ematek, Stellenbosch.

Kamphuis, J W (1975). The coastal mobile bed model. C E Research Report No 75. Dept Civil Engineering, Queens University, Kingston, Ontario.

Nir, Y (1982). Offshore artificial structures and their influence on the Israel and Sinai Mediterranean beaches. 18 Intern. Conf. on Coastal Eng., ASCE, Volume 3: 1837-1856, Cape Town.

Schoonees, J S (1986). Breaker wave characteristics and longshore sediment transport along a bay. M.Eng. Thesis, University of Stellenbosch, Stellenbosch.

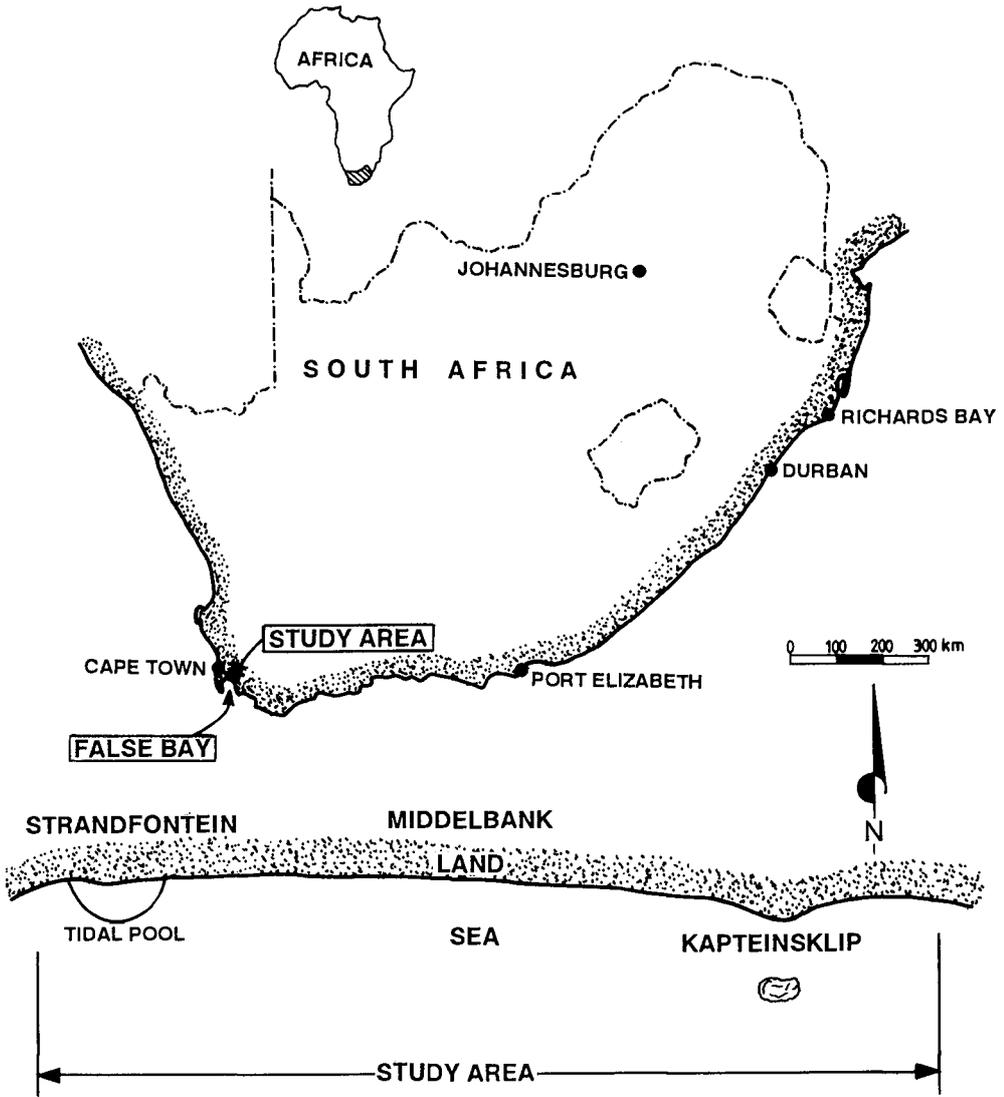


FIGURE 1: LOCATION MAP

Schoonees, J S and Möller, J P (1982). Design and calibration of False Bay sediment model. 18 Intern. Conf. on Coastal Eng., ASCE, Vol. 2: 1161-1180, Cape Town.

Swart, D H (1974). Offshore sediment transport and equilibrium beach profiles. Delft Hydraulics Laboratory Publication No 131, Delft.

Swart, D H (1976a). Coastal sediment transport. Computation of longshore transport. Report R968, Part 1, Delft Hydraulics Laboratory, Delft.

Swart D H (1976b). Predictive equations regarding coastal transports. 15 Intern. Conf. on Coastal Eng., ASCE, Vol. 2: 1113-1132, Honolulu, Hawaii.

Toyoshima, O (1976). Changes of sea bed due to detached breakwaters. 15 Intern. Conf. on Coastal Eng, ASCE, Volume 2: 1572-1589, Honolulu, Hawaii.

United States Army, Corps of Engineers (1977). Shore Protection Manual. Volumes I, II and III. Fort Belvoir, Virginia.

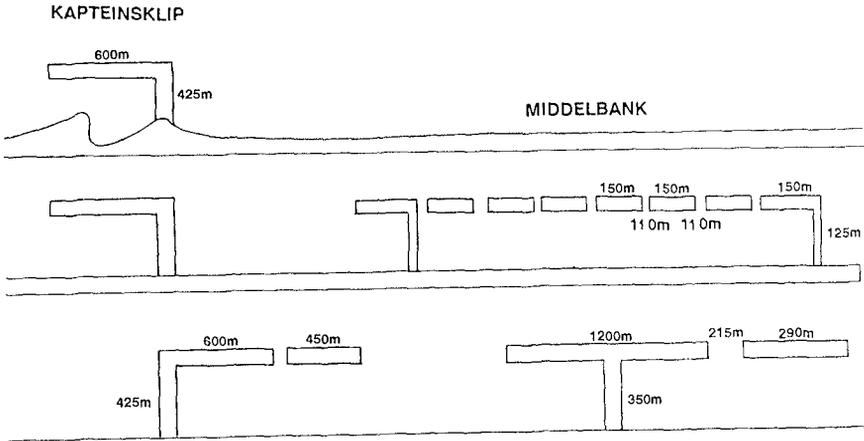


FIGURE 2b: GROUYE LAYOUTS AT MIDDELBANK

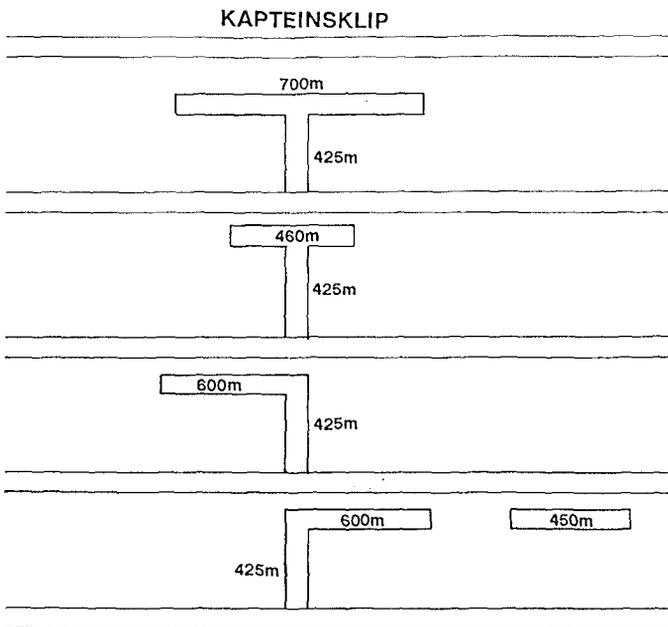


FIGURE 2a: GROUYE LAYOUTS AT KAPTEINSKLIP

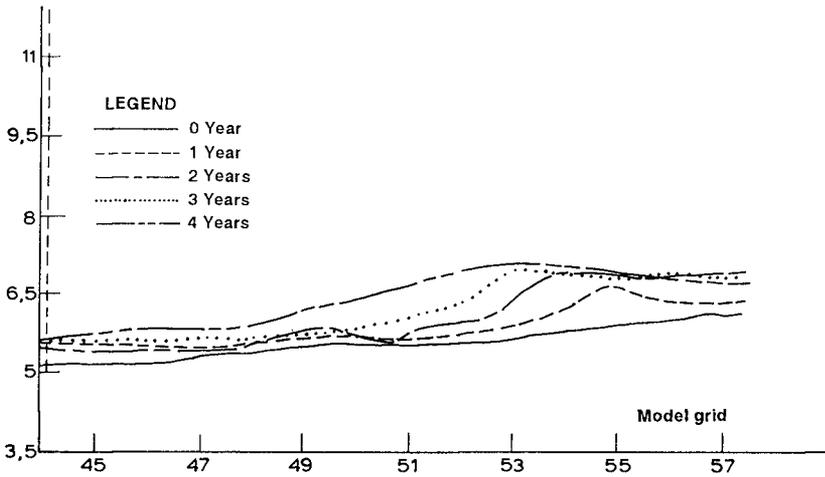


FIGURE 3: COASTLINE CHANGES DURING A ZERO TEST (A5)

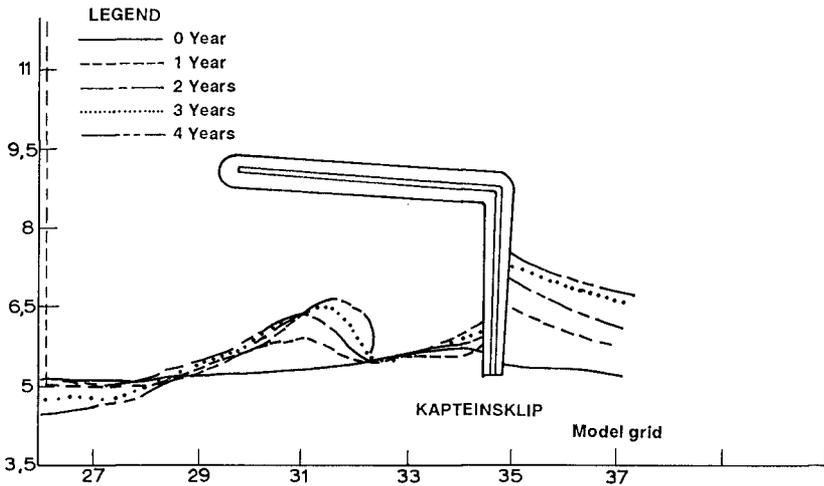


FIGURE 4a: COASTLINE CHANGES DURING TEST A4  
(NOT CORRECTED FOR SCALE EFFECTS)

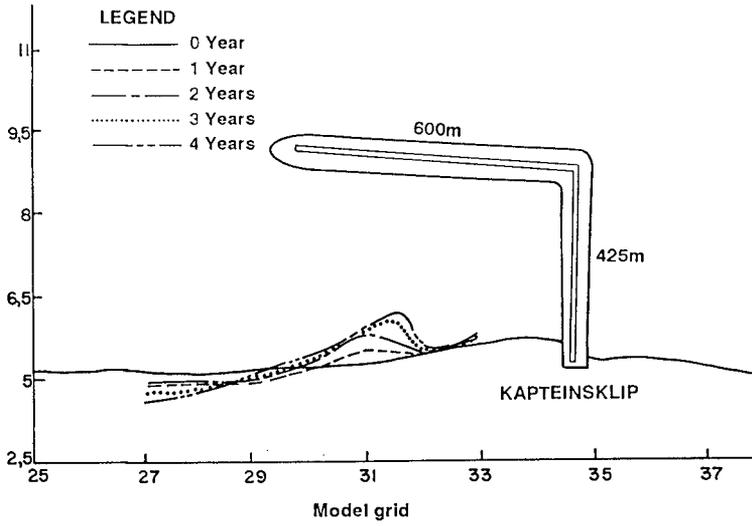


FIGURE 4b: COASTLINE CHANGES DURING TEST A4  
(CORRECTED FOR SCALE EFFECTS)

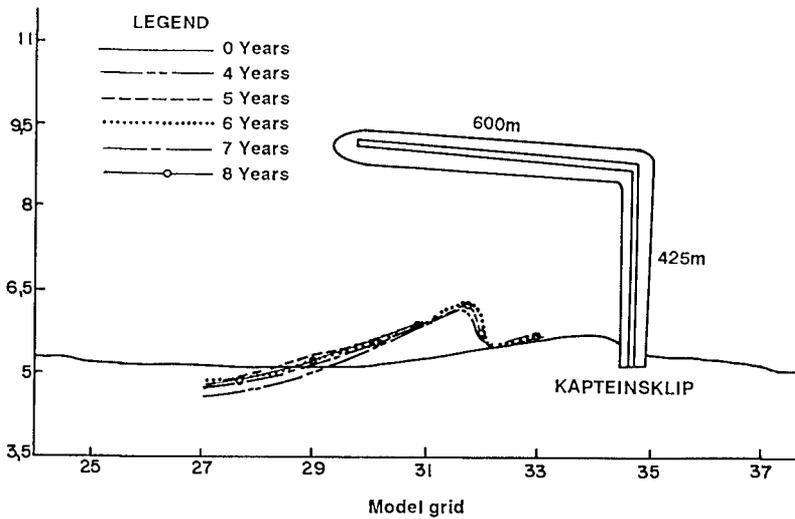


FIGURE 4c: COASTLINE CHANGES DURING TEST B3  
(CORRECTED FOR SCALE EFFECTS)

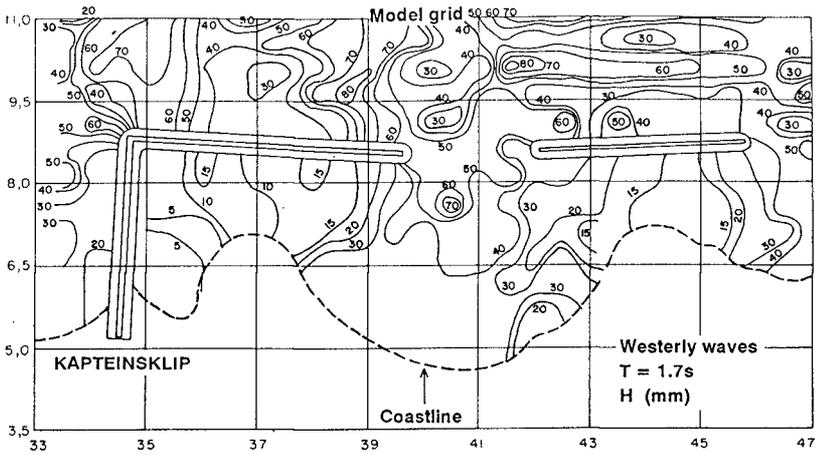


FIGURE 5a: WAVE HEIGHT CONTOURES

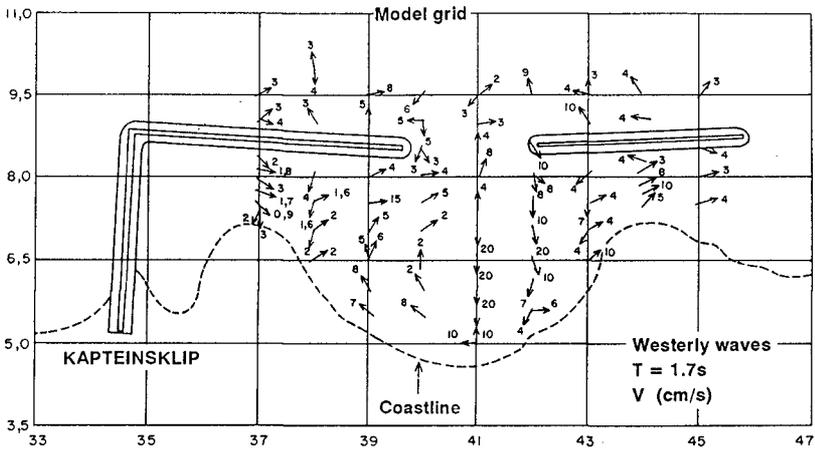


FIGURE 5b: CURRENT VELOCITIES

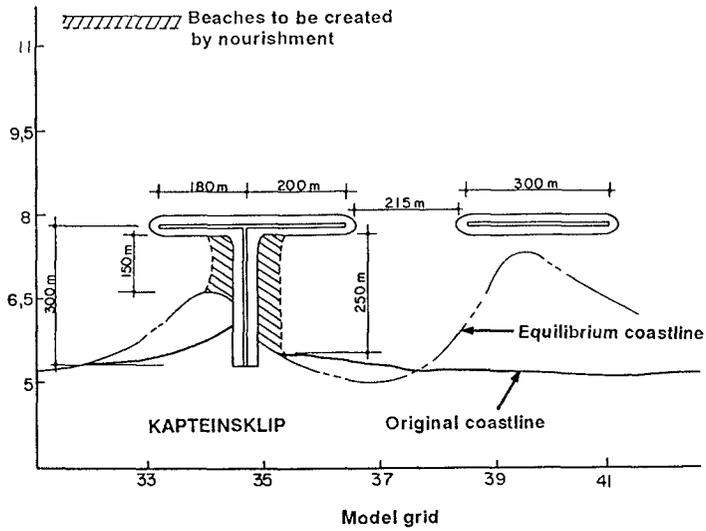


FIGURE 6a: PROPOSED LAYOUT AT KAPTEINSKLIP

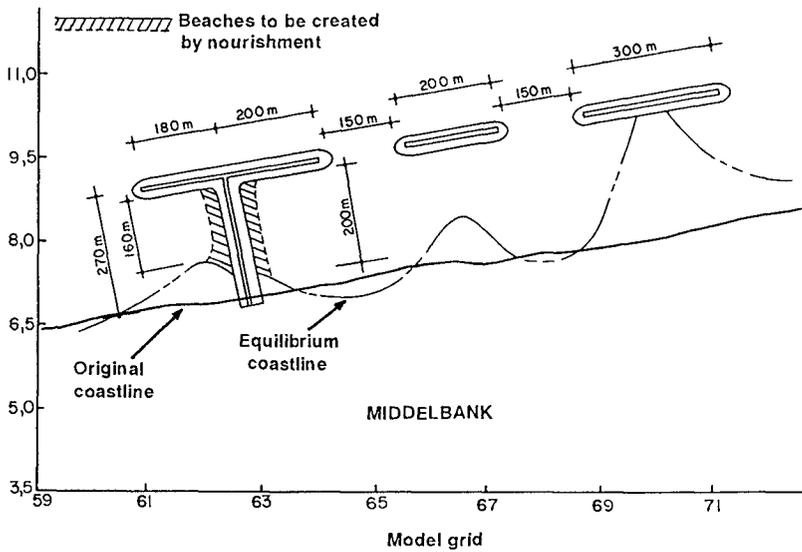


FIGURE 6c: A PROPOSED LAYOUT AT MIDDELBANK

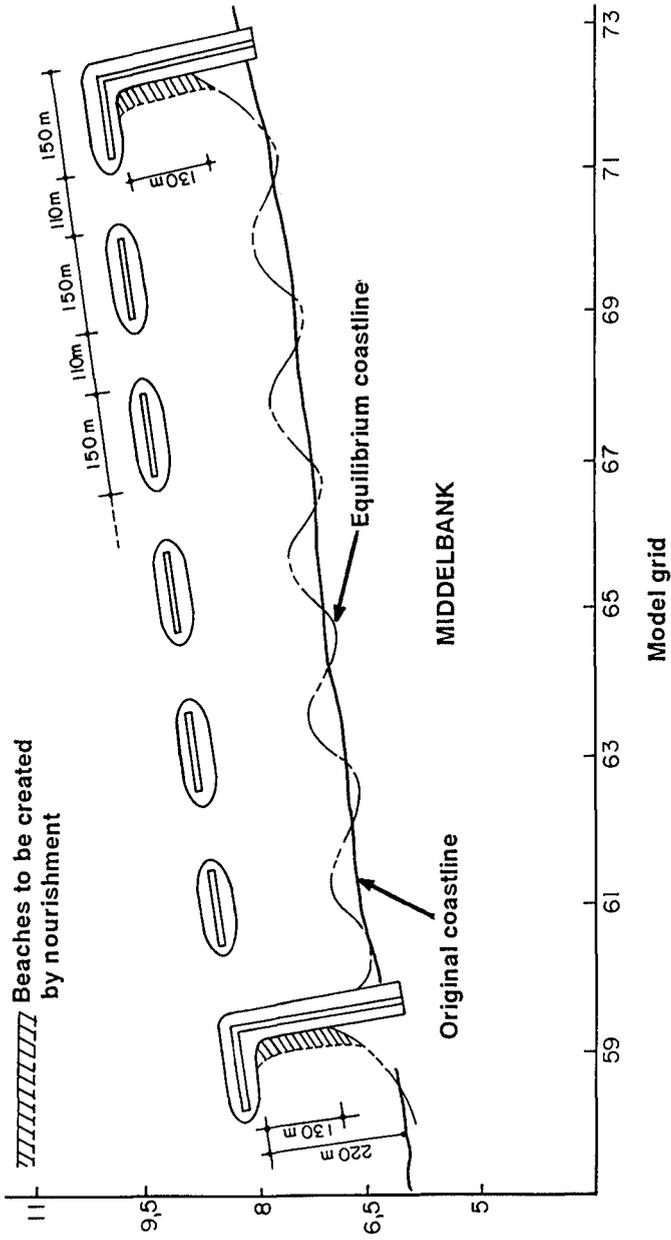


FIGURE 6b: A PROPOSED LAYOUT AT MIDDELBANK