Modelling of wave penetration into small harbours

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Abstract

In situations where harbours are located in areas where rough wave conditions occur they often lie sheltered behind one or more breakwaters. The location of the breakwater should be chosen such that the ships in the harbour will be subjected to a gentle wave climate. Due to diffraction around the breakwater head and internal reflection in the harbour basin, waves can penetrate into the harbour causing an inconvenient wave climate.

In the design phase, the wave climate in the harbour can be predicted by numerical calculations and by execution of physical model tests. In this paper a number of problems that can occur when predicting wave penetration in small harbours will be discussed by means of a case study of the harbour of Fregate Island.

Fregate Island, part of the Seychelles group in the middle of the Indian Ocean, is a private island upon which a five star leisure resort is located. For further development of the resort a yacht and small craft harbour was planned, consisting of an inland harbour basin together with an entrance channel protected by two parallel breakwaters.

In the design of Fregate Harbour physical model tests of the harbour as well as calculations have been used. At that stage it was already found that significant wave action could be expected in the harbour basin. During construction several changes were implemented in order to reduce the wave penetration.

Despite the measures taken, it was found after completion of construction that the waves in the harbour basin were still unacceptable. Therefore wave penetration calculations have been made which resulted in a re-routing of one of the breakwaters.

Delta Marine Consultants (DMC) became involved in the Fregate project towards the end of 1999. DMC was asked to design the reconstruction of one of the breakwaters, which was heavily damaged within the first six months after it was constructed.

In addition to the design for reconstruction, DMC carried out an additional wave penetration study with the aim of advising on measures to be taken to reduce further the wave action in the harbour basin. In that study a numerical diffraction model was used. Following on from the wave penetration study a set of three solutions was formulated in a design.

The reconstruction of the breakwater was completed midway through 2000. The recommended measures to reduce the wave action have however not yet been implemented.

1 Introduction

The islands of the Seychelles are located in the Indian Ocean, approximately 1600 km off the coast of east Africa, just south of the Equator. The islands are the rocky outcrops of an underwater plateau without volcanic activity. Fregate Island is located about 40 km east of Mahé, which is the capital island of the Seychelles. The three main central islands of the Seychelles - Mahé, Praslin and La Digue - are granite, while the outlying islands are coral atolls.

Fregate Island extends to 200 Hectares and rises to 125 m above mean sea level. The island consists of granite and is surrounded by coral reef on which the incoming waves break.

2 Development Fregate harbour

For Fregate, an efficient and functional harbour is of great importance as apart from air transport all supplies arrive via the harbour. In the past only a small channel existed which was excavated out of the coral of the reef flat. Through this channel little boats could reach the shore, where they were pulled up on the beach.

Two monsoon periods are present in the area. The south-east monsoon (late May to September) is cooler and drier than the north-west monsoon (March to May). The south-east monsoon generates the highest waves, which can reach the harbour, and during this season large waves break almost continuously on the steep coral foreshore.

For further development of the resort on the island it was planned to build a yacht and small craft harbour. In 1996 a design was made, consisting of an onshore tidal basin excavated in the swamp area behind the beach. The harbour was connected to the sea by a new channel, excavated to a level of CD – 3.5 m. The channel was to be protected on both sides by two straight breakwaters. The southern breakwater was the longest to give more protection against the south-east monsoon. The toe of the breakwater head extends to CD –2 m.

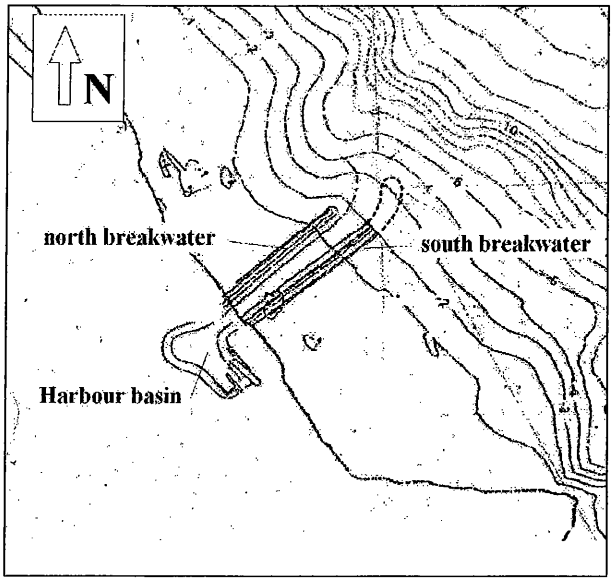

Figure 1 Initial design harbour

2.1 Numerical refraction calculations

In order to estimate the environmental conditions for the breakwater design a numerical wave refraction study had been carried out in 1996. In the study a numerical model (HISWA) was used to transform the offshore wave climate to an inshore wave climate, taking into account refraction, shoaling and wave breaking.

The modelling was carried out with a nested grid, i.e. the computational grid was fairly coarse offshore (grid spacing 200 m), whilst near the site a grid of 10 m spacing was used. In between, a grid size of 50 m was used in the calculations.

The offshore wave data used in the model were derived from VOS data, i.e. Visual Observations from Ships, which comprised a set of more than 15000 observations. It was found that the predominant offshore wave directions are south to south-east during the winter months and north to north-west during the summer. The waves resulting from the south-east monsoon have the highest wave height.

From the results of the calculations it was found that the waves near the harbour site were coming from north to north-east directions due to refraction, as they originated from south-east directions. This refraction also caused a significant reduction in wave height. The wave heights are largely dependent on the water level and are depth limited.

A resulting design wave height of H_s = 2.8 m (at a water depth of 3.6 m) has been used in physical model tests.

2.2 Physical model tests

The design with the two parallel breakwaters was tested in physical model tests on a scale of 1:50. These tests were used to optimise the layout of the approach channel and protection structure as well as to check the stability of the rock armour of the breakwaters.

During the test several design changes were made in the model. For example the rock armour weight at the head of the breakwaters was increased from 3.5-4 tons to 4-4.5 tons.

During the tests it was found that the wave motion in the harbour basin was excessive. For a wave height outside the harbour, H_s = 2 m, that is exceeded 7% of the time a significant wave height of 0.6 m was measured in the basin. This means a penetration of 30%. In the design criteria however a wave height of 0.6 m is only acceptable for the 1:50 year design condition.

To reduce the excessive wave motion in the harbour basin, a rock revetment with slope of 1:1.5, was applied in the model as side slope of the harbour basin. The following penetration amounts were measured in the model (in percentage of the wave height just outside the harbour):

Wave period	Wave penetration	
	without revetment	with revetment
8 sec	20%	10%
10 sec	25%	15%
12 sec	30%	25%

Although the revetment helped in reducing the wave movements in the harbour basin, it was found to be less effective for the wave periods that caused the most penetration.

All physical model tests were conducted with smooth side slopes in the entrance channel, a factor which was considered to exaggerate the wave penetration into the basin. In reality the sides would be constructed of rock and therefore the amounts of wave penetration would in reality be expected to be less than those found in the model tests.

In one of the final model tests a crescent spending beach with slope of 1:6, was constructed in the harbour basin to evaluate the effect of this measure on the wave heights in the harbour basin. The measured reduction in wave penetration was about 5%. For improvement of this solution a rip rap slope was recommended as the sand slope of 1:6 was not found to be effective in dissipation of wave energy.

This option was however not further developed out in a design.

2.2 Design changes during construction

The design was implemented with the original layout, i.e. with two parallel breakwaters. The breakwater heads were constructed of rock armour with weight 4-4.5 tons.

During the construction it was found that the waves were still too high in the entrance channel and the harbour basin. Therefore the southern breakwater was lengthened by 25 m. Although in the original design the toe of the breakwater head was to be founded at a depth of CD –2 m, due to the lengthening it was would be located at a level of CD –3 m.

As the design wave height increased to H_s = 3.9 m the design was changed to 5-7 tons rock at the breakwater head. However, as the heaviest rock size available was 4-4.5 tons, the same rock weight was applied as that at the head of the original breakwater design.

The toe of the breakwater head as completed in 1999 was located at the CD –4 m contour.

During excavation of the channel, the side slopes were found to be unstable. The slopes of the entrance channel were therefore protected by a rock revetment over the full length of the channel. It was expected that this revetment would also contribute to decrease of wave penetration.

2.3 Numerical study wave penetration

After execution of the major part of the harbour works, the wave action in the harbour basin was still found to be unacceptable. As found in the physical model tests some improvement of the inner wave climate resulted from a re-routed northern breakwater, and in 1998 a study of the effect of this re-routing was carried out by a consultant.

The idea was that by creating a basin with a stilling beach between the two breakwaters, the incoming wave energy would be dissipated and thereby reduce the wave action in the harbour basin.

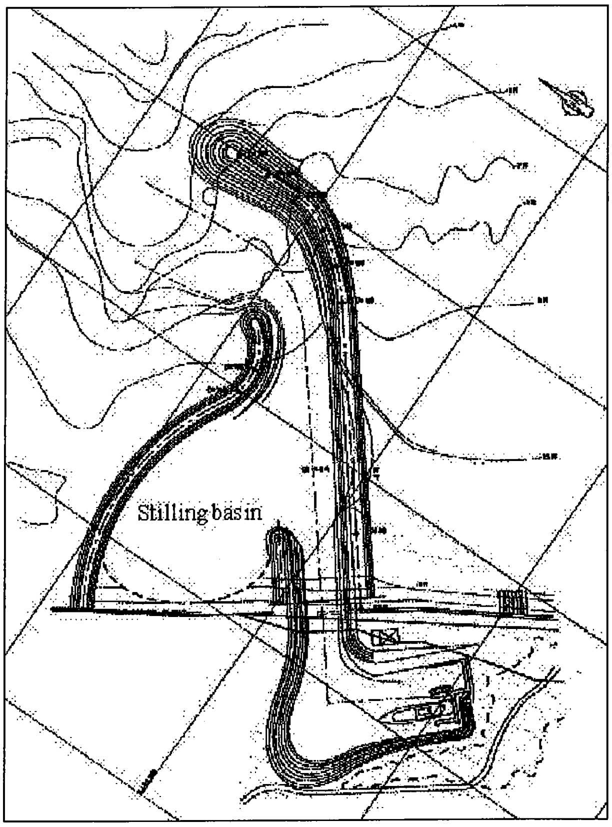

Figure 2 Re-routed northern breakwater

Numerical modelling was carried out for the existing situation (two straight parallel breakwaters) and for the stilling basin. Calculations were made for two wave periods, and furthermore a deepening of the basin to CD –3.5 m was evaluated.

Although the idea of re-routing did not show a significant reduction in previous physical model tests, it was found in the numerical calculations that the wave penetration into the harbour basin was much less. At the entrance of the harbour basin the change resulted in reduction of 25 to 50 % of the wave height compared to the existing situation.

Following from this investigation the change in layout of the northern breakwater was recommended including more gentle slopes of the revetment in the harbour basin. In 1998 the re-routing of the northern breakwater was executed.

2.4 Involvement of DMC

Within 6 months after completion of the breakwater extension DMC was asked to design the reconstruction of the south breakwater as it was heavily damaged during the south-east monsoon of 1999. The wave climate was therefore studied by means of a two-dimensional refraction model (SWAN).

It was found that the existing armour on the extended breakwater was too light to withstand the severe wave attack. For the reconstruction DMC made a design consisting of concrete units. The reconstruction of the breakwater was finalised end of April 2000.

Although the re-routing of the northern breakwater resulted in a gentler wave climate in the harbour basin, DMC was asked to investigate the possibilities of a further reduction of the wave climate.

3 Numerical modelling of wave penetration

In order to investigate the reasons and solutions for the amount of wave penetration, calculations were made using a finite element model. For this type of calculations DMC uses a two-dimensional Finite Element wave penetration model.

This model can take into account the variation in wave celerity due to variation in water depth. At the boundaries of the model wave energy absorption coefficients are applied. Where the model borders on the sea infinity elements can be applied.

During the south-east monsoon, waves approach the harbour from the east. Therefore the focus of the investigation was on waves from this direction. However the effects of variation of wave direction have also been investigated.

It was found that in the existing situation the wave amplitude in the harbour basin was higher than would be expected from diffraction around the breakwater head. The occurrence of these large wave heights is largely caused by resonance.

Figure 3 shows the decay of the relative wave amplitude (i.e. the wave amplitude at a certain location divided by the amplitude outside the harbour) as a function of the distance from the breakwater tip. The response shown in the graph results from waves with a period of 9 s.

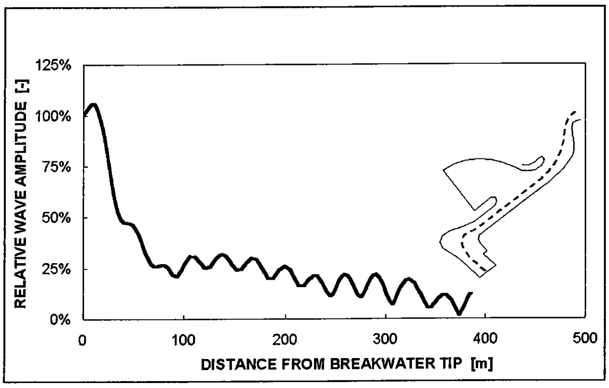

Figure 3 **Decay of wave action**

The initial rapid decay of wave amplitude is caused by diffraction. The average wave height then gradually decreases because of wave energy absorption on the breakwater slopes and loss of wave energy in the stilling basin. The local peaks are caused by wave reflection.

In order to estimate the degree of resonance, the relative amplitude has been calculated for several wave periods. For the governing wave periods it was found that strong wave excitations occurred in the harbour basin, as already observed in practice.

In order to decrease the wave penetration, the effectiveness of the following options was investigated:

1. Excavation of the (existing) stilling basin
2. Increasing wave damping on southern slope of the harbour
3. Re-routing the southern breakwater

3.1 Excavation of the (existing) stilling basin

The bottom of the stilling basin is located at 1 m below CD. The idea behind the first option was that by increasing the water depth in the stilling basin, less wave energy would be reflected against the steep coral slope at the entrance of the basin.

It was however found that this measure is not effective, as the refraction due to the large difference in water depth between the channel and the basin becomes less. This measure will therefore not contribute to a decrease of wave action in the harbour basin.

The effect of excavation of the stilling basin can be seen by comparing Figure 4 and Figure 5.

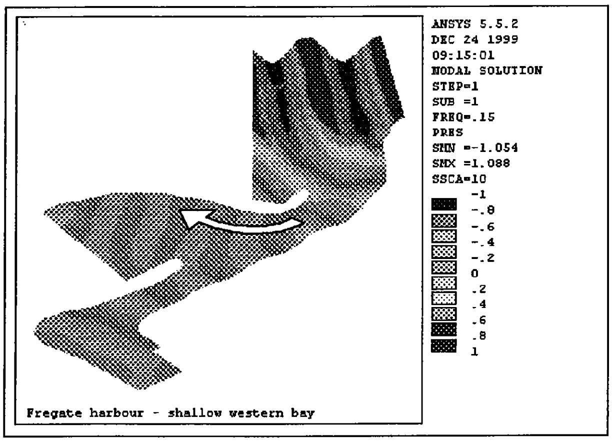

Figure 4 **Shallow stilling basin**

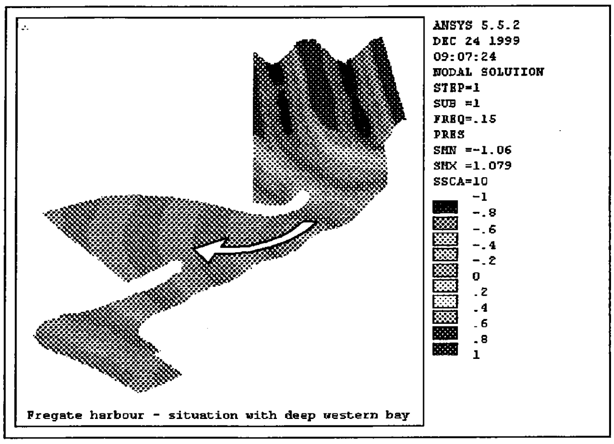

Figure 5 **Excavated stilling basin**

3.2 Increasing wave damping on southern slope of the harbour

Waves which pass through the entrance channel will reflect against the harbour slope at the end of the channel. By reducing the slope or applying more layers of rock, less wave energy will be reflected.

The effect of an absorbing slope in the harbour basin is modelled by decreasing the reflection value of the local boundary up to a realistic estimate for a decreased slope or an absorbing rock slope.

It is found that the slope angle should be decreased significantly in order to realise a significant reduction of wave action. It is however uncertain if the reduction obtained in this way is sufficient. In order to decrease the wave heights in the basin substantially, a large reduction of the slope is required. For maximum effect a small beach should be realised. This beach should consist of gravel to avoid the risk of spilling sand into the basin.

A flat sand beach is in any event not practical as there is limited space for construction of such a beach and sand can be accreting in the basin.

3.3 Re-routing the southern breakwater

Shifting part of the southern breakwater eastward, creating a new stilling basin, can significantly reduce the wave action in the harbour. This measure is more effective than the decreasing the slope of part of the harbour basin. Wave heights can be reduced by up to 50% compared to the situation with one stilling basin. From the calculations it can also be concluded that this measure is the most insensitive of all investigated measures for variations in wave period.

Figure 6 shows the calculated wave penetration in case of a re-routing of the southern breakwater.

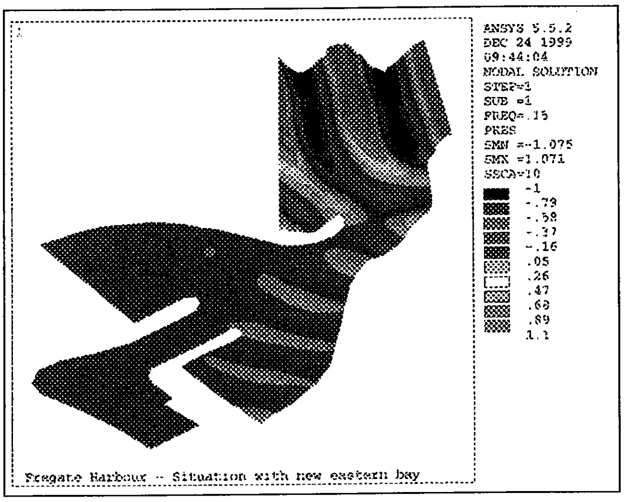

Figure 6 **Re-routed southern breakwater**

In order to estimate the effect of this measure during seasons other than the southeast monsoon, calculations were made with waves coming from other directions. It was found that also for waves coming from other directions this solution is very effective in reducing the wave height in the harbour basin.

5 Proposed measures to be taken

5.1 Re-routed southern breakwater

Re-routing of the southern breakwater is a reliable and effective solution and has been worked out in a design. The proposed layout of the harbour with a relocated southern breakwater is shown in Figure 7.

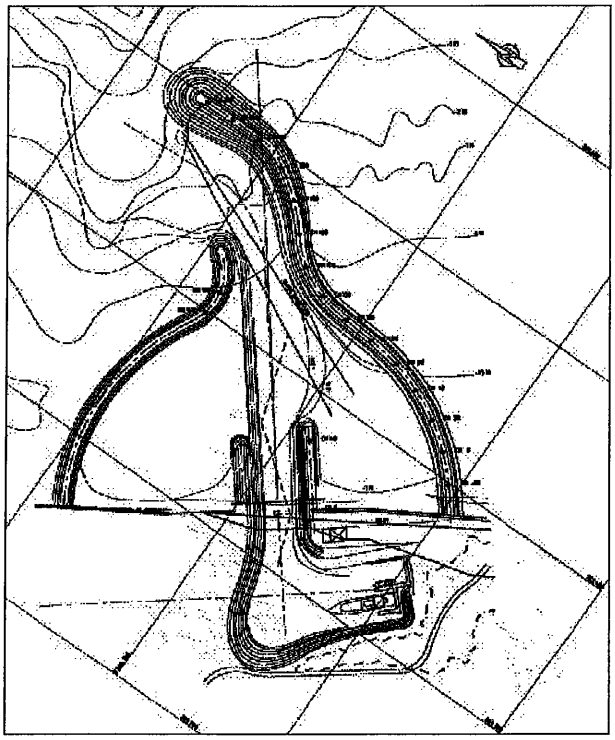

Figure 7 **Re-routed southern breakwater**

As the re-routing is a major job and therefore costly construction has not yet been carried out.

5.2 Gentle south slope harbour basin

Based on the previous study it is proposed to develop the design for the gentle slope at the southern harbour, as this is a relatively low cost option compared to the re-routing of the southern breakwater. The slope will consist of a rock grading class that remains statically stable under design conditions, to prevent deposition in the harbour basin.

The present slope is about 1:2. At the harbour side, space is limited by manoeuvring distance for the moored vessels, whilst at the land side space is restricted by a sand road, a coastal forest and a housing area. It was therefore decided to design a 1:7 slope above the CD + 0m line and a 1:3 slope below the CD + 0 m line, to conserve the available space.

Figure 8 shows an example of the proposed revetment at the south slope of the harbour basin.

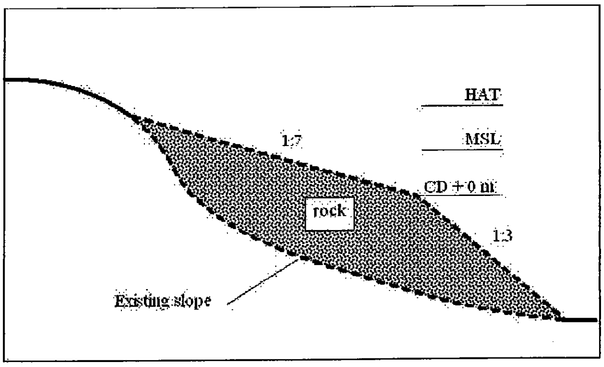

Figure 8 **south slope harbour basin**

5.3 Additional armour breakwater head

During the wave penetration calculations, execution of the reconstruction of the southern breakwater was in full progress. Due to the fact that a gentle bottom slope is present at the head of the reconstructed breakwater, DMC was asked to investigate the possibility of placement of additional armour at the breakwater head to increase the breakwater length.

The purpose of the additional armour at the breakwater head is to reduce the effect of waves refracting on the head of the reconstructed breakwater, and at the same time create a more gradual transition in wave height at the lee side of the breakwater. Furthermore the amount of sand penetrating the harbour will be reduced as sand, stirred up by wave action, is transported into north-west directions by the wave generated longshore current.

In finite element calculations it was found that a maximum reduction of wave heights in the harbour of about 20-25 % can be expected at the most unfavourable wave period (T=9 s).

The additional armour has been designed as an 'add on' extension, which means that this measure is designed to be placed against the reconstructed breakwater head.

As the toe of the additional armour lies at a greater depth than the toe of the reconstructed breakwater head, the wave attack is more severe. In order to keep material quantities and wave impacts low, the additional armour shall be located at the shallowest parts of the gently sloped area in front of the reconstructed breakwater.

Although the wave attack is more severe than at the reconstructed breakwater head, the same armour units can be applied. This is achieved by reducing the crest height of the additional armour to design water level. This will result in a reduced wave impact.

5.4 Follow-up

Although both options of a gentle harbour slope and additional armour at the breakwater have been worked out in a design, these measures have not yet been executed.

6 Conclusions

6.1 Design process

When designing a harbour, special interest should be given to wave penetration. In physical model tests for a small harbour attention should be paid to proper modelling of the boundaries as they can have a large influence on the penetration of waves. It is not sufficient to rely on an assumption of a reducing effect, such as for example a rough side slope, without further testing.

Implementing large changes in the design during the construction period should not be done without investigating the risks, as the effects can potentially be extremely adverse.

Results of numerical wave penetration calculations should be studied in detail in order to draw the proper conclusions about adverse actions and solutions to problems.

6.2 Harbour layout

Waves penetrating into a narrow channel that is protected on both sides by two straight breakwaters will cause a rough wave pattern due to reflection. Little wave energy will be dissipated by the breakwaters as they are parallel to the wave direction of the penetrating waves.

The construction of stilling basins will help in absorbing wave energy.

7 References

[1] Russel, K.S, Smith, G. and Smit, F (1996), Final report on design studies.
[2] AVECO (1998), Modified breakwater design
[3] DMC, (1999), Wave penetration study
[4] DMC, (1999), Detail design Fregate breakwater
[5] DMC, (2000), Wave penetration study