OVERTOPPING PHYSICAL MODEL TESTS FOR THE REHABILITATION OF SINES WEST BREAKWATER

by

M.T. Reis[1], M.G. Neves[2], M.R. Lopes[3] and L.G. Silva[4]

ABSTRACT

The failure of the West Breakwater of the Portuguese harbour of Sines in 1978/79 is well known and much was learnt from it by the scientific and technical communities. The paper presents an historical perspective on the design, construction, failure and rehabilitation of the breakwater, as well as on the overtopping physical model studies performed to check the effectiveness of the different proposed solutions for its rehabilitation.

Studies are now being conducted for Sines Port Authority for the final rehabilitation of the breakwater, with the primary objective of reactivating Berth1 but also to generally improve the shelter and operating conditions within the port. The paper presents the three proposed solutions for the West Breakwater, which differ mainly in the crest area. It describes the two-dimensional physical model tests of stability and wave overtopping carried out in 2008 at the National Civil Engineering Laboratory, Portugal, to check the effectiveness of the three proposals. The mean overtopping discharges measured in the tests for Solution 3, having a higher crest level with a recurved concrete superstructure, were significantly lower than for Solutions 1 and 2 (which were similar). All three proposed solutions were structurally stable under design conditions.

The paper also illustrates the application of a new version of the NLSW numerical model, AMAZON, and of the methodologies/tools recommended in the EurOtop overtopping manual to study the mean wave overtopping discharge over the breakwater. There was good agreement between the physical model data and the AMAZON results for Solutions 1 and 2, although AMAZON tended to slightly over-predict the discharges, especially for Solution 2. The CLASH Neural Network mean overtopping discharges tended to under-predict the physical model results, especially for Solution 3. For the three solutions, the physical model results were within the confidence intervals obtained with the Neural Network. The Empirical Methods considerably over-predicted the mean overtopping discharges.

1. INTRODUCTION

The Port of Sines is located on the west coast of Portugal, sheltered by two breakwaters: the west and the east breakwaters (Figure 1). The West Breakwater is the main one and shelters berths for deep water oil tankers and four main terminals (for liquid bulks, petrochemical products, multipurpose and ro-ro, and LNG).

Between February 1978 and February 1979, during completion and immediately after construction of the West Breakwater, storms occurred that caused failure of almost the entire armour layer and superstructure, leading to urgent repair works during the 1980s. The last rehabilitation works were concluded in 1992.

Since 1979, a number of two-dimensional physical model studies have been carried out to analyse the stability and overtopping effectiveness of different proposed solutions for the rehabilitation of the West Breakwater. These studies included tests in the experimental facilities of Delft Hydraulics Laboratory (DHL), The Netherlands, and of the National Civil Engineering Laboratory (LNEC), Portugal (e.g. DHL, 1981b,c, 1982, 1986, 1987; LNEC, 1979, 1980, 1981, 1983a,b, 1989, 2008).

At present, only two of the three oil tanker berths are in operation: Berth 2 and Berth 3 (Figure 1). Studies are now being conducted for Sines Port Authority with the primary objective of reactivating

[1] Research Officer, Laboratório Nacional de Engenharia Civil (LNEC), Portugal, treis@lnec.pt
[2] Research Officer, Laboratório Nacional de Engenharia Civil (LNEC), Portugal
[3] Senior Consulting Engineer, CONSULMAR, Portugal
[4] Higher Research Technician, Laboratório Nacional de Engenharia Civil (LNEC), Portugal

Berth1 but also to generally improve the shelter and operating conditions within the port (CONSULMAR, 2006, 2008). These studies involve the final rehabilitation of the West Breakwater, for which three solutions have been proposed for the cross-section between Berths 2 and 1. To check the effectiveness of these solutions, two-dimensional physical model tests of stability and overtopping were performed in 2008 at LNEC (LNEC, 2008).

Following this introduction, the paper starts with an historical perspective on the design, construction, failure and rehabilitation of the West Breakwater, and on the physical model studies with overtopping measurements performed to check the effectiveness of the different proposed solutions for its rehabilitation. Then, the final rehabilitation of the breakwater is described: the three proposed solutions, their physical model testing and their numerical modelling of overtopping using the non-linear shallow water model, AMAZON (Hu, 2000; Reis *et al.*, 2008, 2009a,b), and the methodologies recommended in Pullen *et al*. (2007) for a structure like the Sines West Breakwater. The overtopping results from the physical and the numerical modelling are shown and discussed. Finally, there are some concluding remarks.

Figure 1: Port of Sines: location and current layout

2. HISTORICAL PERSPECTIVE

2.1 Design, Construction, Failure and Rehabilitation

The West Breakwater was constructed between 1973 and 1978. Originally, it was 2 km long in depths up to 50 m, protected by two armour layers of unreinforced 42 ton dolos units on a 2(V):3(H) slope (Figure 2). It was a dual-purpose structure, supporting oil pipelines as well as providing the port with shelter from the Atlantic Ocean. The concrete superstructure included a wave wall, an inner roadway and support for the oil pipelines, which were intended to serve the three oil tanker berths built on caissons and connected to the breakwater (Figure 3).

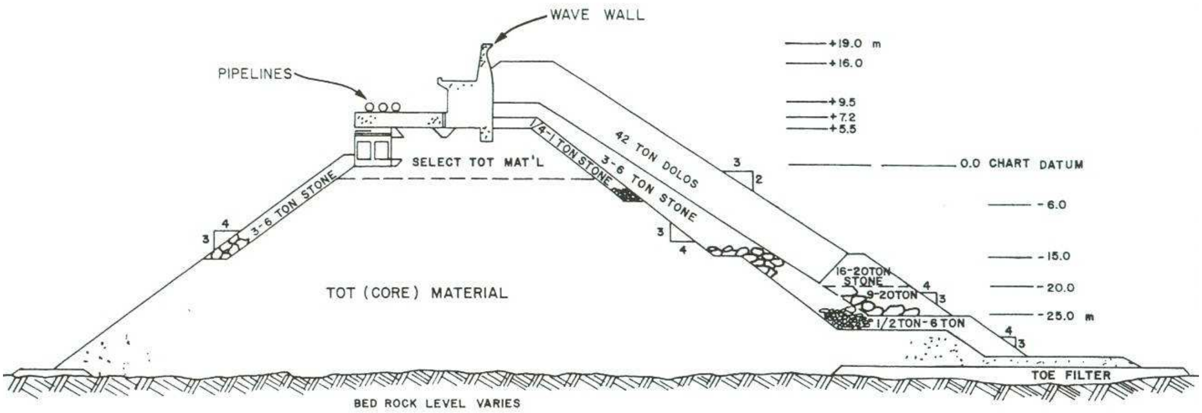

Figure 2: Breakwater cross-section of the final design (Port Sines Investigating Panel, 1982)

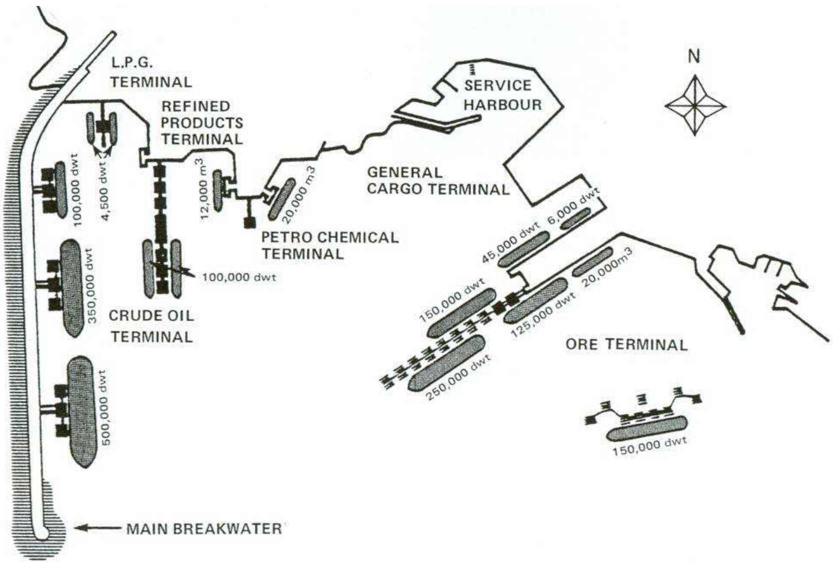

Figure 3: Port of Sines: original layout (Port Sines Investigating Panel, 1982)

The breakwater was originally designed for the 100-year return period significant wave height, H_{sd}, of 11 m (Table 1), being the largest structure of its kind in such an exposed environment. Construction was nearly complete when, on 26 February 1978, critical damage occurred during a storm with waves thought by most to be below 11 m significant wave height. There was loss of about two-thirds of the armour layer and severe damage to the superstructure at a few locations. In December 1978 and February 1979, storm action caused failure of almost the entire armour layer and superstructure (Figure 4).

Return Period (years)	Original Project & Emergency Repair (1973-1981)	Rehabilitation Works (1989-1992)	Final Rehabilitation (not implemented yet)
50	10 m	13.0 m	11.0 m
100	11 m	14.0 m	12.0 m

Table 1: Deep water design wave heights, H_{sd}: dates refer to construction period (Port Sines Investigating Panel, 1982; Dinis & Toppler, 1993; Ligteringen *et al*., 1993a,b; Abecassis & Pita, 1993; Consulmar, 2006, 2008)

Figure 4: Port of Sines: failure of armour layer and superstructure

Subsequent to failure, studies were undertaken by several hydraulic laboratories, such as LNEC, DHL, the National Research Council (NRC) of Canada and the Laboratoire Central d'Hydaulique de France (LCHF) (see section 2.2). These studies were mainly directed towards the hydraulic design and performance of the breakwater.

Several investigations were conducted to understand and identify the cause(s) of failure. The following list provides examples, noted in the literature, of possible factors that may have contributed to the failure (e.g. Zwamborn, 1979; Anon., 1979; Port Sines Investigating Panel, 1982):

- shortcomings in the selection of design waves (conception deficiency);
- communication problems between owner-designer-contractor (deficiency in project management);
- differences between the constructed breakwater and the design specifications (deficiency in construction and in construction supervision);
- the breakwater was hit by storms when construction was not fully complete (inevitable);
- low structural strength of Dolosse (structural deficiency);
- removal of the 16 ton-20 ton toe stones (structural deficiency).

Failure was most probably a consequence of a combination of different factors, some more relevant than others. It led to urgent repair works to protect and reactivate Berths 2 and 3 (the so-called Emergency Repair, carried out between Berths 3 and 2 just after the accidents of 1978/79) and to the subsequent rehabilitation works, performed between 1989 and 1992 (Figure 5). These included the rehabilitation of the Root Portion of the breakwater (the length of breakwater nearest to the shore, also known as the Dolosse Portion, north of Berth 3), improvements to the Emergency Repair works, the rehabilitation of its Outer Portion (the stretch beyond the Emergency Repair towards the south) and the construction of the New Head. The layout of the breakwater after the last rehabilitation works is shown in Figure 6.

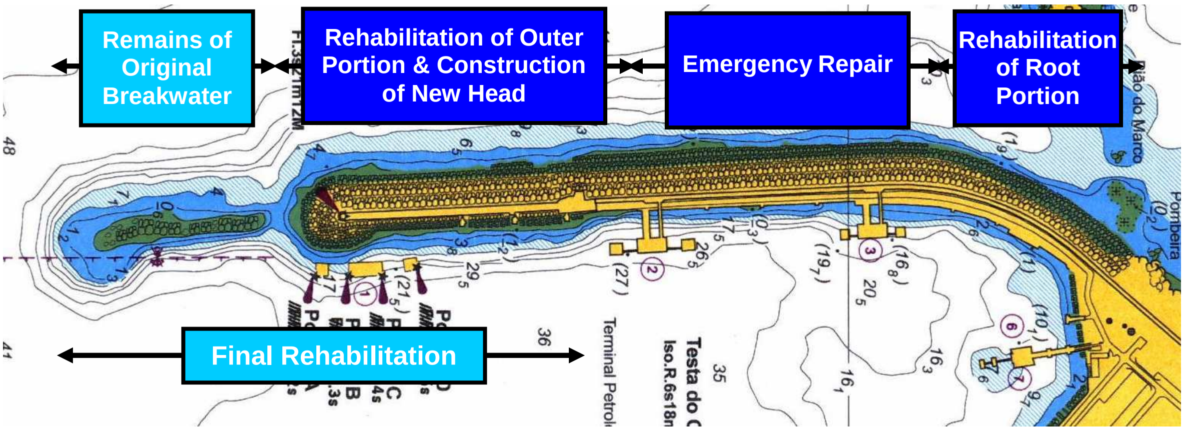

Figure 5: Port of Sines rehabilitation works

Figure 6: Sines West Breakwater: layout after last rehabilitation works (including remains of the original breakwater)

At present, the breakwater has a reduced crest length of about 1.6km and it is protected by Antifer cubes of 400 kN at the Root Portion, of 900 kN along the trunk and of 900 kN and 1050 kN at the New Head. Only Berths 2 and 3 are in operation at the moment; Berth 1 is not accessible, is directly subject to wave action and, with its foundation at -38 m (CD), is covered by the rehabilitation works up to the -15 m (CD) level. The damaged but stabilised area south of Berth 1 has never been reconstructed. Studies are now being conducted for Sines Port Authority for the Final Rehabilitation of the breakwater (Figure 5). The primary objective is to reactivate Berth1, for reception, loading and unloading of oil tankers (400 000 DWT), but also to generally improve the shelter and operating conditions within the port.

2.2 Physical Model Tests

After February 1978 and before the last rehabilitation works started, several physical model studies were performed to analyse the stability and, in some cases, the overtopping of different solutions for the rehabilitation of the Sines West Breakwater. Table 2 summarizes most of these studies: in bold are those studies which have included overtopping measurements.

Breakwater Section	Laboratory	Used Reference	2D/3D	Stability/Overtopping
Root Portion	DHL	DHL (1987)	3D	Stability
	LNEC	**LNEC (1989)**	**2D**	**Stability & Overtopping**
Emergency Repair	LNEC	LNEC (1979)	2D	Stability
	NRC	LNEC (1980)	2D & 3D	Stability
	LCHF	DHL (1987)	2D	Stability
	LNEC	**LNEC (1981)**	**2D**	**Stability & Overtopping**
	DHL	**DHL (1981b)**	**2D**	**Stability & Overtopping**
	DHL	DHL (1981c)	2D	Stability
	LNEC	**LNEC (1983b)**	**2D**	**Stability & Overtopping**
Transition between Emergency Repair & Outer Portion	DHL	DHL (1987)	3D	Stability
Outer Portion	**DHL**	**DHL (1982)**	**2D**	**Stability & Overtopping**
	LNEC	**LNEC (1983a)**	**2D**	**Stability & Overtopping**
	DHL	DHL (1986)	2D	Stability
Transition between Outer Portion & New head	DHL	DHL (1986)	2D	Stability
New Head	DHL	DHL (1987)	3D	Stability

Table 2: Physical model tests of stability and overtopping carried out for Sines West Breakwater before last rehabilitation works

The subsequent sections briefly describe the overtopping tests, highlight the main differences between them and compare overtopping results from similar tests performed both at LNEC and DHL.

2.2.1 Emergency Repair

The first physical model studies carried out after February 1978 related to the emergency repair works and were carried out by LNEC, NRC and LCHF in the course of these works (LNEC, 1979, 1980; DHL, 1987). They will not be described hereafter since they did not include overtopping measurements.

After the Emergency Repair was concluded, additional hydraulic investigations were performed by LNEC and DHL to study further improvements/modifications to the Emergency Repair (LNEC, 1981, 1983b; DHL, 1981b,c). They included overtopping measurements.

The DHL (1981b) model tests were carried out between July and August 1981 in a 2 m wide, 100 m long, wind-wave flume. The geometrical scale was 1:78. The foreshore was reproduced by a concrete bottom starting at - 53 m (CD) up to a level of - 34 m (CD) behind the breakwater.

The model structures corresponded to the original design and to the Emergency Repair. For the Emergency Repair, tests were performed for a Standard Solution and for five different cross-sections (Alternatives I-a to I-e), which corresponded to small adjustments in the design of the crest berm. For the original design and for Alternative I-b, there were no overtopping measurements. The model armour layer of the Emergency Repair was composed of Antifer units made of an aluminium alloy.

The wave conditions for the overtopping tests were characterised by a Pierson-Moskowitz spectrum with some minor peak enhancement (as in a JONSWAP spectrum), with a peak period, T_p, of 20 s. Random phase groupiness was applied. The design wave height, H_{sd}, was 14 m and the maximum significant wave height tested was 16.8 m ($1.2H_{sd}$). The water level was kept constant at + 4.0 m (CD) (corresponding to a high water level plus storm surge). Each test comprised five runs, each of 12 hrs (in the prototype), and was performed with increasing H_s values from 5.6 m (40% of H_{sd}) to 16.8 m.

For the Standard Solution, four tests were performed: i) test with a groupiness factor, GF, of 0.39; ii) test with GF=0.65; iii) test with GF=0.69; and iv) test with GF=0.60 and wind. For the alternative solutions, the tests were carried out for GF=0.60 and included wind.

Overtopping was evaluated during 20 minutes (2.94 hrs in the prototype) in two different ways: by visual observations of the number of waves that overtopped as green water and by measuring the total overtopping volume collected in a basin located directly behind the superstructure. The results showed that the wind and the groupiness factor strongly affected the number of overtopping waves. Wind increased overtopping considerably as well as the increase in groupiness factor. The cross-sections with adjustments in the crest berm and Alternative I-e, with greater crest level, showed a smaller quantity of overtopping (maximum mean overtopping discharge of 6.7E-2 m^3/s/m).

The LNEC (1981) model tests were carried out between June and July 1981. Their main goal was to study the influences of the wave period, the water level and damaged armour units on the stability and overtopping of the Emergency Repair as built at that time. The tests were undertaken in a 1.6 m wide (1.0 m operating width), 50 m long, irregular wave flume. The geometrical scale was 1:85. The foreshore was reproduced by a concrete bottom over a prototype distance of 1500 m, from - 60 m (CD) to - 40 m (CD) (the toe of the structure): a 1:50 slope up to – 50 m (CD) and a 1:70 slope up to – 40 m (CD).

The model structures corresponded to the original design and to the Emergency Repair. For the Emergency Repair, tests were performed for a Standard Solution, for two different alternatives (Alternative A and A-1), which corresponded to small adjustments in the design of the crest berm and for a Standard Solution with damaged units in the top layer. The armour layer was composed of Antifer units made of concrete.

The wave conditions were again characterised by a Pierson-Moskowitz spectrum with some minor peak enhancement, with peak periods of 16 s, 20 s and 24 s and with GF varying from 0.60 to 0.70. The significant wave heights of 5.6 m, 8.4 m, 11.2 m, 14 m and 16.8 m were employed with two different water levels: 0.0 m (CD) and + 4.0 m (CD). Each test comprised five runs, each of 12 hrs (in the prototype), with the five different significant wave heights. If after the last run the damage was small, the run was extended for another 12 hrs.

The number of overtopping waves was measured using a wave gauge. Overtopping quantities were measured by collecting water in an overtopping tray 30 cm wide by 106 cm long.

Overtopping started for H_s=11.2 m, and for H_s=14 m overtopping was very severe. The test carried out for the Standard Solution, for a water level of + 4.0 m (CD) and for T_p=20 s showed the highest mean overtopping discharge (1.8E-1 m^3/s/m for H_s=14.0 m), with the overtopping tray being insufficient to measure the volume of overtopping for the highest significant wave height.

Table 3 summarizes the main differences between the tests undertaken at LNEC and at DHL. The table shows that one of the tests was carried out for the same wave and water level conditions in both laboratories: the test for the Standard Solution and T_p=20 s. Figure 7 presents a comparison of the mean overtopping discharges measured for this test at both laboratories, together with the results for the corresponding DHL test performed with wind. The results of the DHL tests suggest that the influence of the wind was not very important. However, a significant difference was obtained between LNEC's and DHL's overtopping discharge measured for H_s=14.0 m, with LNEC's measurement being almost three times that of DHL.

Test	LNEC				DHL				
Scale	1:85				1:78				
Overtopping	over 3 hrs				over 3 hrs				
N. of overtopping waves	with a wave gauge				counted visually (green water only)				
total volume	collected				collected				
Alternatives tested		**Water level**	**GF**	**T_p**			**Water level**	**GF**	**T_p**
	Standard	+ 4.0 m (CD)	0.6-0.7	20 s	Standard		+ 4.0 m (CD)	0.65	20 s
	Standard	+ 0.0 m (CD)	0.6-0.7	20 s	Standard	with wind	+ 4.0 m (CD)	0.60	20 s
	Standard	+ 4.0 m (CD)	0.6-0.7	16 s	Standard		+ 4.0 m (CD)	0.69	20 s
	Standard	+ 4.0 m (CD)	0.6-0.7	24 s	Standard		+ 4.0 m (CD)	0.39	20 s
	Alternatives A, A-1 & Standard with damaged units	+ 4.0 m (CD)	0.6-0.7	20 s, 24 s	Alternatives I-a, I-b, I-d & I-e	with wind	+ 4.0 m (CD)	0.60	20 s
Armour units material	Concrete				Concrete with an aluminium alloy				

Table 3: Emergency Repair: characteristics of the overtopping tests performed at DHL and at LNEC

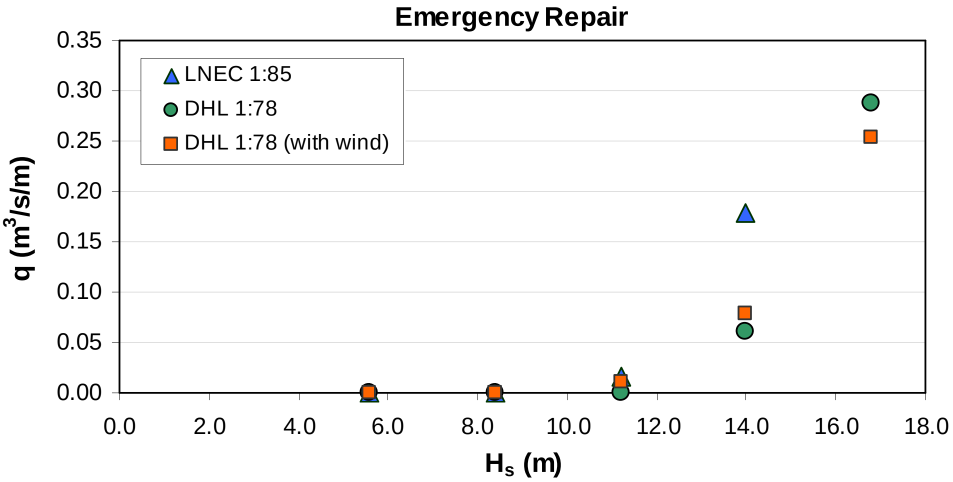

Figure 7: Mean overtopping discharges (prototype values) for tests carried out at DHL (with and without wind) and at LNEC (without wind) for the standard cross-section of the Emergency Repair for T_p=20 s and + 4.0 m (CD)

LNEC (1983b) carried out additional tests between September and November 1983 in the same flume and using the same 1:85 geometrical scale for the Emergency Repair and for two alternative cross-sections with Robloc units at the breakwater crest berm. These two alternatives differed in the crest berm level: + 19 m (CD) in Alternative I and + 21 m (CD) in Alternative II.

As before, the wave conditions were characterised by a Pierson-Moskowitz spectrum with some minor peak enhancement, this time with peak periods of 20 s and 24 s for the Emergency Repair and for Alternative I and of 20 s for Alternative II. The groupiness factor varied between 0.60 and 0.70. Three significant wave heights were tested (9.8 m, 11.9 m and 14.0 m) for a water level of + 4.0 m (CD). Each test comprised three runs, each of 12 hrs (in the prototype).

The number of overtopping waves and the overtopping volumes were each measured using a wave gauge. Additionally, the number of waves that overtopped the superstructure was counted visually. Overtopping quantities were measured by collecting water in an overtopping tray, 30 cm wide by 106 cm long. Overtopping started for H_s=9.8 m, and for H_s=14 m overtopping was very severe. As far as overtopping is concerned, the behaviour of the three solutions was similar, except for T_p=20 s, with Alternative I being the least overtopped structure (maximum mean overtopping discharge of 1.1E-1 m^3/s/m).

2.2.2 Outer Portion

To study the rehabilitation of the Outer Portion of the West Breakwater, DHL (1982) and LNEC (1983a) performed physical model studies that included overtopping measurements.

The model tests carried out by DHL (1982) were undertaken between April and July 1982 in the same flume and with the same 1:78 geometrical scale as the tests performed for the Emergency Repair. A cross-section near Berth 1 was chosen as a typical cross-section for the Outer Portion including the average foreshore perpendicular to the breakwater. The foreshore was reproduced by a 1:60 concrete slope from – 53 m (CD) to – 45 m (CD).

Eight different cross-sections (Alternatives 1 to 8) were tested, varying from complete rehabilitation to almost no repair. Three different solutions were considered for Alternative 8 (final design). The characteristics of the armour were exactly the same as in the Emergency Repair tests.

The wave conditions were similar to those employed earlier, this time with a peak period of 22 s and GF=0.9. A water level of + 4.0 m (CD) was used. Nine test runs were undertaken with different significant wave heights ranging from 5.0 m to 16.5 m. Test runs 1 to 6 corresponded to a combined sea and swell condition, whereas runs 7 to 9 corresponded to swell alone. Additionally, tests were performed for the final design for: i) T_p=16 s, GF=0.60 and GF=0.90; and ii) T_p=22 s, the design storm, GF=0.6 and two water levels (+ 1.0m (CD) and + 4.0 m (CD)).

The total overtopping volume was collected during 3 hrs (in the prototype) by a basin located directly behind the superstructure and, in two tests (for the design storm), the measurement was made continuously. The final design was the alternative that globally presented the smallest quantity of overtopping (maximum mean overtopping discharge of 7.2E-2 m^3/s/m for H_s~14.0 m).

The model tests carried out at LNEC (1983a) were undertaken between January and May 1983. Their main goals were to check the final solution and to verify the influence on stability and overtopping of the rounded and broken armour units. These tests were performed in the same irregular wave flume and at the same 1:85 geometrical scale as the Emergency Repair tests.

The foreshore was reproduced over 500 m and the foundation of the breakwater was located at approximately – 41 m (CD) at the superstructure section. Different breakwater cross-sections were tested, corresponding to adjustments in the back slope of the structure, with no influence on the overtopping measurements.

The wave conditions were similar to those used in the Emergency Repair tests, with peak periods of 16 s, 20 s and 24 s, significant wave heights of 5.6 m, 8.4 m, 11.2 m, 14.0 m and 16.8 m, and GF values between 0.60 and 0.70. Two different water levels were tested: + 1.0 m (CD) and + 4.0 m (CD). Each test comprised five runs, each of 12 hrs (in the prototype).

The mean overtopping discharges were only presented quantitatively for the test carried out with a water level of + 4.0 m (CD) and T_p=20 s. The results showed that the overtopping started for H_s=8.4 m, and for H_s=14 m the overtopping was very severe (5.1E-1 m^3/s/m).

2.2.3 Root Portion

To check and optimize the design of the cross-section for rehabilitation of the Dolosse Transition, LNEC (1989) undertook physical model studies between April and July 1988, which included overtopping measurements. These tests were carried out in the same irregular wave flume and at the same 1:85 geometrical scale as the tests for the Emergency Repair and the Outer Portion of the breakwater.

The foreshore was reproduced over a prototype distance of 1300 m from – 54 m (CD) to – 15 m (CD) at the toe of the breakwater. Two different alternatives for the armour units were tested: 60 ton and 40 ton Antifer units, both with a density of 2.6 ton/m^3.

Two wave conditions were tested for water levels of + 1.0 m (CD) and + 4.0 m (CD) and for deep water significant wave heights varying from 8 m to 14 m: a wind-sea wave spectrum with T_p=16 s and a combined wind-sea and swell spectrum with maximum T_p=22 s. Each test comprised 4 to 5 runs with increasing H_s values, each lasting 12 hrs (in the prototype).

The overtopping volume was collected during 80 minutes in a 30 cm wide by 120 cm long tray. For the last 20 minutes of each run, a video continuously recorded the overtopping conditions. The results showed that the cross-section with 40 ton Antifer units could replace the cross-section with 60 ton Antifer units with the same degree of overtopping and with the maximum mean overtopping discharge reaching around 3.9E-3 m^3/s/m and 6.1E-2 m^3/s/m for the lowest and the highest water levels, respectively. The combined wind-sea and swell conditions produced larger mean overtopping rates than the wind-sea conditions alone and overtopping started for lower values of H_s.

3. FINAL REHABILITATION

3.1 Overview

The reactivation of Berth 1 implies the prior rehabilitation of the breakwater sections south of Berth 2, which will improve its shelter and operating conditions. The satisfactory behaviour of the existing solutions, the experience gained with the previous rehabilitation works carried out between 1979 and 1992 and the enormous volume of materials to be removed (from the existing breakwater and from the remains of the original breakwater) and then placed, led to the consideration of similar solutions and materials to those used in the existing breakwater. It is estimated that more than 400 000 m^3 of material will be reused, comprising damaged superstructure demolition products, core and armour with graded rock of up to 120 kN, and high density Antifer blocks of 900 kN and 1050 kN (35 m^3 each).

Three proposed solutions for the cross-section of the breakwater between Berths 2 and 1 were selected for two-dimensional physical model tests (Figure 8):

- Solution 1 (Figure 8a) – corresponds to the existing cross-section between Berths 2 and 1 and it is similar to the cross-section proposed to be used south of Berth 1; it is characterized by a crest berm at + 13.2 m (CD), not protected against overtopping, a two-layer irregular placement (0.09 units/m^2) of 900 kN Antifer blocks below CD and a one-layer regular placement (0.125/2 units/m^2) above;

- Solution 2 (Figure 8b) – to be used between Berths 2 and 1, differs from Solution 1 mainly in the crest area (in shadow in the figure); it has a concrete superstructure incorporating a tunnel, with platforms at + 13.2 m (CD) (not protected against overtopping) and at + 8.0 m (CD) (protected against overtopping);
- Solution 3 (Figure 8c) – to be used only at Berth 1 or also between Berths 2 and 1, is similar to the existing cross-section between Berths 3 and 2, and it differs from Solution 1 mainly in the crest area (in shadow in the figure); it has a recurved concrete superstructure with its crest at + 19.0 m (CD), platforms at + 13.0 m (CD) and + 8.0 m (CD), and a 20 m crest berm at + 18.0 m (CD); it has a two-layer regular placement (0.125 units/m^2) of Antifer blocks above CD.

a)

b)

c)

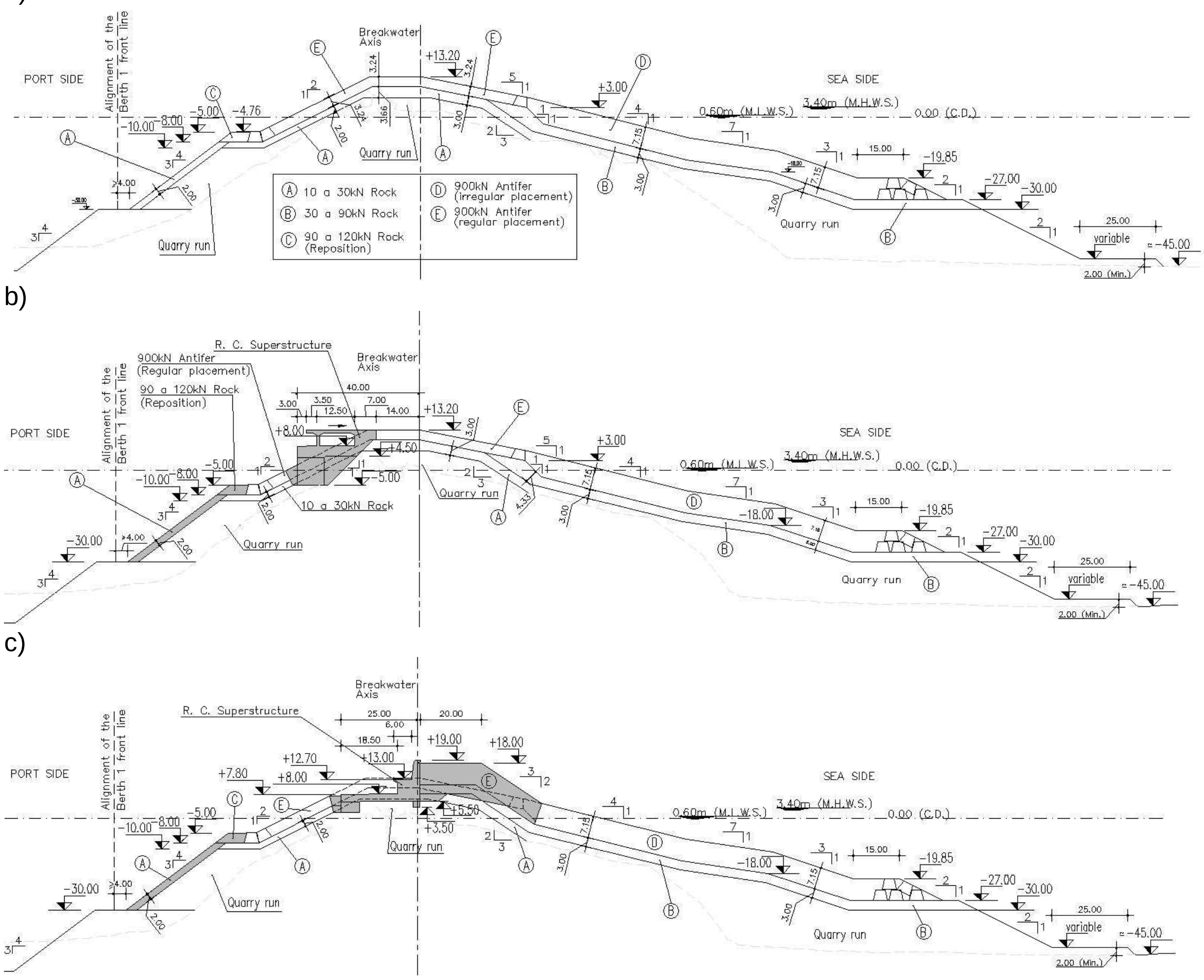

Figure 8: Proposed cross-sections for the final breakwater rehabilitation: a) Solution 1; b) Solution 2; c) Solution 3

The proposed solutions were designed using extreme wave conditions based on 18 years of wave data from the Sines directional buoy. Conditions for a return period of 100 years (within a confidence margin of 99%) were defined by a significant wave height of 12 m (see Table 1) with peak periods of up to 20 s. Note that the wave regime used for some of the rehabilitation works, based on a wave hindcast of 25 years (DHL, 1981a), when insufficient wave buoy data were available, led to a deep water significant wave height of 14 m for the same return period (Table 1).

3.2 Physical Model Tests

To check the suitability of the three proposed solutions for the cross-sections of the breakwater, two-dimensional physical model tests of stability and overtopping were performed in one of LNEC's wave flumes, which is approximately 73 m long, 3 m wide and has an operating water depth of 2 m. The flume is equipped with a piston-type wave-maker and an active wave absorption system, AWASYS (Troch, 2005), which allows the absorption of reflected waves. Wall effects were minimized by placing the model in the middle of the flume, separated from the flume walls by two side wave guides.

The models were built and operated according to Froude's similarity law: the geometrical scale was 1:60. Two prototype still-water-levels were tested: + 0.0 m (CD) and + 4.0 m (CD), giving depths, d_s, at the toes of the model structures of 0.75 m and 0.82 m, respectively (Figure 9). The sea bed in front of the model breakwaters was represented by different slopes down to a level of - 55.0 m (CD), giving operating water depths, h, in front of the wave-maker of 0.92 m and 0.98 m, for the two water levels tested.

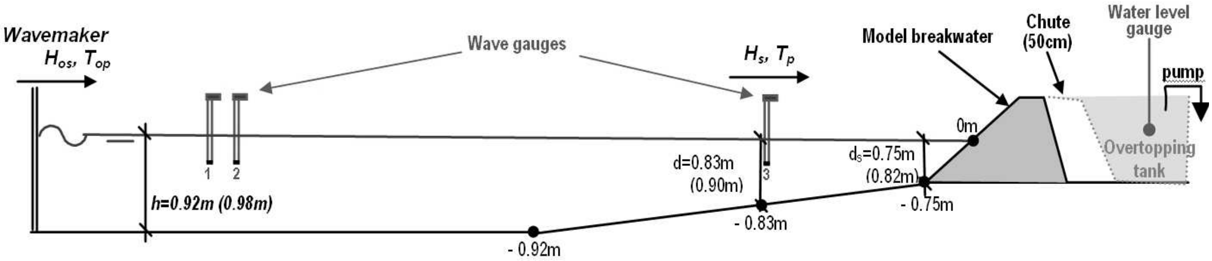

Figure 9: Sketch of wave fume, model breakwater and location of measuring equipment

To measure the free-surface elevation, the flume was equipped with three resistive-type wave gauges (Figure 9). A fixed array of two wave gauges (gauges 1 and 2), required for the dynamic wave absorption system, was located in front of the wave-maker and the third gauge (gauge 3) was located in front of the structure, where the prototype sea bed was at - 50.0 m (CD) (model depths, d, of 0.83 m and 0.90 m for the tested water levels).

To determine the mean overtopping discharges per metre length of structure, q_{PM}, a tank was located at the back of each structure to collect the overtopping water (Figure 9). The water was directed to the tank by means of a chute, 50 cm wide (Figure 10). A pump and a gauge were deployed in the overtopping tank and connected to a computer that monitored and recorded the water level variation within a test run. Once a preset maximum water level was reached in the tank, the pump was activated for a fixed period. The pumped volume of water was derived from a pump calibration curve. The measurement of the water level variation inside the tank, together with the pump calibration curve, allowed the determination of the mean overtopping rates. A computer collected and stored the data in digital format at a frequency of 40 Hz.

Figure 10: Overtopping observed during physical model tests for Solutions 1 (left), 2 (middle) and 3 (right)

Irregular waves conforming to a mean JONSWAP spectrum were employed with significant wave heights, H_{os}, and peak periods, T_{op}, in front of the wave-maker ranging from 4 m to 14 m and 10 s to 20 s in prototype values. The test programme was defined by the client, CONSULMAR, and specified a sequence of runs, each with predefined values of H_{os} and T_{op} for each water level. The test duration ranged from 23 to 46 minutes for each run (approximately 1000 waves).

A detailed description of the tests can be found in LNEC (2008). This paper concentrates on the mean overtopping discharges per metre length of structure, q_{PM}, of 24 tests carried out for Solutions 1, 2 and 3 (Table 4). In Table 4, the values for the wave conditions, T_p and H_s, refer to measured values at gauge 3, located at depth d, in front of the structure, presented at prototype scale. Given the deep water conditions in front of the structure (approximately - 45.0 m (CD) at the toe), the differences in wave conditions between gauges 1 and 3 were small. The precision of the measurements of the mean overtopping discharges was ± 5E-6 m^3/s/m.

Test	d (m)	T_p (s)	H_s (m)	q_{PM} (m^3/s/m) Solution 1	Solution 2	Solution 3
1	50.0	10	4	0.0E+00	0.0E+00	0.0E+00
2		**10**	**6**	**0.0E+00**	**0.0E+00**	**0.0E+00**
3		12	8	0.0E+00	0.0E+00	0.0E+00
4		**14**	**9**	**0.0E+00**	**0.0E+00**	**0.0E+00**
5		16	10	4.1E-03	5.1E-03	2.8E-05
6		18	11	1.6E-02	1.9E-02	1.1E-03
7		**18**	**11.5**	**2.2E-02**	**2.5E-02**	**1.8E-03**
8		12	12	1.8E-03	2.1E-03	1.9E-04
9		**16**	**12**	**1.4E-02**	**1.5E-02**	**2.8E-04**
10		20	12	3.3E-02	6.0E-02	4.8E-03
11		**20**	**13**	**1.4E-01**	**1.6E-01**	**1.4E-02**
12		**20**	**14**	**1.7E-01**	**2.0E-01**	**3.5E-02**
13	54.0	10	4	0.0E+00	0.0E+00	0.0E+00
14		**10**	**6**	**0.0E+00**	**0.0E+00**	**0.0E+00**
15		12	8	0.0E+00	0.0E+00	0.0E+00
16		**14**	**9**	**5.1E-03**	**6.4E-03**	**6.1E-04**
17		16	10	3.5E-02	4.3E-02	2.7E-03
18		18	11	1.2E-01	1.5E-01	1.4E-02
19		**18**	**11.5**	**2.1E-01**	**2.5E-01**	**4.4E-02**
20		12	12	8.4E-03	9.6E-03	5.8E-04
21		**16**	**12**	**1.4E-01**	**1.6E-01**	**3.3E-02**
22		20	12	3.2E-01	3.5E-01	7.5E-02
23		**20**	**13**	**6.1E-01**	**6.5E-01**	**1.4E-01**
24		**20**	**14**	**9.0E-01**	**9.3E-01**	**2.5E-01**

Table 4: Mean overtopping discharges per metre length of structure, q_{PM}, obtained for Solutions 1 to 3 (all values scaled up to prototype)

The table shows that, unsurprisingly, given its higher crest level, Solution 3 was the least overtopped structure and that the greatest quantity of overtopping water was measured for Solution 2.

Although the local wind conditions were not reproduced during the physical model tests, the overtopping water adhered to the structure's crest, reaching little distance behind the breakwater. In particular, the Berth 1 caissons were hit only by the biggest waves, on their lower sections, close to the water level.

The measured discharges were compared with the critical values recommended in the technical literature (U.S. Army Corps of Engineers, 2006; Pullen *et al.*, 2007) and some critical values were exceeded. However: i) the rubble mound breakwater was stable in all three proposed solutions for all test conditions, with only a few blocks being moved during the more energetic conditions; and ii) the proposed works are similar to the existing (and much-studied) structures. Consequently, the final decision on which solution to adopt was based mainly on comparisons with the results from previous physical model studies, on the local day-to-day operating experience and on economic and risk considerations. Thus, Solution 3 was selected for detailed design.

A risk assessment was undertaken for the construction, in-service life and maintenance/repair phases. The risk assessment for the first and last phases quickly led to a conceptual option in which construction took place mainly by land. It aimed to improve accessibility, safety and the operational time of the contractor's equipment and personnel, up to certain levels of exposure to wave action, thus enhancing the quality of the construction (and the effectiveness of eventual future repair works) within the intended duration for the works. Solution 3 best complied with the above requirements. Regarding the design life, the importance of the intended investment and the expected return requires that Berth 1 is accessible and operational with low down-times during extreme overtopping events. All studies have shown that the difference between the estimated costs of implementing Solution 2 instead of Solution 3 is minor on the scale of the global investment, unlike the difference in the quantities of overtopping water, which is significant. The final factor that led to the choice of Solution 3 was the use of a similar solution to that existing at the cross-section between Berths 3 and 2, whose performance and operating conditions are well known by all the breakwater's users.

3.3 AMAZON Numerical Model

The physical model overtopping data collected at LNEC for Solutions 1 and 2 have been used to check the applicability of the non-linear shallow water (NLSW) numerical model, AMAZON, to porous structures, since the development of the porous flow model was carried out only recently. A detailed description of AMAZON can be found in Hu (2000) and in Reis *et al.* (2008, 2009a). Reis *et al*. (2009b) describe in detail the application of AMAZON to estimate mean overtopping discharges for Solutions 1 and 2, for the twelve test conditions shown in bold in Table 4. The numerical model AMAZON was not applied to Solution 3 since it is not valid for recurved walls, given the fact that it is a depth averaged model.

This paper presents the AMAZON results (Table 5), which have been obtained at model scale, using the Darcy equation to govern the water exchange between the porous cells of the model structures, for IP=0.125 m/s and n=0.4, where IP is the maximum velocity that the water can reach during the exchange of water between the free-flow (surface) and porous layers of the model and n is the porosity of the porous layer. The table shows that the mean overtopping discharges per metre length of structure obtained with AMAZON, q_{AM}, present a similar trend to that shown in the physical model, q_{PM}; that is, for each of the 12 tests, q_{AM} for Solution 2 is always greater than that for Solution 1, except when they are both zero.

Test	d (m)	T_p (s)	H_s (m)	q_{PM} (m^3/s/m) Solution 1	q_{PM} (m^3/s/m) Solution 2	q_{AM} (m^3/s/m) Solution 1	q_{AM} (m^3/s/m) Solution 2
2	50.0	10	6	0.0E+00	0.0E+00	0.0E+00	0.0E+00
4		14	9	0.0E+00	0.0E+00	0.0E+00	0.0E+00
7		18	11.5	2.2E-02	2.5E-02	2.5E-02	3.3E-02
9		16	12	1.4E-02	1.5E-02	1.4E-02	2.4E-02
11		20	13	1.4E-01	1.6E-01	1.6E-01	2.1E-01
12		20	14	1.7E-01	2.0E-01	2.1E-01	3.5E-01
14	54.0	10	6	0.0E+00	0.0E+00	0.0E+00	0.0E+00
16		14	9	5.1E-03	6.4E-03	2.0E-03	1.1E-02
19		18	11.5	2.1E-01	2.5E-01	2.3E-01	3.2E-01
21		16	12	1.4E-01	1.6E-01	1.4E-01	2.6E-01
23		20	13	6.1E-01	6.5E-01	7.1E-01	7.6E-01
24		20	14	9.0E-01	9.3E-01	9.7E-01	1.0E-00

Table 5: Mean overtopping discharges per metre length of structure obtained for Solutions 1 and 2 with the physical, q_{PM}, and the numerical, q_{AM}, models (all values scaled up to prototype)

Figure 11 shows the AMAZON results versus the physical model results, all scaled up to prototype values. Most data points are within the range $0.5 \le q_{AM}/q_{PM} \le 2$. However, $q_{AM}/q_{PM} < 0.5$ for Test 16, Solution 1. Figure 11 shows that there is a good agreement between the AMAZON results (obtained for IP=0.125 m/s and n=0.4) and the physical model data, although AMAZON tends to slightly over-predict the discharges, especially for Solution 2. These results are rather promising, given the different approximations made for modelling the porous breakwater (such as using the Darcy equation rather than the Forchheimer equation to govern the internal flow and employing only one porous layer to represent a multi-layered structure) and AMAZON's inherent restrictions (mainly relating to the shallow water assumptions and the fact that waves entering the computational domain will already have broken or will begin to break).

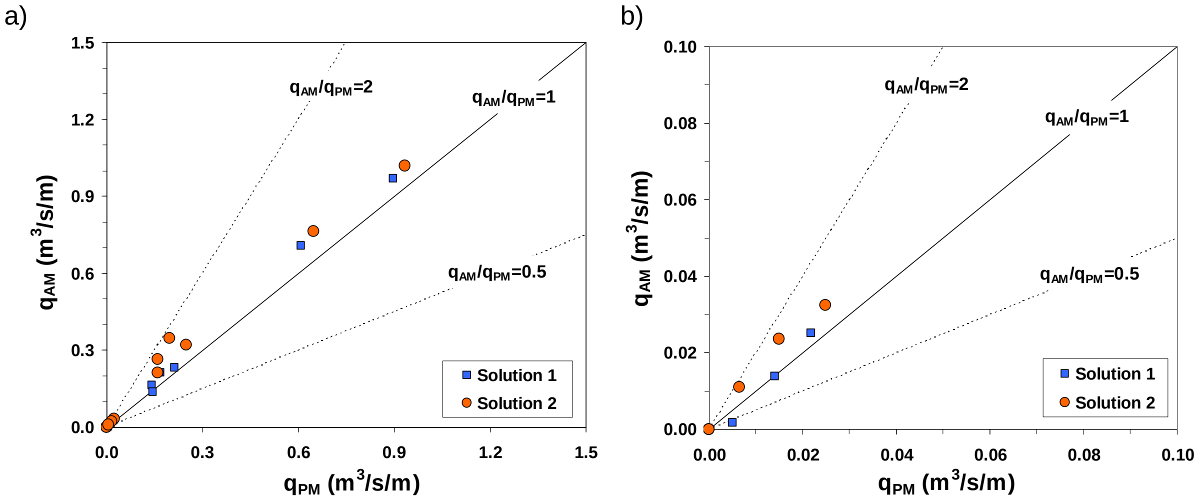

Figure 11: Comparison between the AMAZON, q_{AM}, and the physical model, q_{PM}, mean overtopping discharges obtained for Solutions 1 and 2 (all values scaled up to prototype) using the Darcy equation with IP=0.125 m/s and n=0.4: a) all results; b) results for discharges less than 0.1 m^3/s/m

3.4 Other Methodologies

The physical model overtopping data collected at LNEC for Solutions 1 to 3 for the twelve test conditions shown in bold in Table 4 have also been used to check the applicability of the methodologies/tools recommended by Pullen *et al.* (2007) to estimate mean overtopping discharges for a structure like the Sines West Breakwater. In order to apply these methodologies/tools, each of the three solutions has to be represented by the structural types/configurations recommended by Coeveld *et al*. (2005) and Pullen *et al.* (2007). However, since these solutions do not strictly fall in any of the structural types/configurations available, different representations have been tested as follows (Table 6):

- Solutions 1 & 2:
    - Armoured slope with a berm at – 19.85 m (CD);
    - Armoured slope with a toe at – 19.85 m (CD) (no berm);
    - Armoured slope with a zero-width berm at - 7.00 m (CD) and a toe at – 19.85 m (CD);
- Solution 3:
    - Bermed armour (berm at – 19.85 m (CD)) with wave wall;
    - Armoured composite slope with wall and with a toe at – 19.85 m (CD) (no berm);
    - Armoured composite slope with wall with a zero-width berm at - 7.00 m (CD) and a toe at - 19.85 m (CD).

The literature (e.g. Coeveld *et al*., 2005; Pullen *et al.*, 2007) suggest values for the roughness/permeability factor, γ_f, of 0.47 or 0.5 for Antifers, but they have been based on physical model tests carried out with 2 layers of Antifer blocks placed randomly on a 1:1.5 slope (Pearson *et al.*, 2004). At Sines West Breakwater, the Antifers have a regular placement at the upper part of the slope and at the crest berm (Figure 12), which increases run-up and overtopping considerably. A sensitivity analysis was performed on γ_f, considering values between 0.5 and 0.8 for the different slope sections of the breakwater.

Figure 12: Sines West Breakwater: detail of Antifer placement at the upper part of the armour slope and at the crest berm

For each of the above representations, the applied overtopping methods are described in Table 6.

Solution	Label	Structure Type	Recommended Method
1	1A	Armoured Slope with Berm	Neural Network
	1B	Armoured Slope with Toe Detail (no berm)	Neural Network
	1C	Armoured Slope with Toe Detail (zero berm width)	Neural Network
2	2A	Armoured Slope with Berm	Neural Network
	2B	Armoured Slope with Toe Detail (no berm)	Neural Network
	2C	Armoured Slope with Toe Detail (zero berm width)	Neural Network
3	3A	Bermed Armour with Wave Wall	Neural Network
	3B	Armoured Composite Slope with Wall (no berm)	Empirical Methods & Neural Network
	3C	Armoured Composite Slope with Wall (zero berm width)	Empirical Methods & Neural Network

Table 6: Adopted representations of Solutions 1 to 3 and corresponding recommended methods for overtopping estimation according to Pullen *et al*. (2007)

The mean overtopping discharges obtained with the Neural Network (NN), q_{NN}, for Solutions 1 to 3 using the representations shown in Table 6 showed that, for each solution, the differences in the NN predictions for the three representations were much less important than the differences due to the use of different values of γ_f, showing that, as expected, γ_f has a major influence on the results. Consequently, the NN predictions for the twelve test conditions were carried out using representation B for Solutions 1 to 3 and the method presented by Pullen *et al.* (2007) to combine the effect of slope sections with different roughness/permeability to provide a simple estimate of the combined roughness/permeability factor, which varies with the test conditions considered. The combined γ_f values obtained varied between about 0.51 and 0.70. The mean overtopping discharges obtained with the Neural Network for Solutions 1 to 3 using representation B are presented in Figure 13, both for the mean values of q_{NN} and for the upper and the lower limits of the 95% confidence intervals.

The results show that the NN tends to under-predict all the results, especially for Solution 3. For the three solutions, the physical model results are within the confidence intervals obtained with the NN, with these intervals being much wider for Solutions 1 and 2 than for Solution 3, regardless of the wave conditions and water levels considered. The confidence intervals are very useful in letting the NN user know what confidence to have in the values of q_{NN}. Note that for $10^{-6}<q_{NN}<10^{-5}$ m^3/s/m, the NN predictions are less reliable: they are only indicative (Coeveld *et al*., 2005).

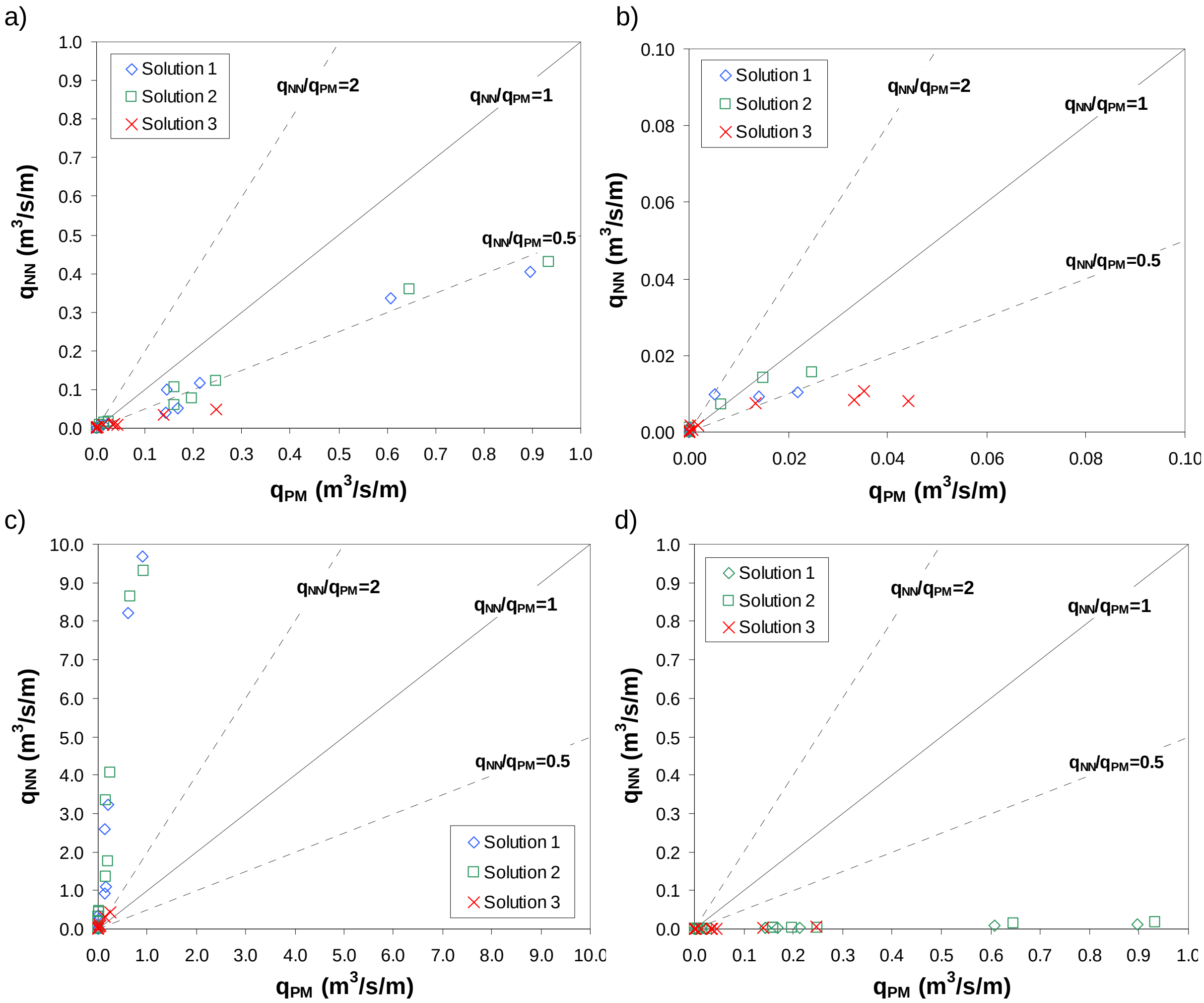

Figure 13: Comparison between the Neural Network, q_{NN}, and the physical model, q_{PM}, mean overtopping discharges obtained for Solutions 1 to 3 using representation B (all values scaled up to prototype): a) mean values of q_{NN}, all results; b) mean values of q_{NN}, results for discharges less than 0.1 m^3/s/m; c) upper limits of the 95% confidence intervals; d) lower limits of the 95% confidence intervals

The mean overtopping predictions for the twelve test conditions using the Empirical Methods were carried out, as for the Neural Network, with γ_f calculated using the method presented by Pullen *et al.* (2007) to combine the effect of slope sections with different roughness/permeability. The results again show that γ_f has a major influence on the predicted discharges. The mean overtopping discharges obtained with the Empirical Methods for Solution 3 are presented in Figure 14. Note that as far as the Empirical Methods are concerned, representations 3B and 3C are the same.

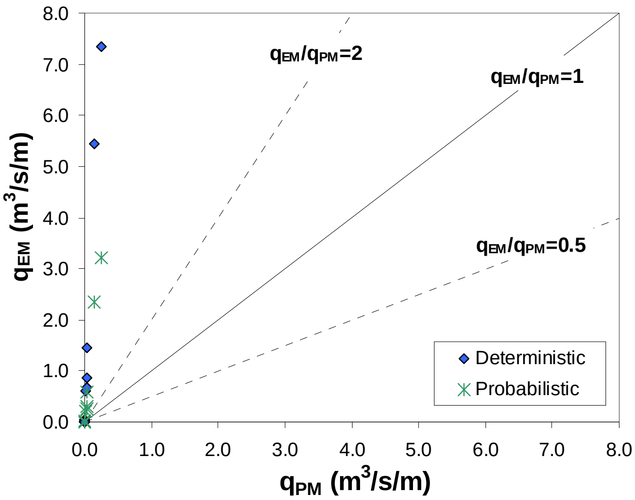

Figure 14: Comparison between the Empirical Method, q_{EM}, and the physical model, q_{PM}, mean overtopping discharges obtained for Solution 3 (all values scaled up to prototype)

The results show that the Empirical Methods over-predict the physical model results both for deterministic and probabilistic calculations, despite the fact that, as expected, this over-prediction is more noticeable for deterministic calculations. As mentioned in Pullen *et al.* (2007), for comparison with measurements undertaken or to predict a measurement in a laboratory (or in real situations), probabilistic calculations should be adopted, whereas for deterministic design or safety assessment, deterministic results should be used.

4. CONCLUDING REMARKS

The paper presents an historical perspective on the design, construction, failure and rehabilitation of the West Breakwater of the Portuguese harbour of Sines, as well as on the overtopping physical model studies performed to check the effectiveness of the different proposed solutions for its rehabilitation.

Additionally, the paper describes the final rehabilitation to be carried out on the breakwater, which has the primary objective of reactivating Berth 1 but also to generally improve the shelter and operating conditions within the port. It presents the three proposed solutions for the rehabilitation: Solution 1 is basically a rubble mound breakwater, not protected at the crest against wave overtopping, with armour composed of Antifer cubes of 900 kN (trunk) and 1050 kN (head); Solutions 2 and 3 differ from Solution 1 mainly in the crest area, with Solution 2 having a concrete superstructure and Solution 3 having a higher crest level with a recurved concrete superstructure. The paper also presents a description of the two-dimensional physical model tests of stability and wave overtopping carried out in 2008 at the National Civil Engineering Laboratory, Portugal, to check the effectiveness of the three solutions. Solution 3 significantly reduced the mean overtopping discharges measured in the tests for Solutions 1 and 2 (which were similar). All three solutions proved to be stable under design conditions. Based on comparisons with results from previous physical model studies, on the local day-to-day operating experience and on economic and risk considerations, Solution 3 was selected for detailed design. It will protect and support the road access and pipelines for Berth 1.

The paper also illustrates the application of the new version of the NLSW numerical model, AMAZON, and of the methodologies/tools recommended in the EurOtop overtopping manual to study the mean wave overtopping discharge over the breakwater. AMAZON was applied to Solutions 1 and 2; it was not applied to Solution 3 since it is not valid for recurved walls, given the fact that it is a depth averaged model. There was good agreement between the physical model data and the AMAZON results calculated using the Darcy equation to govern the water exchange between the porous cells, although AMAZON tended to slightly over-predict the discharges, especially for Solution 2. The CLASH Neural Network Generic Overtopping Prediction Tool was applied to the three solutions, whereas the Empirical Methods were applicable to Solution 3 only. The Neural Network (NN) mean overtopping discharges tended to under-predict the physical model results, especially for Solution 3. For the three solutions, the physical model results were within the confidence intervals obtained with the NN, with these intervals being much wider for Solutions 1 and 2 than for Solution 3, regardless of the wave conditions and water levels considered. The differences in the NN predictions for the various representations of each solution were much less important than the differences found by varying the values of the roughness/permeability factor, showing that, as expected, this factor has a major influence on the results. The Empirical Methods considerably over-predicted the mean overtopping discharges.

5. ACKNOWLEDGEMENTS

The authors wish to thank the Administração do Porto de Sines, S.A. (APS), Portugal, for authorization to publish some of the results of the physical model tests undertaken at LNEC for the solutions proposed for the rehabilitation of the West Breakwater of Sines Harbour. Thanks are also due to Dr Van der Meer for his suggestions in using the Clash Neural Network and to Eur Ing Terry Hedges for his suggestions and help in improving the English.

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