

## CHAPTER 242

### BERM BREAKWATER CONTAMINATION STUDY SERGIPE MARINE TERMINAL, BRAZIL

Charles P. Fournier <sup>1</sup>, Otavio J. Sayao <sup>1</sup>, M.ASCE and Felipe Caldas <sup>2</sup>.

#### ABSTRACT

A physical modelling programme was undertaken to investigate the contamination of a berm breakwater armour stone layer with fines. For construction purposes, it became necessary to significantly increase the berm breakwater core crest width (construction roadway) beyond the design value. Thus, contamination of the armour material by fines, much beyond the specified tolerance, immediately adjacent to the core section resulted. As the original design did not consider the influence of this contamination on the armouring stability, the present modelling study was carried out, in response to the above construction limitation. The implications of a contaminated berm armour section was investigated. The results showed that for a high crested breakwater structure, the contamination of the armouring close to the core did not influence the breakwater stability and, profile reshaping was virtually the same for the contaminated and the original structure. This is due to the fact that the contaminated section was not placed in the active berm area. However, for a low crested structure, the contaminated section was less stable than the uncontaminated counterpart, due to the increased volume of water which overtopped the structure resulting in back slope instability.

#### 1.0 INTRODUCTION

The berm breakwater under investigation is presently under construction for the Sergipe Marine Terminal in Brazil (see Figure 1). The terminal (Terminal Portuário de Sergipe, TPS) consists of a 2.4 km long trestle way connecting the berthing pier to shore. The pier is protected from wave attack by an offshore berm breakwater 543 m long, located in about 10 m depth at low water (see Figure 2).

The alternative design for the berm breakwater was developed by Construtora Norberto Odebrecht S.A. (CNO) in 1988 (Sayao and Hall, 1988). The final design specified an 11 m berm width and a 4 m core crest width both at 0.75 m above the Design High Water Level (see Figure 3). The contractor, CNO, was interested in constructing the structure by widening the core crest to 8 m, resulting in contamination of 4 m of armour material adjacent to the core. The

---

<sup>1</sup> Atria Engineering Hydraulics Inc., Ottawa, Ontario, Canada.

<sup>2</sup> Construtora Norberto Odebrecht S.A., Salvador, Bahia, Brazil.

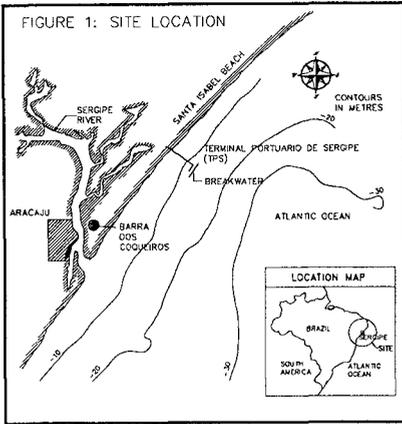


Figure 1 - Site Location

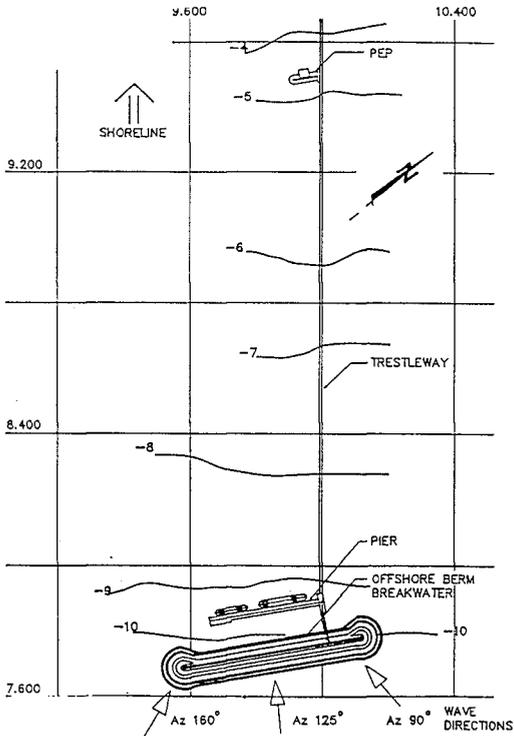
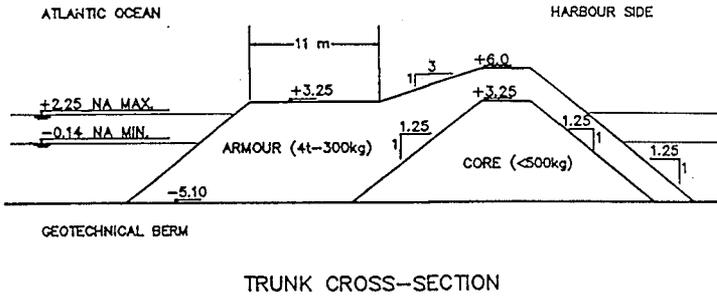


Figure 2 - Structure Layout



TRUNK CROSS-SECTION

Figure 3 - Original Berm Cross Section

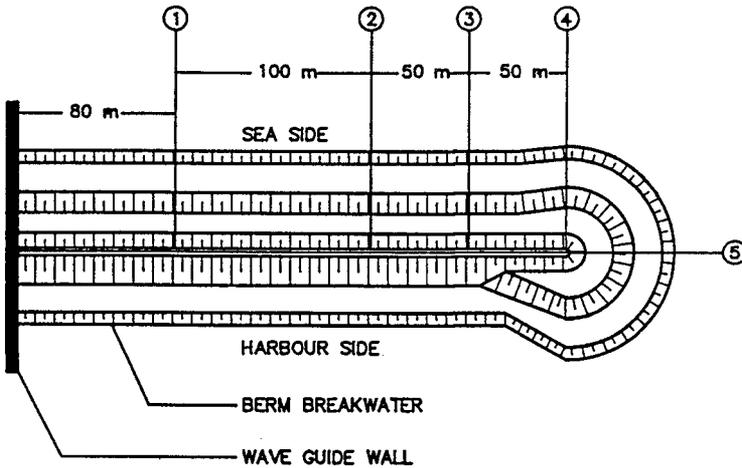


Figure 4 - Queen's University Model Layout Profile Reshaping Measurement Lines

berm breakwater stability depends on a porous berm of armour stones to dissipate wave energy. It is, therefore, essential that the expected prototype armour stone gradation and structure cross-section geometry be modelled correctly. The original design did not consider a contaminated section during the modelling programme. Thus, a physical modelling programme was undertaken at the National Research Council Canada (NRC) Hydraulics Laboratory to investigate the contamination of a berm breakwater armour stone layer with fines adjacent to the core crest. The purpose of the testing programme was to compare profile reshaping which occurred in the original (uncontaminated) design tests to those with a high percentage of fines (contamination) adjacent to the core.

## 2.0 MODEL STUDIES

This paper draws on information obtained from three separate model studies undertaken over the past couple of years (for details, see Murray and Sayao, 1990). The first, the original berm breakwater design conducted at Queen's University (Sayao and Hall, 1988), is referred to as the Queen's 3D tests. The second model study was dedicated to the investigation of armour contamination by fines and is the subject of the present paper. This model study is referred to as the NRC 2D tests. Finally, a redesign of the structure was carried out for geotechnical reasons (Murray and Sayao, 1990) and is referred to as the NRC 3D tests.

### *Queen's University 3D Design Model Tests (Queen's 3D tests)*

The design of the berm breakwater for Sergipe Marine Terminal was achieved by means of a series of three dimensional hydraulic model studies at a geometric scale of 1:35. The tests were undertaken in a wave basin at the Queen's University Coastal Engineering Laboratory in Kingston, Canada. The modelling programme consisted of a total of 24 tests, all conducted using irregular waves. Variables in the test included design water levels, wave direction, geometry of the structure and the occurrence of single and multiple design storms (Sayao and Hall, 1988). Profile reshaping measurements were taken after each storm at the 5 locations shown in Figure 4.

### *NRC 2D Contamination Model Tests (NRC 2D tests)*

A berm breakwater contamination study was conducted at the National Research Council Canada with the aid of two-dimensional physical hydraulic model tests at a geometric scale of 1:42.5 (Atria, 1989). The purpose of the testing programme was to assess the influence of a contaminated armour layer on structural stability by comparing the contaminated profile reshaping results with the ones obtained for the original design section. The design storm and prototype armour stone gradation for the present study were virtually the same as the Queen's 3D tests described above (for comparative purposes).

### *NRC 3D Redesign Model Tests (NRC 3D tests)*

In November 1989, CNO commissioned the coastal engineering redesign of the offshore breakwater, due to the weak bearing capacity of the sub-soil (Atria, 1990). The redesign breakwater was a low crested structure which allowed overtopping. The objective of the design process was to achieve a modified berm breakwater structure founded on soft clays ensuring its safe performance for both the geotechnical and the hydraulic aspects of the site.

The 3D physical model studies carried out for the redesign of the breakwater were undertaken in the National Research Council

Hydraulics Laboratory at a geometric scale of 1:35. The modelling programme consisted of hydraulic stability tests which lead to the development of the breakwater redesign for the TPS. Included in this modelling programme was a series of tests to assess the influence of the contaminated armour material immediately adjacent to the core. In fact, the approach to modelling contamination during this series of tests was conceptually identical to that used previously in the NRC 2D contamination tests discussed here.

#### *INPH Design Storm*

Details of the environmental conditions which were utilized throughout the course of the design and modelling programme are given in Sayao and Hall (1988) and Atria (1990). However, a brief review of the specific storm used for the NRC 2D tests is presented.

This design storm utilized in all three model studies was developed at the Instituto Nacional de Pesquisas Hidroviárias (INPH) in Rio de Janeiro for the original conventional breakwater design (Murray and Sayao, 1990). The individual wave train characteristics were based on prototype measurements. Table 1 illustrates the 12 segments of the INPH design storm representative of the 100 year return event (Hidroservice, 1987). Irregular waves were used throughout the course of the NRC 2D modelling programme and the influence of different sequences of water levels were considered analogous to the Queen's 3D tests. The peak significant wave height of the design storm was 4.0 m.

The maximum design high water level was specified as elevation +2.5 m above chart datum. The design low water level was specified as -0.15 m below chart datum. To achieve a comparison with the Queen's 3D tests, a low water, high water storm sequence was necessary to reproduce the test conditions in the Queen's 3D tests (Sayao and Hall, 1988).

TABLE 1: INPH DESIGN STORM

Segment	Duration (hrs)	Cumulative Duration (hrs)	Significant Wave Ht. (metres)	Peak Wave Period (sec)
1	3	3	2.5	9.3
2	3	6	3.0	9.3
3	3	9	3.5	9.3
4	3	12	4.0	9.3
5	3	15	2.5	11.5
6	3	18	3.0	11.5
7	3	21	3.5	11.5
8	3	24	4.0	11.5
9	3	27	2.5	14.3
10	3	30	3.0	14.3
11	3	33	3.5	14.3
12	3	36	4.0	14.3

#### *Armour Stone*

The model stones used for the berm armour at the Queen's 3D tests were not available for the NRC 2D tests. Thus, the geometric scale of 1:42.5 was chosen such that the armour stone distribution of the berm armour material matched (reasonably well) the armour stone distribution from the Queen's 3D tests. Figure 5 shows the comparison of the armour stone distributions for both the 2D contamination study and the original Queen's 3D tests. At this

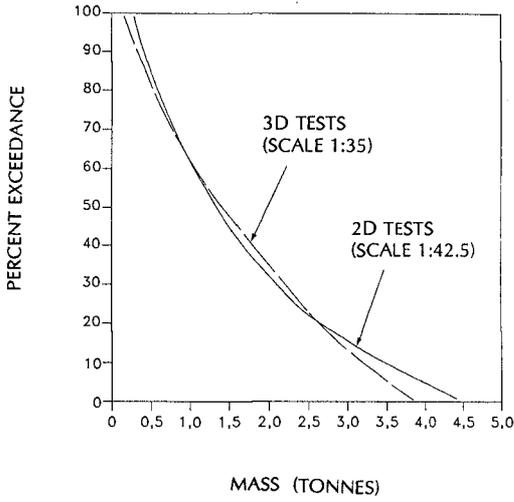


Figure 5 - Berm Armour Stone Gradation Curves

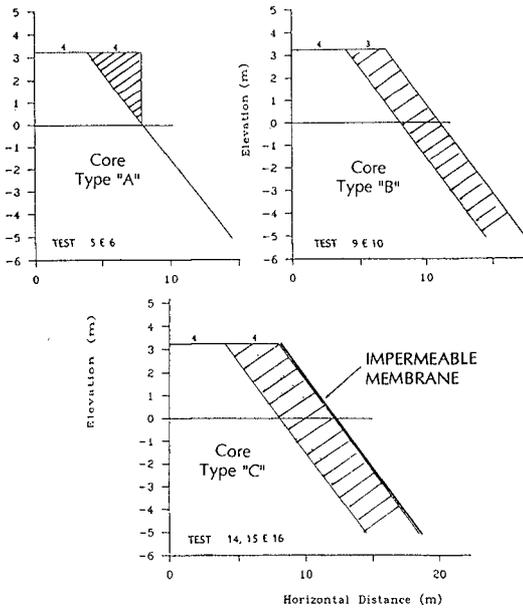


Figure 6 - Contamination Core Types

scale, the Reynolds number in the armour layer is maintained at a value exceeding  $5 \times 10^4$ , which is sufficient to minimize scale effects resulting from the inability to simultaneously model both Reynolds and Froude criteria (Dai and Kamel, 1969, U.S. Army Corps of Engineers, 1984). As can be seen from Figure 5, the armour stone gradation ranged from 300 kg to 4 tonnes. This resulted in the determination of the following parameters:

$$\frac{D_{60}}{D_{10}} = 1.65; \frac{D_{85}}{D_{15}} = 1.86; \frac{H_s}{\Delta D_{n50}} = 3.1$$

where  $D_{85}$ ,  $D_{60}$ ,  $D_{n50}$ ,  $D_{15}$  and  $D_{10}$  are the armour stone diameters exceeded by 15%, 40%, 50%, 85% and 90% of the armour stone distribution respectively,  $H_s$  is the (maximum) significant wave height and  $\Delta$  is the submerged relative density of the armour stone.

### 3.0 INFLUENCE OF ARMOUR STONE CONTAMINATION

The approach adopted for the contamination study (NRC 2D tests) was to compare the reshaping characteristics of the contaminated structure versus the uncontaminated breakwater cross section. For this, profiles 1 and 2 from the Queen's Tests (see Figure 4) were used for comparative purposes. Firstly, a calibration procedure was necessary to ensure that the reshaping of the NRC 2D structure was (practically) identical to that of the Queen's 3D structure. That is, the reshaped profile after the design storm in the NRC 2D tests should be comparable to the reshaped profile after the same design storm from the Queen's 3D tests. This calibration technique therefore addresses the issue of the 2D versus 3D modelling effects, i.e. the 2D berm width is not the same as the 3D berm width to achieve the same profile reshaping. Having achieved an acceptable calibration, contamination of the breakwater was considered by comparison of contaminated versus uncontaminated reshaped profiles.

#### *Simulation of Contamination*

The simulation of contamination immediately adjacent to the core crest was achieved by constructing (in the model) various core extensions as shown in Figure 6. Core type "A" perhaps represented a physically realistic geometry of a contaminated armour layer given a construction roadway extension of 4 m over already placed armour. Core type "B" adopted a narrower contaminated section than core type "A" with a more extensive degree of contamination over the front slope. Finally, Core type "C", considered a worst case scenario for the simulation of contamination in the physical model study. This section utilized an additional 4 m of core material extended to the base of the structure over the entire front slope of the core. Furthermore, an impermeable membrane was placed over the entire front slope of the (new) core to eliminate any permeability effects into the core.

#### *Summary of NRC 2D Tests and Results*

A summary of the 2D tests conducted at the National Research Council Canada to study the contamination of a berm armour layer is given in Table 2.

TABLE 2: TEST SUMMARY - NRC 2D CONTAMINATION STUDY

Test No.	Berm	Berm Width	Water Level	Core Type	Purpose
1	1	11.0	LW	"U"	calibration to 3D model
2	1	(11.0)	HW	"U"	calibration to 3D model
3	2	8.0	LW	"U"	calibration to 3D model
4	2	(8.0)	HW	"U"	calibration to 3D model
5	3	8.0	LW	"A"	assess contamination type "A"
6	3	(8.0)	HW	"A"	assess contamination type "A"
7	4	9.0	LW	"U"	calibration to 3D model
8	4	(9.0)	HW	"U"	calibration to 3D model
9	5	9.0	LW	"B"	assess contamination type "B"
10	5	(9.0)	HW	"B"	assess contamination type "B"
11	5	(9.0)	HW	"B"	assess contamination type "B"
12	6	8.0	LW	"UI"	calibration to 3D model
13	6	(8.0)	HW	"UI"	calibration to 3D model
14	7	8.0	LW	"C"	assess contamination type "C"
15	7	(8.0)	HW	"C"	assess contamination type "C"
16	7	(8.0)	LW	"C"	assess contamination type "C"
17	7	(8.0)	HHW	"C"	assess contamination type "C"

Legend: "U" - Uncontaminated Core  
 "UI" - Uncontaminated with Impermeable Membrane over Core  
 ( ) - reshaped profile from previous test  
 "A" - Core Type "A" LW - low water level  
 "B" - Core Type "B" LW - low water level  
 "C" - Core Type "C" HW - high water level  
 HHW - high water + 1 m

#### Calibration of 2D Contamination Tests

The calibration phase of the modelling programme consisted of modelling in the 2D flume various berm widths until the reshaped profile from the 2D tests matched those from the Queen's 3D tests. It is important to note that the same prototype armour stone distribution and design storm (wave climate) was utilized in both model studies, and therefore a 2D berm width existed which produced the same reshaped profile as in the 3D tests.

Tests 1 and 2 (low water, high water storm sequence) started with an 11 m berm width identical to the Queen's 3D tests. As expected, the berm reshaped less in the 2D testing than measured in the 3D Queen's tests. For tests 3 and 4, an 8 m berm width was used which yielded a reshaped profile very similar to that obtained in the 3D Queen's test. Tests 5 and 6 tested the same structure with contamination defined by core type "A".

Tests 7 and 8 tested a 9 m berm width which reshaped somewhat less than the 3D tests. Comparison of tests 7 and 8 results with the Queen's reshaped profile after low and high water respectively showed inadequate agreement in 2D versus 3D reshaped profiles, particularly after the high water test. It was therefore concluded that a 9 m berm in the 2D tests had more reserve capacity than the 11 m berm in the Queen's 3D tests. Nevertheless, contamination tests using core type "B" were conducted for comparative purposes between an uncontaminated structure with a contaminated structure (tests 7 with 9, 8 with 10). Although not shown in this paper, the profile reshaping for contaminated versus uncontaminated sections was (practically) identical. These tests provided a preliminary indication that contamination of the berm armour stone adjacent to the core did not influence the reshaping of the structure.

Due to the inability to model permeability in the core, it was decided to proceed testing (subsequent to test 11) with an impermeable membrane over the outer edge of the core and/or contaminated section. This effectively eliminated any debate with respect to the correct scaling of flow through the core since flow was not permitted, and thus it was guaranteed that the contamination had a maximum impact on stability.

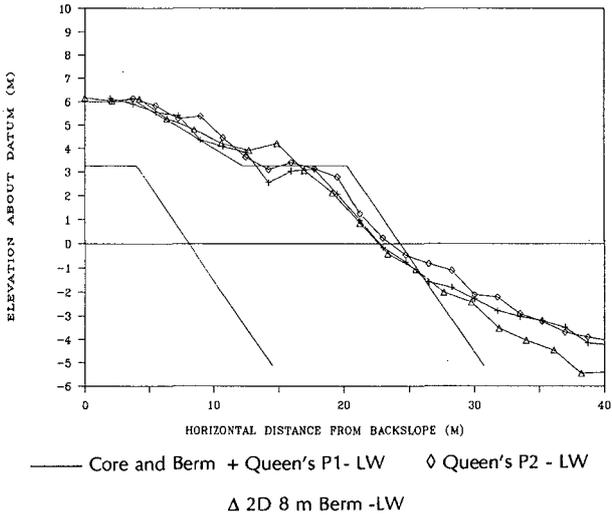
Tests 12 and 13 represented the "calibrated test section" in that the profile reshaping after low water (test 12) and high water (test 13) were very similar to the Queen's 3D tests. Figures 7 and 8 illustrate the 2D versus 3D comparison of reshaped profiles after low water (test 12) and high water (test 13) respectively. A comparison of tests 12 and 13 results with the Queen's reshaped profiles showed acceptable agreement which lead to the conclusion that an 8 m berm width in the 2D model was equivalent to an 11 m berm width in the 3D model from a stability (reshaped profile) point of view, and for the tests boundary conditions.

#### *Contaminated versus Uncontaminated Profile Reshaping*

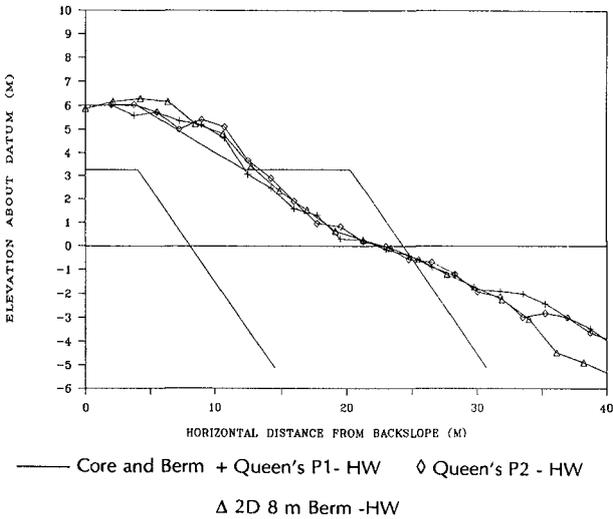
Having achieved a reasonable calibration of the 2D reshaped profile to that of the 3D Queen's tests, i.e., having considered the 2D versus 3D modelling effects, a valid assessment of contamination was achieved. Tests 14 and 15 were conducted using the same structure geometry as tests 12 and 13 with the contaminated core type "C" as shown in Figure 6. A comparison of low water and high water uncontaminated versus contaminated profile reshaping (test 12 with 14 and 13 with 15) are shown in Figures 9 and 10 respectively. It can be seen from these figures that very similar reshaped profiles resulted in both the uncontaminated and contaminated test sections, particularly at the water line. Some additional onshore transport of material towards the berm crest is visible for the contaminated section which is not an unrealistic result. Therefore, the results of the study indicate that contamination of the armour material immediately adjacent to the core as shown in core type "C" (Figure 6) does not significantly affect profile reshaping for the given structure and wave climate.

#### *NRC 3D Contamination Tests*

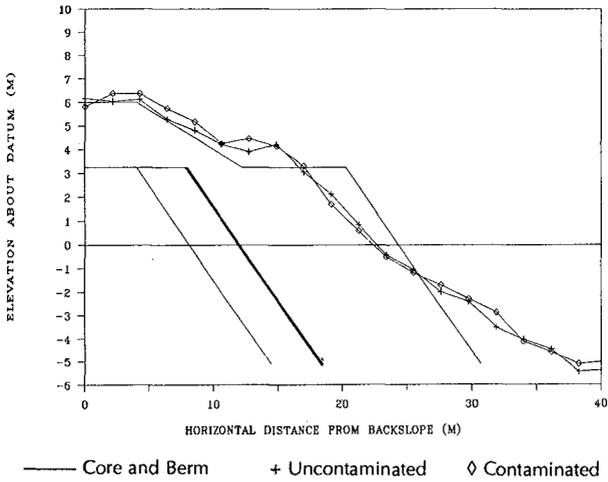
As mentioned in Section 2.0, a redesign of the breakwater was required due to geotechnical restrictions. To meet these restrictions, a low crested structure was adopted (crest 1.5 m above high water) which utilized heavier armour material (ranging from 1 to 4 tonnes) in the berm and conventionally placed armour stone over the core crest and backslope. During the course of the modelling programme, contaminated structures were tested using the core type "C" shown in Figure 6. For the redesigned structure, very little profile reshaping occurred for either the uncontaminated or contaminated structures. Figure 11 shows a typical plot of profile reshaping of the contaminated structure after a low water, high water and a second high water storm sequence. However, the contamination had a pronounced effect on the stability of the crest and backslope stones. Localized zones of damage were quite apparent (by eye) on the crest and backslope of the contaminated structure which were not present during testing of the identical uncontaminated structure. The displacement of crest and backslope stones occurred for a direction of wave attack perpendicular to the structure centerline. The effect of varying direction of wave attack was not considered during the contamination component of the structure redesign because the contamination option for construction was immediately ruled out.



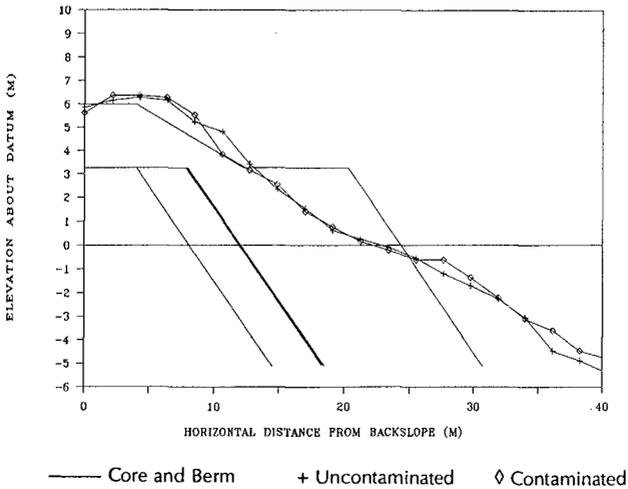
**Figure 7 - 3D versus 2D Reshaping (Low Water)**



**Figure 8 - 3D versus 2D Reshaping (High Water)**



**Figure 9 - Contaminated versus Uncontaminated Reshaping (Low Water)**



**Figure 10 - Contaminated versus Uncontaminated Reshaping (High Water)**

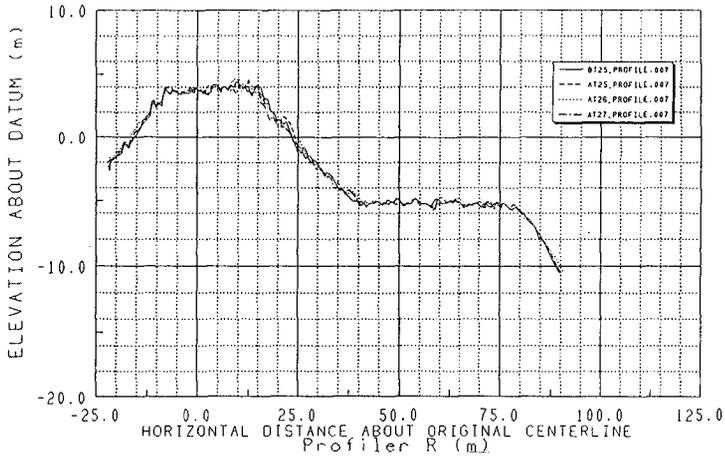


Figure 11 - Reshaping of Contaminated Redesign Structure (3D)

#### 4.0 CONCLUSIONS

The results of the present study showed that for a high crested breakwater structure where overtopping is minimum, increasing the core dimensions to facilitate construction resulting in the contamination of the armouring close to the core did not influence the breakwater stability. In this case, profile reshaping was virtually the same for the contaminated and the original (uncontaminated) structure. This was due to the fact that the contaminated section was not placed in the active berm area. However, for a low crested structure where overtopping is significant, increasing the core dimensions to facilitate construction could have a significant effect on the stability of the crest and backslope stones. During tests of this nature, the contaminated section was less stable than the uncontaminated counterpart, due to the increased volume of water which overtopped the structure resulting in back slope instability.

If the construction methodology is changed after any breakwater design, rather than investigating the increase of core width and the contamination of the adjacent armouring, an alternative armouring design could be considered. This may consist of a heavier gradation resulting in a reduced berm width and an increased core width, thus making effective use of quarry materials.

It should be realized that this study was site specific and thus more research is required to determine the permissible dimensions of core and armour extremities.

#### REFERENCES

- ATRIA ENGINEERING HYDRAULICS INC., 1990. Hydraulic Model Studies and Redesign of the Offshore Breakwater for Terminal Portuário de Sergipe. Final report submitted to Construtora Norberto Odebrecht S.A., May.
- ATRIA ENGINEERING HYDRAULICS INC., 1989. Estudo do Alargamento da Pista de Rolamento e sua Influência na Estabilidade da Berma do Quebramar do Terminal Portuário de Sergipe. Final report submitted to Construtora Norberto Odebrecht S.A., August.
- DAI, Y.B. and KAMEL, A.M., 1969. Scale Effects Tests for Rubblemound Breakwaters. U.S. Army Corps Waterways Experiment Station, Report H-69-2.
- HIDROSERVICE, 1987. Final Design, Terminal Portuário de Sergipe. São Paulo, Brazil.
- MURRAY, M. and SAYAO, O.J., 1990. Offshore breakwater for the Sergipe Marine Terminal, Brazil. Proc. 22nd Int'l Conf. on Coastal Engineering, Delft, The Netherlands.
- SAYAO, O.J. and HALL, K.R., 1988. Berm Breakwater Design for the Sergipe Offshore Terminal, Brazil. Final report prepared by F.J. Reinders and Associates, submitted to Construtora Norberto Odebrecht S.A., March.
- U.S. ARMY CORPS of ENGINEERS, 1984. Shore Protection Manual, 4th Edition, Coastal Engineering Research Center.

#### ACKNOWLEDGEMENTS

Appreciation is extended to Petróleo Brasileiro S.A. (Petrobrás) and Construtora Norberto Odebrecht S.A. for permission to publish this paper.