

CHAPTER 98

DESIGN OF BREAKWATERS AND BEACH NOURISHMENT

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Abstract

The subject of this paper is to compare the design forecast and the actual performance of a coastal protection scheme combining segmented near shore breakwaters and beach nourishment.

Introduction

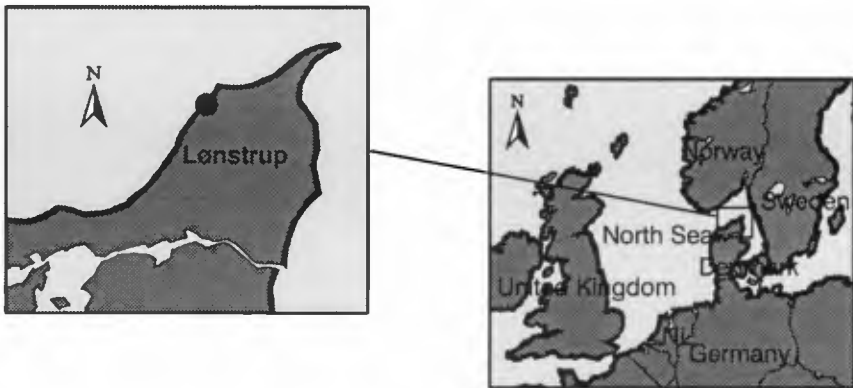


Figure 1. Area Map

The location is a 1100 m coastline at Loenstrup on the Danish North Sea coast. The long term erosion at Loenstrup is 1.4 m per year and the tidal range is 0.3 m.

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Figure 2. The Beach and the Cliff

During a storm in 1981, 5–15 m of the cliff was eroded and it was decided to protect the town.

Design Method

The design layout was a combination of near shore breakwaters and beach nourishment. The reasons behind this layout were psychological and economical. At that time there was a feeling among the local population that the main component of a coastal protection scheme should be a rigid structure and not "only" beach nourishment. The use of hard structures in this area to stabilize the beach can be justified by the fact that nourishment costs were and are quite high. The cheapest way to get the sand was to transport it by lorries from a harbour 15 km away. The project aims to stop the retreat of the profile and to protect the cliff from further erosion during storm surge. Consequently the scheme should include a revetment. Furthermore it should be taken into

consideration that the beach is used by many tourists in the summer season.

In the design phase a c/b study was carried out with the purpose to determine the combination of breakwater design and beach nourishment volumes with the lowest total costs during the lifetime of the project. The idea of the analyses is that the further off shore and the higher the breakwaters are and the smaller the gaps are, the less nourishment is needed but also the higher the construction costs are.

To do that, the costs and effects of a large number of designs of segmented breakwaters were calculated. The parameters were the breakwater height, the position of the breakwaters relative to the coastline and the gaps between the breakwaters.

For each set of breakwater parameters a wave frequency table for long term wave impact was calculated based on wave recordings and calculations of refraction, shoaling breaking and transmission. The wave decay in the breaker zone was calculated according to Goda (1975).

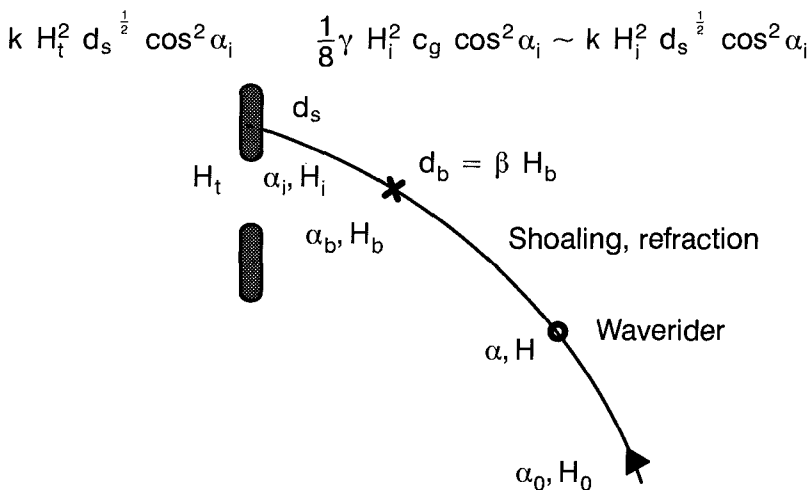


Figure 3. Wave Transformation

For each wave component the wave energy flux per m coast perpendicular to the coast is

$$1/8 \gamma H_i^2 c_g \cos^2 \alpha_i f \sim \text{constant} * f H_i^2 d_s^{1/2} \cos^2 \alpha_i$$

H_i = incoming wave height

c_g = group velocity $\sim (g d_s)^{1/2}$ at the structure

α_i = incoming wave angle

f = frequency of the wave component

d_s = depth at the structure

To calculate the transmitted wave energy flux, H_t replaces H_i . The transmitted wave height over the breakwaters was calculated according to Saville (1963). It was assumed that wave energy would pass the gaps without reduction. By use of the frequency tables, the energy reduction by the breakwaters for average weather conditions could be calculated.

We then assumed that the long term erosion of the profile segment landward of the breakwaters would be reduced with the same percentage as the percentage of energy reduction caused by the breakwaters. The still remaining erosion of the profile out to a certain closing depth should be compensated for by beach nourishment. For different combinations of breakwater parameters the associated building costs and the need for beach nourishment to compensate for erosion were calculated. Only breakwaters that could be built using land based equipment were considered which limited the water depth to be used in the study to apr. 2 m. It was clear from the start that using sea based equipment would be far too expensive.

The present value of the total costs (building + maintenance + nourishment) in the lifetime was calculated for each alternative layout.

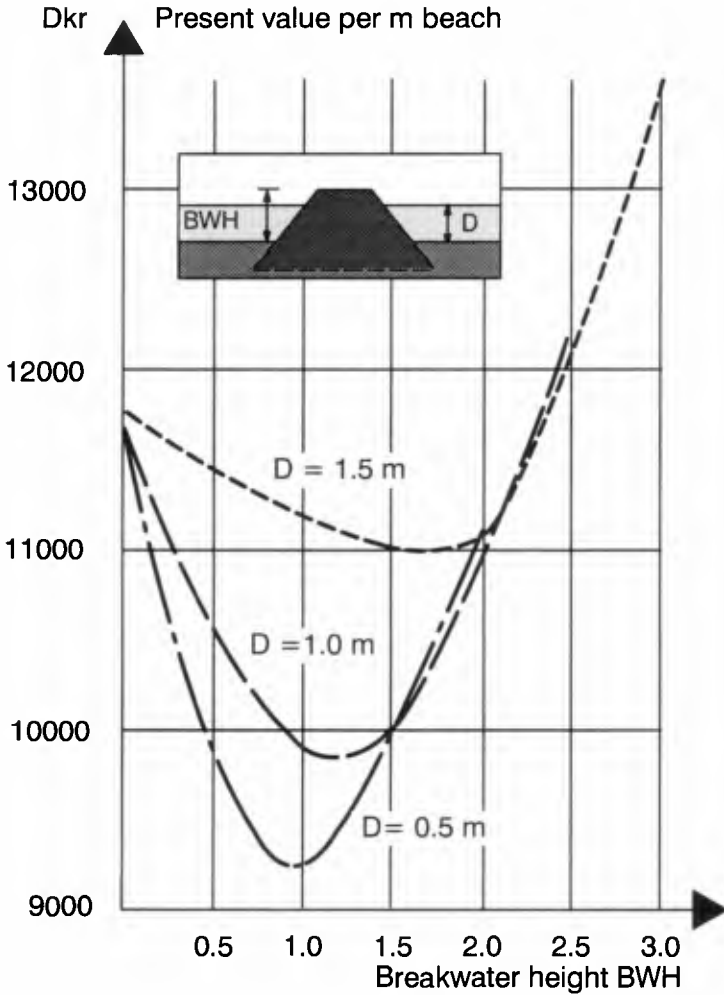


Figure 4. Present Value as a Function of Height and Depth (distance from the beach).

Figure 4 shows examples of design curves where the case of no breakwaters and only nourishment is the intersection with the y -axis. This example represents a case where 50% was breakwaters and 50% was gaps. It shows that down to a water depth of about 1.7 m there is a minimum of costs for certain breakwater heights.

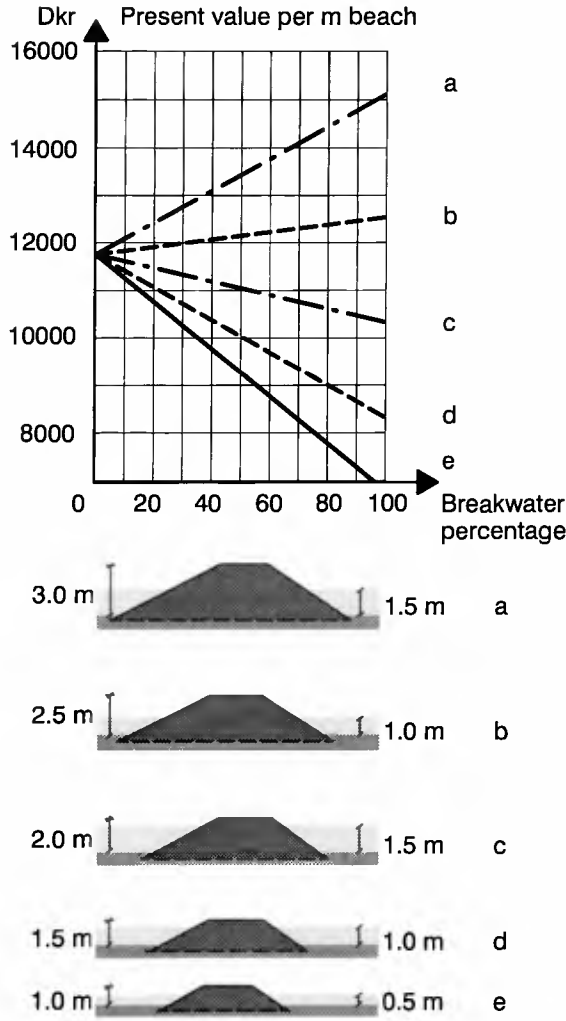


Figure 5. Present Value as a Function of the Percentage of Breakwaters Versus Total Length.

On figure 5 the variation of costs as a function of the percentage of breakwaters is presented and not surprisingly this shows a linear variation. When the total costs are below the costs of only nourishment, the most cost effective solution theoretically is to have 100% breakwaters and no gaps.

Project Layout

When selecting the practical solution we had to consider the need for a certain width of the beach for recreational use. This led to a minimum construction depth of 0.5 m. Furthermore, the gaps should at least be 50% of the total length to allow for swimming. The necessary dimension of the stones in the cover layer resulted in a breakwater height of 1.8 m above sea bed equivalent to a freeboard of 1.3 m relative to normal sea level. The total present value is read from figure 4. It is 10,650 Dkr (1,600 US\$) which is still below the case of no structures.

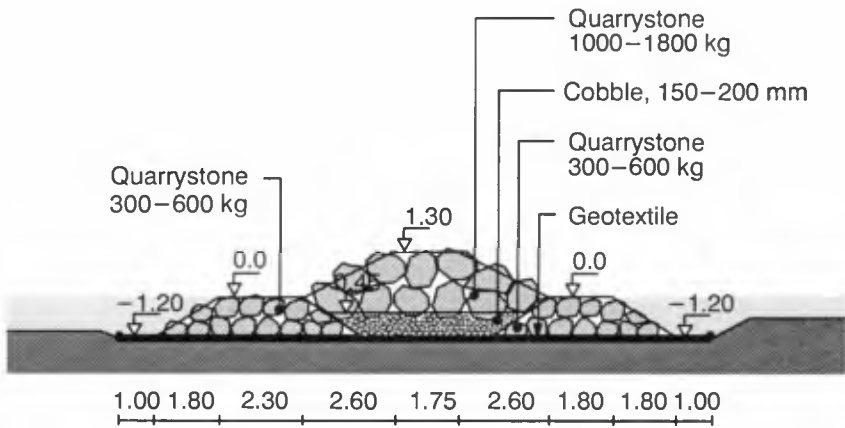


Figure 6. Cross Section.



Figure 7. Aerial Photo of Breakwater Group.



Figure 8. Single Breakwater.

Monitoring and Results

The breakwaters were built in 1983 and annual beach nourishment has been carried out since then. The annual nourishment volume has been decided by the need to maintain the beach behind the breakwaters. The profile seaward of the breakwaters was not nourished, so it could be expected that the profile would eventually stabilize in a steeper position.

To find out if the breakwaters have had the expected effect i.e. if they have reduced the erosion behind the breakwaters with the calculated percentage, a monitoring programme was set up. The chosen design should according to our calculations give a reduction of the erosion of about 49.9% in a year with normal weather conditions.

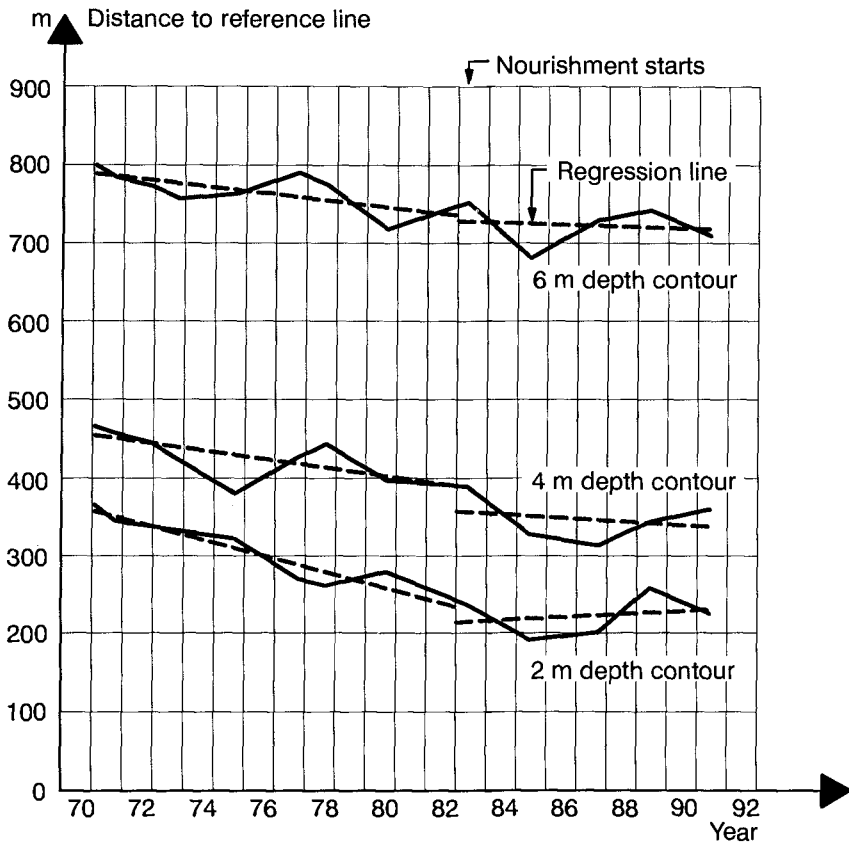


Figure 9. Time Variation of Depth Contours.

Figure 9 shows the time variation of the location of the depth contours. They seem to respond to the new situation a couple of years after the breakwaters were built and the nourishment had started. The 4 and 2 m contours seem to stabilize their position around 1985 resulting in a steeper equilibrium profile.

If the weather had been average since 1983, it would be easy to compare the design forecast of the nourishment volume with the actual annual volumes. The weather has, however deviated significantly from normal since 1983.

Elevation, cm

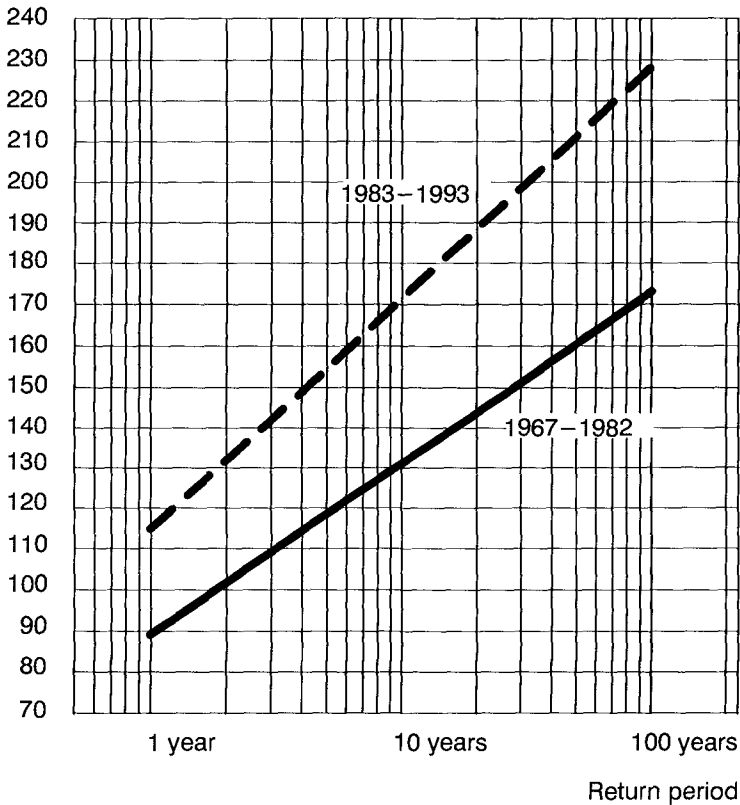


Figure 10. Statistics of Extreme Water Levels

On figure 10, the statistics of extreme water levels are shown. They show a significant difference between the period before and the period after 1983.

The question is if the actual nourishment volumes and the actual weather conditions since 1983 could be expected to result in the measured time variation of volumes in the control box behind the breakwaters when we use the design theory of energy reduction and erosion. To verify this, we have hindcasted this variation of volumes using the actual volumes, the theoretical erosion on the unprotected beach and the theory of energy reduction.

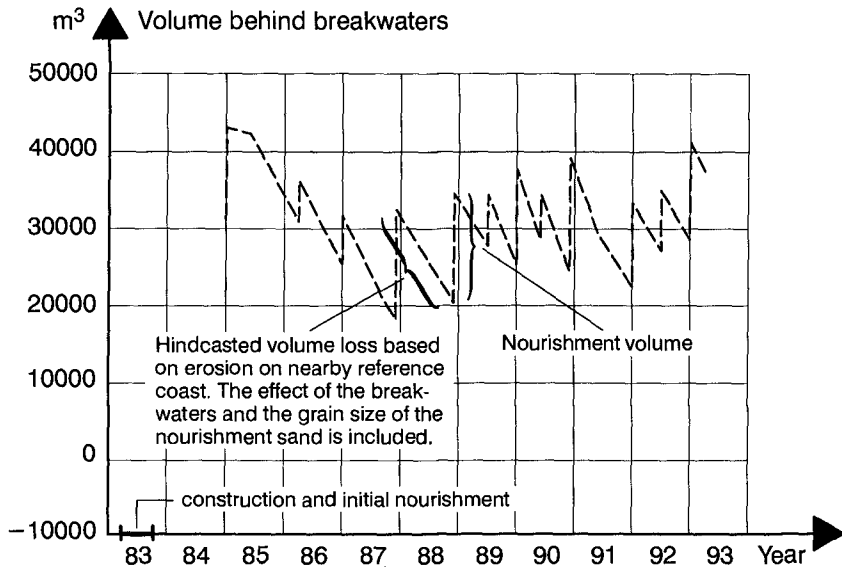


Figure 11. Theoretical Development of Volume Behind Breakwaters

This hindcast of volume development is shown on figure 11. The nearly vertical lines represent the nourishment. The length of the lines represent the actual sand volumes. The sloping lines represent the erosion that would theoretically have occurred in the periods between nourishments if the design theory of the breakwater effect is correct. The slope of the lines is calculated as the rate of erosion that would have occurred if the breakwaters had not been built. Since this rate is unknown the erosion rate of an unprotected reference beach is applied. This erosion rate is then reduced according to the theory of energy reduction caused by the breakwaters developed in the design phase. Since it is the nourishment sand and not the native sand which is eroded, a correction is made using the renourishment factor method (James 1975).

On figure 12, the hindcasted volume curve and the measured volume curve are both shown. It appears, that there is a good agreement between the curves.

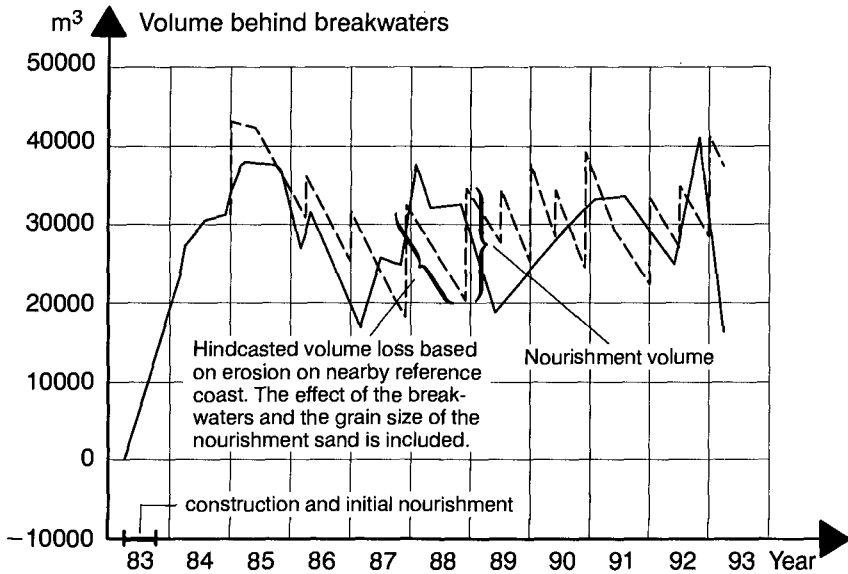


Figure 12. Theoretical(— —)and Measured Volume (——)Development Behind Breakwaters.

Conclusions

Based on measurements and hindcast calculations it can be concluded, that the effect of the near shore breakwater group in terms of reducing erosion behind the breakwaters is proportional to the ability of the breakwater group to reduce the wave energy flux perpendicular to the coast. However this conclusion should probably be used with caution in cases where the conditions differ significantly from the conditions described in this paper.

References

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