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OFFSHORE BREAKWATER FOR THE SERGIPE MARINE TERMINAL, BRAZIL

MALCOLM MURRAY¹, MICE and OTAVIO J. SAYAO², M.ASCE

ABSTRACT

This paper describes the development of an alternative design for the offshore breakwater of the Sergipe Marine Terminal (Terminal Portuário de Sergipe, TPS), in northeast Brazil. Earlier alternative designs for the breakwater are presented and the chosen solution, a berm breakwater design, is described. Details of the hydraulic model testing carried out during the designing stages are given and the construction methods are explained together with a brief overview of construction controls and organization. In order to accommodate the unfavourable subsoil conditions which were found to be worse than initially expected, a redesign of the offshore breakwater was carried out. The new redesign is presently under construction. The TPS breakwater is the first berm type structure to be constructed in Brazil and one of the few in the world to be constructed over a soft marine clay layer.

1. INTRODUCTION

The growing industrialization of the state of Sergipe led to the need for the construction of a port to handle solid and liquid bulk cargo near the industrial pole to be established to the north of Aracaju, the state capital. Engineering feasibility studies (e.g. Sondotécnica, 1978) comparing an estuarine port on the Sergipe River and an offshore terminal located 17 km north of the river led to the adoption of the latter. With a total forecast investment of US\$ 110 million, the implementation of the new port was started in 1986 and is due to be concluded in 1991.

A contract was signed between Petróleo Brasileiro S.A. (Petrobrás) and Construtora Norberto Odebrecht S.A. in March 1987 for the construction of the marine civil works. Figure 1 shows the TPS location and layout. The terminal consists of a pier located in 10 m of water (below chart datum), connected to the port area by an access bridge 2,400 m long and protected by a breakwater 543 m long. The 332 m quay is designed for two 15,000 DWT vessels.

² Principal; ATRIA ENGINEERING HYDRAULICS INC., 8 Stavebank Rd. N., Suite 301, Mississauga, Ontario, L5G 2T4, Canada.

¹ Senior Manager; CONTRUTORA NORBERTO ODEBRECHT S.A., Alameda das Espatódias 915, Caminho das Árvores, Salvador, Bahia, CEP 41827, Brazil.



The sea bed at the breakwater site consists of a 4 m layer of fine sand overlaying an extremely soft layer of marine clay about 8 m thick which in turn overlays another sand layer (Figure 2).

2. TENDER DESIGN AND ALTERNATIVE SOLUTIONS

Figure 3 shows the tender design (Hidroservice, 1987), a conventional design adapted to the unfavourable subsoil conditions (Lundgren and Jacobsen, 1987). It involved placing of eight grades of stones during 17 distinct construction phases. These included the placing of a sand layer within the geotechnical berm. Construction of the geotechnical berm is done with floating plant while the superstructure of the breakwater is constructed by land-based equipment using the access trestle.

According to the tender planning, within the contractual period of 24 months construction would proceed with the use of marine equipment such as split barges and floating cranes, as well as land-based equipment. Even though the tender design was subsequently trimmed by the client, it would still involve the use of marine equipment during the winter of 1989. Thus, alternative solutions were sought which would simplify construction, also by maintaining marine operations within the summer months only. A number of structural and mixed solutions were developed in-house by CNO. Here, the structural solutions would require extremely heavy elements to resist hydraulic impacts of the order of 100 t/m (Danish Hydraulics Institute, 1987).

The classic solutions of substituting the marine clay by sand drains to accelerate the dissipation of pore pressures were already eliminated by the client on economical grounds. The alternative solution chosen and developed was a berm breakwater (Figure 3). It consisted of a geotechnical berm designed to distribute loads over the subsoil (as in the tender design) and a hydraulic berm to dissipate wave energy. Table 1 summarizes the comparison between the conventional tendered design and the alternative berm design.



FIGURE 2: TYPICAL SOIL PROFILE



FIGURE 3: TPS BREAKWATER ALTERNATIVES

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3. BERM BREAKWATER SOLUTION

DESIGN CONCEPT

The design concept for the offshore breakwater, known as berm breakwater, has been reported in a number of references (Willis et al., 1987; Foster and Hall, 1987; Van der Meer, 1988). Figure 4 shows examples of recent berm breakwater projects. The features and advantages of this type of structures can be summarized as follows:

• The main armour can be constructed using lighter stones than those required for a comparable design using a conventional double layer armour.

• The structure can be constructed using relatively simple procedures which allow the relaxation of tolerances leading to better control during construction.

• The berm structure exhibits greater stability than a conventional double layer armour breakwater for a number of reasons. The berm has a high bulk porosity which allows the waves to propagate into it, dissipating their energy over a large volume. This does not occur in a conventional design since the flow into the filter and core layers is restricted due to reduced permeability, causing energy reflection and consequent instability of the armour due to down-rush.

• The berm breakwater also increases its stability as a result of wave action which tends to move stones about in the outer layers of armour. This leads to stabilized slopes of about 1:5 inclination as well as consolidation of the mound due to interlocking between the stones. As a result a stable profile is developed which is very resilient to wave action.

The adopted berm breakwater design is shown in Figures 3 and 5. The final structure dimensions was designed after carrying out hydraulic tests and geotechnical stability analyses.

Core is composed of stones in the range of 0 to 500 kg while armour stones range from 300 kg to 4,000 kg. With this simplified design, consisting of only two types of stones, the construction period was reduced from 24 to 15 months.

Item	Unit	Conventional	Berm
Sand fill	m ³	344,000	0
Rock quantities	m ³	735,000	587,000
No. of material grades	-	8	2
No. of construction phases	-	17	4
Width of geotechnical berm	m	169	123
Acceptable percentage of damage	ષ્ઠ	4	0
Maximum settlement	cm	150	120

Table 1 Breakwater Design Quantities

Figure 3 shows cross-sections of both alternatives.



DESIGN WAVE HEIGHT (m)

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OFFSHORE BREAKWATER

FIGURE 5: CONSTRUCTION PHASES



HYDRAULIC MODEL TESTING

The berm design was model tested at the Coastal Engineering Laboratory, Queen's University, Canada (Murray, 1989; Sayao and Hall, 1988). The model structure was subjected to a wide range of wave conditions including two different design storms (DS1 and INPH, see Table 2). Testing was undertaken using the notion of a design storm which had wave characteristics during the peak segment of the storm corresponding to the 100 year wave event. Waves from 2 directions, orthogonal and 35° oblique attack, were tested with both high and low water levels. Δ summary of the design parameters is given in Table 3.

The structure was also tested to assess its performance when subjected to multiple consecutive design storms. The most intensive testing involved subjecting the breakwater to three consecutive design storms undertaken at low water, high-water and low-water respectively or to the 100 years storm at low water; two consecutive following sequence: 100 year storms at high water; 24 hours (prototype) of waves having a height equal to the 100 year height. The final design was successful in surviving these events, both along the trunk and at the heads of the structure. Figure 6 shows a typical profile development after physical model testing (Murray, 1989).

4. CONSTRUCTION

TEMPORARY LOADING FACILITY

One of the first activities to be carried out during the construction phase of the project was the construction of the temporary loading facility (Porto de Embarque Provisório, PEP) which was accomplished during the winter of 1988 (Figure 7). This facility was constructed alongside the access trestle, at 800 m from its abutment, in 5 m of

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water at low tide. Its breakwater was designed for waves with a recurrence period of 5 years and a significant wave height of 3.2 m. The PEP layout and a typical cross-section of the temporary breakwater are shown in Figure 8.

Table 3: Summary of Design Parameters

•	Deepwater Design Wave	≈ 6.0 m
•	Highest Recorded H, at the	30 m
	Moon Water Donth at	5.0 m
•	Mean water Depth at	1.1 5
	Structure	11.5 m
٠	Tidal Range at Structure	≈ 2.5 m
٠	Foundation Conditions	poor (soft
		clav)
٠	Design Wave at Structure	4.0 m
	Design Wave Derieds (T)	6 E. 9.10 -
•	Design wave Perious(1,)	6.5; 6;10 S
٠	Dimensionless Wave Height	
	$(H_s/\Delta D_{n50})$	≈ 3
٠	Dimensionless Wave Period	
	$(\sqrt{(\alpha/D_{-ra})}T_{-}$	22: 27: 34
	Longth of Structure	543 m
	Deligen of Scruccure	J=5 m
٠	Berm Width	11.0/13.0 m
٠	Crest Height	+ 6,0 m
•	Berm Armouring Gradation	300 kg - 4t
•	Mean Stone Weight (W.,)	15+
	mount ocone moight (mso)	1.0 0



FIGURE 7 PEP Photograph (Jan 89)

Design Storm 1 (DS1)

Segment	Duration	Cummulative	Significant	Peak Wave
	(h)	(h)	(m)	(s)
	<u> </u>		<u> </u>	
T	3	3	2.0	8.0
2	3	6	2.5	8.0
3	3	9	3.0	10.0
4	3	12	3.4	10.0
5	3	15	3.8	12.0
6	3	18	3.4	10.0
7	3	21	3.0	10.0
8	3	24	2.5	8.0

Table 2 Design Storms used for Physical Modelling

Design Storm 2 (INPH)

Segment	Duration (h)	Cummulative Duration (h)	Significant Wave Height (m)	Peak Wave Period (s)
1	З	3	2.5	93
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

Design Storm 3 (DS3)

Segment	Duration (h)	Cummulative Duration (h)	Significant Wave Height (m)	Peak Wave Period (s)
1	3	3	2.5	8.0
2	3	6	3.0	10.0
3	3	9	3.5	10.0
4	3	12	3.5	12.0
5	3	15	4.0	12.0
6	3	18	4.0	14.0
7	3	21	.3.5	12.0
8	3	24	3.5	10.0





FIGURE 8: PEP PLAN AND CROSS SECTION

QUARRY

The quarry is situated 73 km from the terminal site. Rock extraction is obtained by conventional methods with bench heights of 10 m. The rock is gneiss and granite of sound quality. Fines are separated at the quarry by a simple grizzly operation and transported to the site by 40 t lorries. A buffer stock pile is maintained at the terminal site to smooth out loading peaks. With the berm design and a suitable blasting pattern it was found possible to utilize nearly all of the quarry production, with the obvious exception of the fines and overburden material.

CONSTRUCTION SEQUENCE

The first main construction activity to be carried out in October 1988, following the winter season was to place the stones of the geotechnical berm. This task involved dumping $340,000 \text{ m}^3$ over an area of 124 m by 669 m, with a depth of water over the berm of 5.10 m.

Dumping was to be carried out in the open sea under waves and littoral currents. In order to conclude the operation in five months before the winter of 1989, weekly productions of 30,000 t would be necessary. After examining various solutions, including the use of locally available split barges, it was decided to charter a side dumping vessel of 1,100 t capacity which loaded down stone carpets on the sea bed in areas of approximately 30 m by 60 m and 0.30 m thickness. As this vessel was equipped with a dynamic positioning system, Schottel driven screws and bow thrusters, it was possible to achieve tolerances on the geotechnical berm construction of plus or minus 0.30 m vertically and plus or minus 1.0 m horizontally, leading to a substantial reduction of wastage. Figure 9 shows the phases of marine construction. The land based construction sequence is shown in Figures 5 and 9. It consists of dumping of the core and placing the armour stone by a combination of lorry tipping and skips handled by cranes. Production during these phases was expected to be about 40,000 m³ per month.

CONSTRUCTION ORGANIZATION AND CONTROL

Construction falls under the responsibility of a Construction Manager reporting to the Contract Manager, while design coordination and macroplanning are the responsibility of a specialized Engineer Manager. In order to maximise production, double shifts have been worked, with Saturday nights off and Sundays reserved for equipment maintenance. Quality control is carried out by members of the contractors staff, with a Quality Control Manager reporting directly to the Contract Manager as well as liaising with the client's supervisory staff.

Control of breakwater construction includes periodic checking of project geometry and construction tolerances, rock quality, and gradation curves for core and berm armour materials. It should be mentioned here that the design gradation curves were based on pilot curves prepared after experimental blasting had been carried out on opening up of the quarry, with the intention of optimizing quarry production.

During the design stage several experimental methods for gradation control were tried, including curves based on computer analysis of photographs of rock samples (Maerz et al., 1987). In most cases, however, gradation curves are carried out simply by weighing randomly chosen samples and ensuring that the measured curve falls within the specified gradation tolerance intervals.



FIGURE 9: CONSTRUCTION SEQUENCE

ELEVATONS IN METRES Ē SCALE

5. REDESIGN

While under construction, evidence that the soft clay layers were much weaker than expected was obtained through subsidence of the breakwater foundation. Subsequently, the breakwater project was reviewed to adapt to the new soils situation. Both hydraulic and geotechnical engineering redesigns were carried out (Atria, 1990a; Geoprojetos, 1990). In order to decrease the loading on the subsoil the hydraulic design has evolved to a low crested berm design, overtopping being reduced by a suitable increase in the hydraulic berm width as indicated in Figures 10 and 11.

The wave climate for the TPS used in the CNO original design (Sayao and Hall, 1988) was the same as given in Hidroservice (1986). Site measurements of the wave climate are still underway. Since more data are collected near the study site, more work has been carried out to examine the given design wave climate (Atria, 1990b). Even though there is evidence that the wave climate for the TPS has changed since Hidroservice (1986) work, the decision was made by Petrobrás to redesign the breakwater structure for the same wave climate as used in the original design.

Physical model studies for the redesign of the breakwater were carried out in the National Research Council Hydraulics Laboratory (NRC), Ottawa, Canada. The modelling programme included hydraulic stability tests for the breakwater structure as well as wave agitation measurements in the lee of the breakwater. Several model structures were tested under 2 different design storms (INPH and DS3, see Table 2), leading to the development of the breakwater redesign for the TPS. A typical section of the final redesign is shown in Figure 10 and a comparison with the original berm design is shown in Figure 11.

The rubble-mound redesign structure consists of an outer berm placed at a slope of 1:3 (due to geotechnical constraints), extending from the geotechnical berm elevation to the crest elevation. The berm armouring material consists of stones weighing between 1 and 4 tonnes; crest and backslope are armoured with stones of 4 to 8 tonnes, providing for protection against wave overtopping. A description of the model testing programme and test results leading to the final redesign of the TPS offshore breakwater are given in Atria (1990a).

In order to avoid the need to contaminate the hydraulic berm by the widening of the temporary access road, it has been decided to advance the cranes over the berm on a system of leap-frogging girders. Contamination of the armouring due to fines is discussed in Fournier et al., (1990).

6. CONCLUSION

The design phases for the offshore breakwater of the Sergipe Marine Terminal were presented. The TPS breakwater is one of the few in the world to be constructed over a soft marine clay layer and to accommodate for unfavourable subsoil conditions an extensive redesign was carried out. The new redesign is presently under construction.

The advantages of the adaptation of the tender breakwater design from a contractors point of view can be summarised as follows: elimination of the sand layer in the geotechnical berm, reduction of rock quantities, reduction of the number of stone gradations, simplification of the construction methods, execution of the marine phases during the summer months only and finally, reduction of the overall construction period.



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There is also the philosophical aspect that for the conventional design a certain percentage of damage of the armour layers is acceptable, whereas the berm design does not suffer 'damage' for the design conditions, thus eliminating the need for maintenance.

REFERENCES

- Atria Engineering Hydraulics Inc. (1990a). "Hydraulic model studies and redesign of the offshore breakwater for Terminal Portuário de Sergipe".
- Final Report for Construtora Norberto Odebrecht S.A., May, 115p. Atria Engineering Hydraulics Inc. (1990b). "Estudo do clima de ondas no Terminal Portuário de Sergipe". Unpublished Report for Construtora Norberto Odebrecht S.A., May.
- nish Hydraulics Institute (1987). "Sergipe terminal, alternative breakwater, phase I, conceptual design". Report for Construtora Norberto Danish Hydraulics Institute (1987).
- Dedwacht, pinse 1, conceptual design . Report for constitution Norberto Odebrecht S.A., in association with Portconsult, Consulting Engineers A/S, Copenhagen, Denmark, April.
 Foster D.N. and Hall N.R. (1987). "Natural armoured rubble-mound structures". Proc. 2nd Int. Conf. on Coastal and Port Engrg. in Developing Countries, Beijing, China, Vol. 1, p.587-601.
 Fournier C.P.; O.J. Sayao and F. Caldas (1990). "Berm breakwater contamination study, Sergipe marine terminal, Brazil. Proc. 22nd Int. Conf. on Coastal Engrg. Delft The Netherlands ASCE
- Conf. on Coastal Engrg., Delft, The Netherlands, ASCE.
- Geoprojetos Engenharia Ltda. (1990). "Apresentação da versão 6 do reprojeto do guebra-mar". Report R.220990-137-17 for Construtora Norberto Odebrecht S.A., March.
- Hidroservice Engenharia de Projetos S.A. (1986). "Terminal Portuário de Sergipe, projeto básico, memorial descritivo". Report HE-1072-R02-0785 for the Governo do Estado de Sergipe, Instituto de Economia e Pesquisas-INEP, July.
- Hidroservice Engenharia de Projetos S.A. (1987). "Terminal Portuário de Sergipe, final design". São Paulo, Brazil.
- Sergipe, final design". São Paulo, Brazil.
 Lundgren H. and Lindhardt Jacobsen H.C. (1987). "Breakwaters on weak soils". Proc. 2nd Int. Conference on Coastal and Port Engrg. in Developing Countries, Beijing, China, Vol. 1, pp. 779-793.
 Maerz, N.H. et al. (1987). "Measurement of rock fragmentation by digital photoanalysis". Proc., International Society for Rock Mechanics, A.A. Balkema, Rotterdam, pp. 687-692.
 Murray, M. (1989). "Projeto e construção do quebra-mar offshore do Terminal Portuário de Sergipe". Presented in the IV Encontro de Engenharia Portuária, Santos, São Paulo, August/September.
 Savao Q.J. and Hall. K.B. (1988). "Berm breakwater design for the Sergipe
- Sayao O.J. and Hall, K.R. (1988). "Berm breakwater design for the Sergipe offshore terminal, Brazil". Unpublished Report to Construtora Norberto Odebrecht S.A. by F.J. Reinders and Associates, May, 140p. Sondotécnica S.A. (1978). "Estudo da viabilidade tecnico-economica e ante
- projeto para a implantação de um terminal fluvial ou marítimo de graneis sólidos e líquidos no Estado de Sergipe". Final report, 1st. phase, Rio. Van der Meer, J.W. (1988). "Rock, slopes and gravel beaches under wave attack". Delft Hydraulics Communication No. 396.
- Willis, D.H.; W.F. Baird and Magoon, T.O. (1987). "Berm breakwaters: unconventional rubble-mound breakwaters". Proc., ASCE workshop held at the Hydraulics Laboratory, National Research Council of Canada, Ottawa, September, 284p.

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