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# Rehabilitation of Sines west breakwater: wave overtopping study

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**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. Studies are now being conducted for Sines Port Authority for the final rehabilitation of the breakwater, with the primary objective of reactivating berth 1 but also to generally improve the shelter and operating conditions within the port. The paper presents the three proposed solutions for the west breakwater and their two-dimensional physical model tests of stability and overtopping carried out in 2008 at Laboratório Nacional de Engenharia Civil (LNEC), Portugal. The paper also illustrates the application of a new version of the non-linear shallow water numerical model, Amazon, and of the methodologies recommended in the EurOtop overtopping manual to study the mean overtopping discharge over the breakwater. The Clash neural network was the only tool applicable to the three proposed solutions, although it tended to underpredict the physical model discharges, mainly for the selected solution 3. There was good agreement between the physical model data and the Amazon results for solutions 1 and 2, although Amazon tended to slightly overpredict the discharges, especially for solution 2. The empirical methods overpredicted these discharges to a great extent, warning of the fact that direct application of these methods is limited to particular structural configurations and wave conditions.**

## Notation

| | |
|---|---|
| $d$ | still-water depth in front of the structure |
| GF | groupiness factor |
| $H_s$ | significant wave height |
| $H_{sd}$ | design significant wave height |
| $q_{EM}$ | mean overtopping discharge of water over unit length of structure obtained with empirical methods |
| $q_{NN}$ | mean overtopping discharge of water over unit length of structure obtained with the Clash neural network |
| $q_{PM}$ | mean overtopping discharge of water over unit length of structure obtained in the physical model |
| $T_p$ | wave period corresponding to peak spectral density |
| $\gamma_f$ | reduction factor to account for influence of roughness and permeability of a slope on wave overtopping |

## 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 bulk liquids, petrochemical products, multipurpose and roll-on roll-off (ro-ro), and liquified natural gas (LNG)).

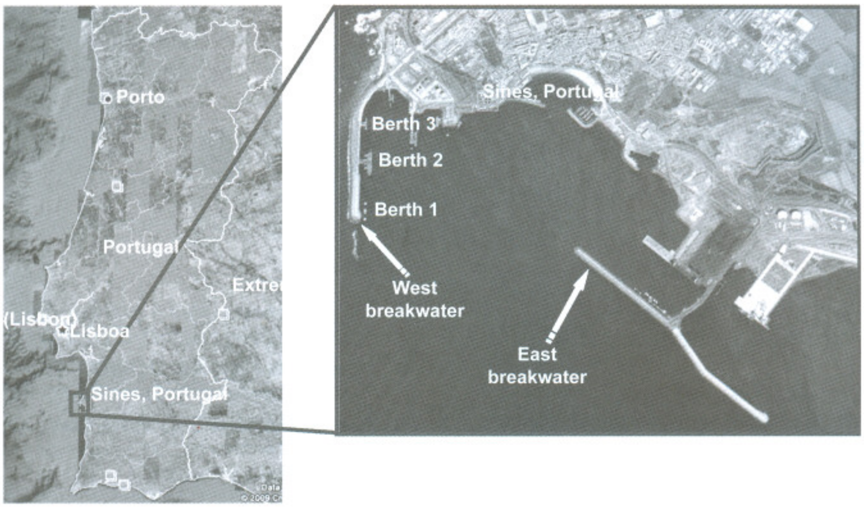


Figure 1. Port of Sines: location and current layout (© 2011 Tele Atlas; Data SIO, NOAA, U.S. Navy, NGA; Image © 2011 GeoEye; Image © 2011 DigitalGlobe; © 2011 MapLink/Tele Atlas; © 2011 Europa Technologies)

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 (2-D) 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.

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 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, 2-D physical model tests of stability and overtopping were performed in 2008 at Laboratório Nacional de Engenharia Civil (LNEC) (LNEC, 2008).

The following section presents 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. The three proposed solutions for the final rehabilitation of the breakwater and their physical model testing is then described. The numerical overtopping modelling of the solutions is presented, using the non-linear shallow water (NLSW) model Amazon (Hu, 2000; Reis *et al.*, 2008, 2009a, 2009b) and using the methodologies recommended in Pullen *et al.* (2007) for a structure like the Sines west breakwater. The overtopping results from the numerical modelling are shown and discussed and the final section offers some concluding comments.

## 2. Historical perspective

### 2.1 Design, construction, failure and rehabilitation

The design of the west breakwater was developed in three main stages (Mettam, 1976): the tender design (Figure 2(a)), the contractor's alternative design (Figure 2(b)) and the final design (Figure 2(c)).

The differences between the cross-sections proposed at the different stages were essentially twofold: (*a*) the tender design had to be based on tests carried out in a wave flume that could only produce regular waves, whereas the final design was based upon flume tests with irregular waves, which increased wave overtopping and the forces on the wave wall; and (*b*) the contractor proposed to widen the breakwater crest to simplify the structural arrangements for supporting the oil pipelines and to give more space for constructional equipment.

For the final design, the breakwater was 2 km long in depths up to 50 m, protected by two armour layers of unreinforced 42 t dolos units on a 2(V) : 3(H) slope, with a crest berm at +16·0 m chart datum (CD) and a recurved concrete superstructure with its crest at +19·0 m CD (Figure 2(c)). It was a

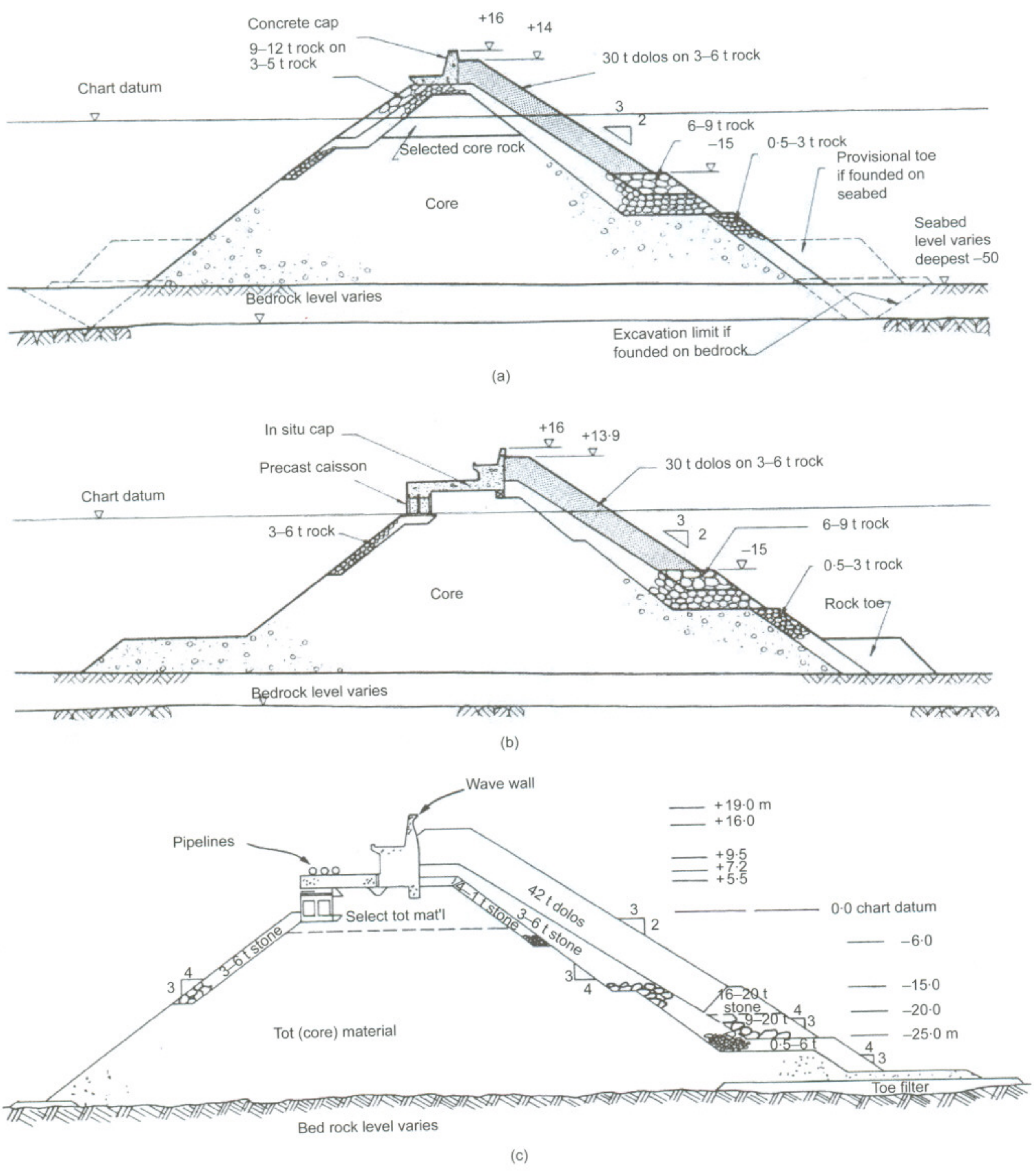


Figure 2. Breakwater cross-sections: (a) tender design; (b) contractor's alternative; (c) final design (after Mettam, 1976; Port Sines Investigating Panel, 1982) (elevations in m above chart datum)

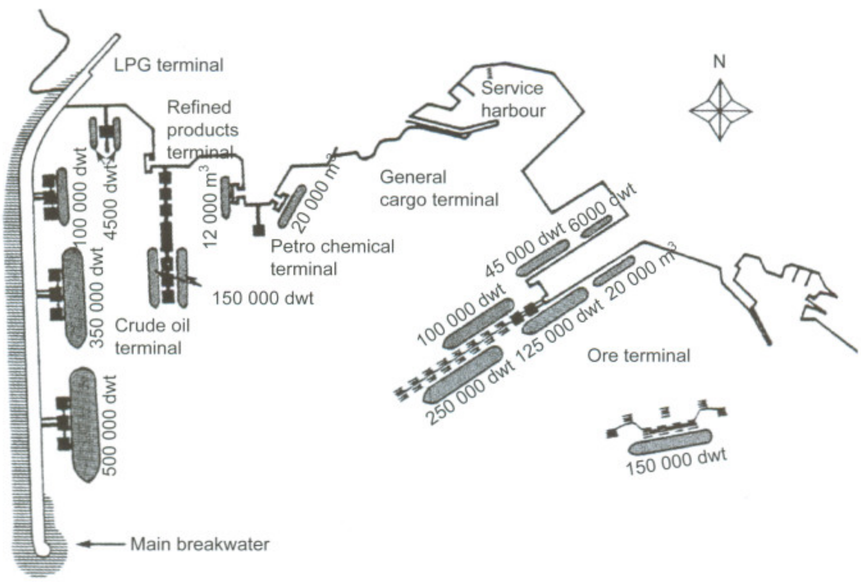


Figure 3. Port of Sines: original layout (after Port Sines Investigating Panel, 1982)

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).

The breakwater was 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. This design significant wave height was based on 1 year of Waverider Buoy measurements at Sines plus 7 years of records at Figueira da Foz, located about 250 km north of Sines (Port Sines Investigating Panel, 1982).

The breakwater was constructed between 1973 and 1978. Construction was nearly complete when, on 26 February 1978, critical damage occurred during a storm with waves thought by most to be below the 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).

Subsequent to the failure, studies were undertaken by several hydraulic laboratories, such as LNEC, Delft Hydraulics Laboratory (DHL), the National Research Council (NRC) of Canada and the Laboratoire Central d'Hydaulique de France (LCHF). 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 the failure. The following list provides examples, noted in the literature, of possible factors that may

| Return period: years | Original project and 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. Design wave heights, $H_{sd}$: dates refer to construction period (Abecassis and Pita, 1993; Consulmar, 2006, 2008; Dinis and Toppler, 1993; Ligteringen *et al.*, 1993a, 1993b; Port Sines Investigating Panel, 1982)

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

have contributed to the failure (e.g. Port Sines Investigating Panel, 1982; Zwamborn, 1979).

(*a*) Shortcomings in the selection of design waves (conception deficiency).
(*b*) Communication problems between owner–designer–contractor (deficiency in project management).
(*c*) Differences between the constructed breakwater and the design specifications (deficiency in construction and in construction supervision).
(*d*) Low structural strength of dolosse (structural deficiency).
(*e*) Physical removal by wave action of the 16 to 20 t toe stones (structural deficiency).
(*f*) Storms hit the breakwater when construction had not been fully completed (inevitable).

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). The emergency repair and the subsequent rehabilitation works were essentially constructed above the remains of the original breakwater, comprising damaged superstructure, core and armour. The wave regime used for the emergency repair was the same as that for the original project, namely 100-year return period significant wave height of 11 m (Table 1). The wave regime used for the subsequent rehabilitation works, based on a wave hindcast of 25 years (DHL, 1981a), when insufficient wave buoy data were available, led to a significant wave height of 14 m for the same return period (Table 1).

The subsequent rehabilitation works included the rehabilitation of the root portion of the breakwater (the length of breakwater nearest to the shore, also known as the dolosse portion), 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 (Figure 5). The layout of the breakwater after the last rehabilitation works is shown in Figure 6.

At present, the breakwater has a reduced crest length of about 1·6 km and it is protected by Antifer cubes of 400 kN at the root portion, of 900 kN along the trunk and of 900 and

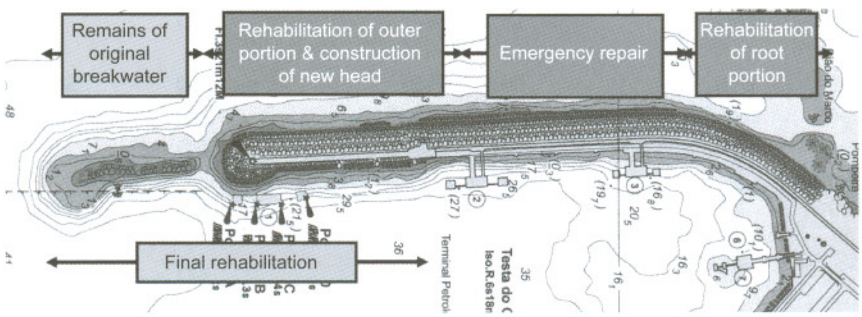


Figure 5. Port of Sines rehabilitation works

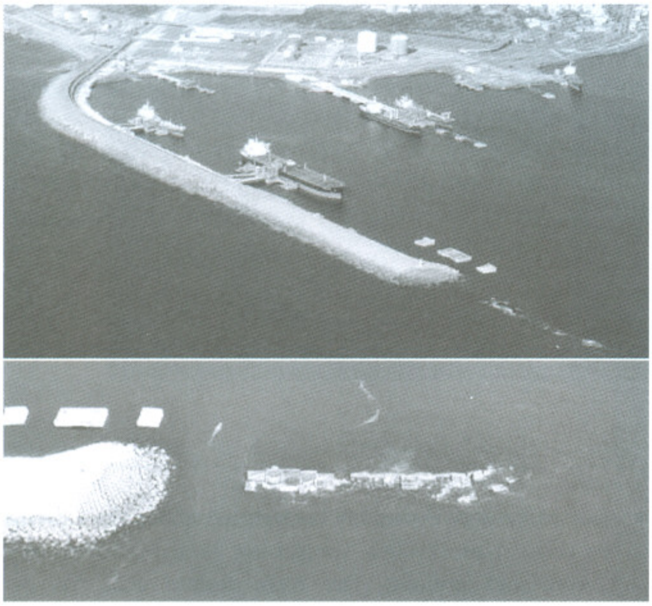

Figure 6. Sines west breakwater: layout after last rehabilitation works (including remains of the original breakwater)

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 remains of the original breakwater 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 berth 1, for reception, loading and unloading of oil tankers, 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 summarises most of these studies: the bold text indicates those studies which have included overtopping measurements.

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

| Breakwater section | Laboratory | Reference used | 2-D/3-D | Stability/overtopping |
|---|---|---|---|---|
| Root portion | DHL | DHL (1987) | 3-D | Stability |
| | **LNEC** | **LNEC (1989)** | **2-D** | **Stability and overtopping** |
| Emergency repair | LNEC | LNEC (1979) | 2-D | Stability |
| | NRC | LNEC (1980) | 2-D and 3-D | Stability |
| | LCHF | DHL (1987) | 2-D | Stability |
| | **LNEC** | **LNEC (1981)** | **2-D** | **Stability and overtopping** |
| | **DHL** | **DHL (1981b)** | **2-D** | **Stability and overtopping** |
| | DHL | DHL (1981c) | 2-D | Stability |
| | **LNEC** | **LNEC (1983b)** | **2-D** | **Stability and overtopping** |
| Transition between emergency repair and outer portion | DHL | DHL (1987) | 3-D | Stability |
| Outer portion | **DHL** | **DHL (1982)** | **2-D** | **Stability and overtopping** |
| | **LNEC** | **LNEC (1983a)** | **2-D** | **Stability and overtopping** |
| | DHL | DHL (1986) | 2-D | Stability |
| Transition between outer portion and new head | DHL | DHL (1986) | 2-D | Stability |
| New head | DHL | DHL (1987) | 3-D | Stability |

Table 2. Physical model tests of stability and overtopping carried out for Sines west breakwater before last rehabilitation works

out by LNEC, NRC and LCHF in the course of these works (DHL, 1987; LNEC, 1979, 1980) but 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 (DHL, 1981b, 1981c; LNEC, 1981, 1983b). 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 concrete with 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 ($1{\cdot}2H_{sd}$). A water level of +4·0 m CD was employed, corresponding to a high water level plus storm surge. Each test comprised five runs, each of 12 h (in the prototype), and was performed with increasing values of the significant wave height from 5·6 m (40% of $H_{sd}$) to 16·8 m.

For the standard solution, four tests were performed: (*a*) test with a groupiness factor (GF) of 0·39; (*b*) test with GF = 0·65; (*c*) test with GF = 0·69; and (*d*) 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 min (2·94 h 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 GF value strongly affected the number of overtopping waves. Wind increased overtopping considerably as well as increasing GF. 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 0·067 $m^3$/s per 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, 20 and 24 s and with GF varying from 0·60 to 0·70. The significant wave heights of 5·6, 8·4, 11·2, 14 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 h (in the prototype), with the five different significant wave heights. If the damage was small after the last run, then the run was extended for another 12 h.

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 a significant wave height of 11·2 m, and for 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 (0·18 $m^3$/s per m for a significant wave height of 14·0 m), with the overtopping tray being insufficient to measure the volume of overtopping for the highest significant wave height.

Table 3 summarises the main differences between the tests undertaken at LNEC and at DHL. The test for the standard solution and $T_p$ = 20 s was carried out for the same wave and water level conditions in both laboratories. 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 overtopping discharge values measured by LNEC and DHL for a significant wave height of 14·0 m, with the LNEC measurement being almost three times that of DHL.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Test | LNEC | | | | DHL | | | |
| Scale | 1 : 85 | | | | 1 : 78 | | | |
| Overtopping | Over 3 h | | | | Over 3 h | | | |
| No. 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 and standard with damaged units | +4·0 m CD | 0·6–0·7 | 20 s, 24 s | Alternatives I-a, I-b, I-d and 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

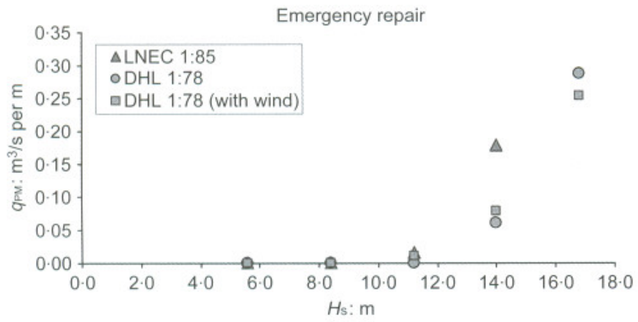


Figure 7. Mean overtopping discharges (prototype values) for tests carried out at DHL (with and without wind) and at LNEC (without wind) for the standard cross-section of the emergency repair for $T_p$ = 20 s and +4·0 m CD

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 and 24 s for the emergency repair and for alternative I and of 20 s for alternative II. The GF value varied between 0·60 and 0·70. Three significant wave heights were tested (9·8, 11·9 and 14·0 m) for a water level of +4·0 m CD. Each test comprised three runs, each of 12 h (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 were counted visually. Overtopping quantities were measured by collecting water in an overtopping tray, 30 cm wide by 106 cm long. Overtopping started for a significant wave height of 9·8 m, and for 14 m overtopping was very severe. As far as overtopping was 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 0·11 m³/s per 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 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: (*a*) $T_p$ = 16 s, GF = 0·60 and GF = 0·90; and (*b*) $T_p$ = 22 s, the design storm, GF = 0·6 and two water levels (+1·0 m CD and +4·0 m CD).

The total overtopping volume was collected during 3 h (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 0·072 m³/s per m for a significant wave height of approximately 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 a distance of about 500 m seaward from the foundation of the breakwater, which was located at approximately −41 m CD. 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, 20 and 24 s, significant wave heights of 5·6, 8·4, 11·2, 14·0 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 h (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 a significant wave height of 8·4 m, and for 14 m the overtopping was very severe (0·51 $m^3$/s per m).

### 2.2.3 Root portion

To check and optimise 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 and 40 t Antifer units, both with a density of 2·6 t/$m^3$.

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 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 four to five runs with increasing values of the significant wave height, each lasting 12 h (in the prototype).

The overtopping volume was collected during 80 min in a 30 cm wide by 120 cm long tray. For the last 20 min of each run, a video continuously recorded the overtopping conditions. The results showed that the cross-section with 40 t Antifer units could replace the cross-section with 60 t units with the same degree of overtopping and with the maximum mean overtopping discharge reaching around 0·0039 and 0·061 $m^3$/s per 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 the significant wave height.

## 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 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 2-D physical model tests (Figure 8).

(*a*) Solution 1: 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.
(*b*) Solution 2: to be used between berths 2 and 1, differs from solution 1 mainly in the crest area (in shadow in the figure).
(*c*) Solution 3: 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).

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.

### 3.2 Physical model tests

The preliminary studies for the final rehabilitation entailed an integrated analysis of alternatives involving their length and their reinforcement (Consulmar, 2006). These studies included the absolute and comparative evaluation of the alternatives regarding wave disturbance (Jensen and Hebsgaard, 2007) and breakwater stability and wave overtopping (LNEC, 2008), using three-dimensional (3-D) and 2-D physical model tests, respectively.

To check the suitability of the three proposed solutions for the cross-sections of the breakwater, 2-D physical model tests of stability and overtopping were performed during 2008 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 (LNEC, 2008). The geometrical scale was 1 : 60. The foreshore was represented by different concrete slopes starting at −55·0 m CD up to a level of −45·0 m CD at the toes of the model breakwaters.

Two prototype still-water levels were tested: +0·0 m CD and +4·0 m CD. Irregular waves conforming to a mean Jonswap spectrum were employed with significant wave heights and peak periods in front of the wave-maker ranging from 4 to

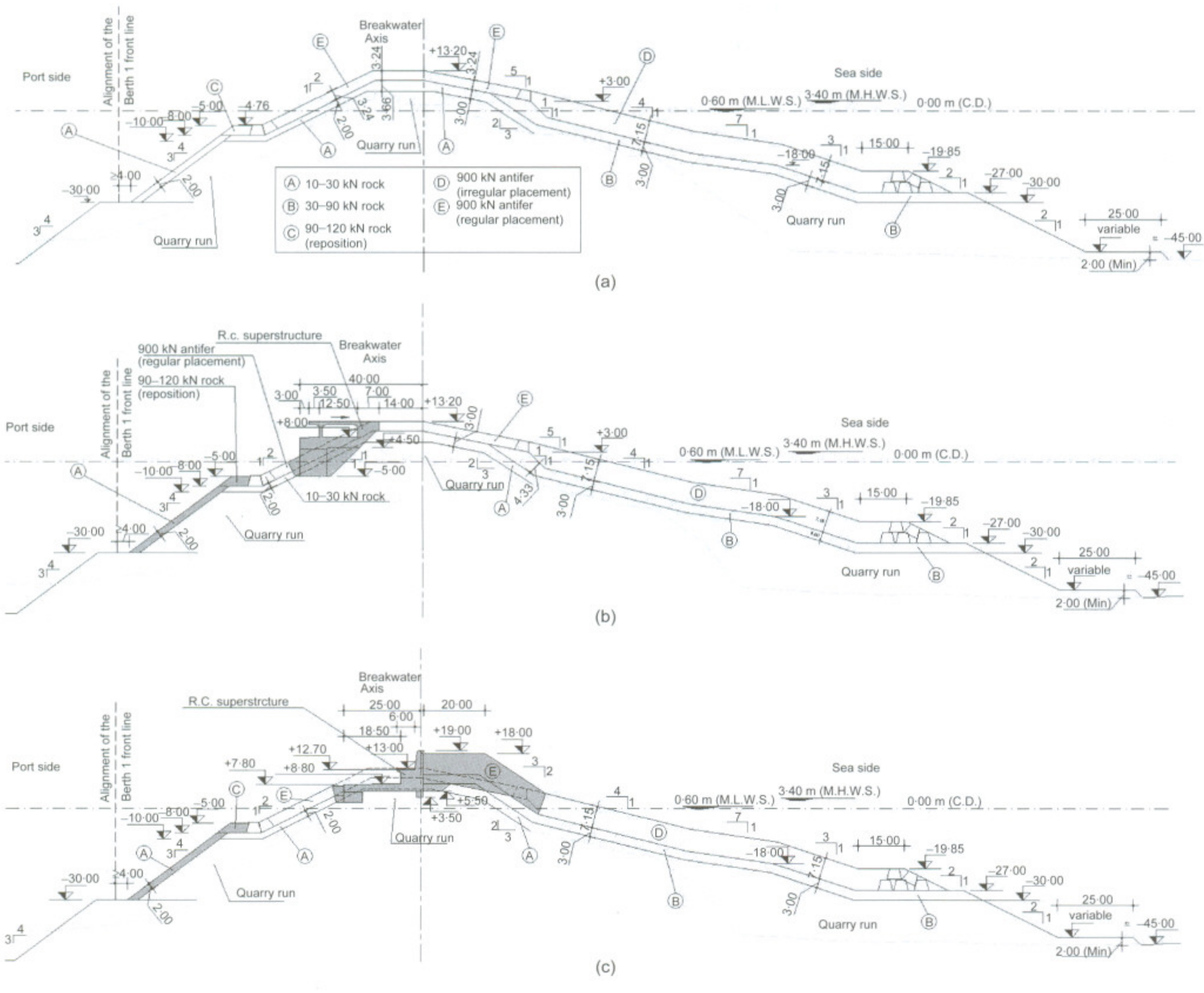

Figure 8. Proposed cross-sections for the final breakwater rehabilitation: (a) solution 1; (b) solution 2; (c) solution 3 (dimensions in m, elevations in m above chart datum)

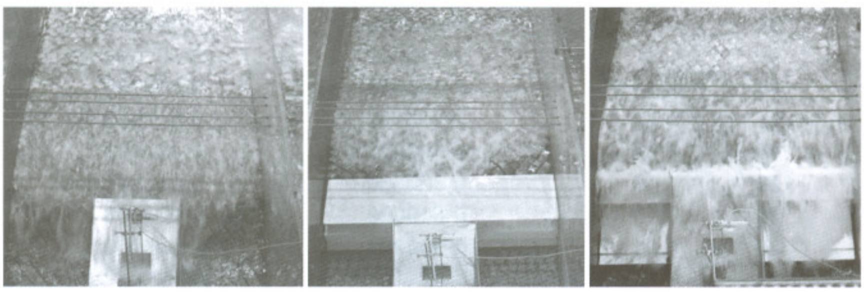

Figure 9. Overtopping observed during physical model tests for solutions 1 (left), 2 (middle) and 3 (right)

14 m and 10 to 20 s in prototype values. The test duration ranged from 23 to 46 min for each run (approximately 1000 waves).

The measurement of the water level variation inside a tank deployed at the back of each model structure, together with the calibration curve of a pump located in the tank, allowed the determination of the mean overtopping discharges per metre length of structure, $q_{PM}$ (Figure 9).

A detailed description of the tests can be found in LNEC (2008) and Reis *et al.* (2009b). 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 in front of the structure, at depth $d$, presented at prototype scale.

The table shows that, unsurprisingly, given its higher crest level, solution 3 was the least overtopped structure. Despite showing a similar behaviour, solution 1 presented mean discharges slightly smaller than those for solution 2, due to its wider permeable crest and due to the absence of an impermeable superstructure at the back of the breakwater armour.

The measured discharges were compared with the acceptable values defined by the designer (Consulmar) as being appropriate for the proposed works, based on current guidance (Pullen *et al.*, 2007; US Army Corps of Engineers, 2006): 0·5 to 1 litre/s per m (at prototype scale). Although some measured discharges exceeded the acceptable values, the rubble mound breakwater was stable in all three proposed solutions for all test conditions and 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 of the above results with the results from previous physical model studies, on the local day-to-day operating experience and on economic and risk considerations. Thus,

| Test | $d$: m | $T_p$: s | $H_s$: m | $q_{PM}$: m$^3$/s per m | | |
|---|---|---|---|---|---|---|
| | | | | Solution 1 | Solution 2 | Solution 3 |
| 1 | 50·0 | 10 | 4 | 0·000000 | 0·000000 | 0·000000 |
| **2** | | **10** | **6** | **0·000000** | **0·000000** | **0·000000** |
| 3 | | 12 | 8 | 0·000000 | 0·000000 | 0·000000 |
| **4** | | **14** | **9** | **0·000000** | **0·000000** | **0·000000** |
| 5 | | 16 | 10 | 0·004100 | 0·005100 | 0·000028 |
| 6 | | 18 | 11 | 0·016000 | 0·019000 | 0·001100 |
| **7** | | **18** | **11·5** | **0·022000** | **0·025000** | **0·001800** |
| 8 | | 12 | 12 | 0·001800 | 0·002100 | 0·000190 |
| **9** | | **16** | **12** | **0·014000** | **0·015000** | **0·000280** |
| 10 | | 20 | 12 | 0·033000 | 0·060000 | 0·004800 |
| **11** | | **20** | **13** | **0·140000** | **0·160000** | **0·014000** |
| **12** | | **20** | **14** | **0·170000** | **0·200000** | **0·035000** |
| 13 | 54·0 | 10 | 4 | 0·000000 | 0·000000 | 0·000000 |
| **14** | | **10** | **6** | **0·000000** | **0·000000** | **0·000000** |
| 15 | | 12 | 8 | 0·000000 | 0·000000 | 0·000000 |
| **16** | | **14** | **9** | **0·005100** | **0·006400** | **0·000610** |
| 17 | | 16 | 10 | 0·035000 | 0·043000 | 0·002700 |
| 18 | | 18 | 11 | 0·120000 | 0·150000 | 0·014000 |
| **19** | | **18** | **11·5** | **0·210000** | **0·250000** | **0·044000** |
| 20 | | 12 | 12 | 0·008400 | 0·009600 | 0·000580 |
| **21** | | **16** | **12** | **0·140000** | **0·160000** | **0·033000** |
| 22 | | 20 | 12 | 0·320000 | 0·350000 | 0·075000 |
| **23** | | **20** | **13** | **0·610000** | **0·650000** | **0·140000** |
| **24** | | **20** | **14** | **0·900000** | **0·930000** | **0·250000** |

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

solution 3, which is similar to the existing cross-section between berths 3 and 2, was selected for detailed design. Solution 2 was abandoned and a cross-section similar to solution 1 was retained for future 3-D physical model tests of stability and overtopping of the section south of berth 1.

Despite the fact that no measurements were carried out to analyse how the reduction in overtopping discharge improved wave conditions behind this part of the breakwater, visual observation of the physical model tests suggested that this improvement was not significant. Although the local wind conditions were not reproduced during the tests, the sheets of water adhered to the structure's crest, reaching little distance behind the breakwater. In particular, the berth 1 caissons (graphically represented on the side walls of the flume), were hit only by the biggest waves, on their lower sections, close to the water level; that is, overtopping was not found to be the critical cause of wave disturbance in the harbour basin.

## 4. Applicability of other overtopping methodologies

The physical model overtopping data collected at LNEC for the solutions proposed for the final rehabilitation of the Sines west breakwater were used to check the applicability of the numerical model, Amazon, to porous structures (Section 4.1) and the applicability of the methodologies/tools recommended by Pullen *et al.* (2007) to estimate mean overtopping discharges (Section 4.2).

### 4.1 Amazon numerical model

Amazon was developed, originally, at Manchester Metropolitan University (Hu, 2000) and it comes as both a 1-D model and as a 2-D plan model. The 1-D version was used and briefly described here.

Amazon is based on solving the NLSW equations and is numerically very stable and robust. The pressure is assumed hydrostatic and the equations describe water motions in terms of the instantaneous total water depth and the depth-averaged velocity. The equations are solved using a high-resolution finite-volume method that is second-order in time and space. It uses a 'zero-equation' turbulence model.

The original version of Amazon did not explicitly account for porous flow. The recent development of the porous flow model includes the addition of one porous layer to the original model design and the porosity is taken as constant for the whole porous element. To govern the water exchange between the porous cells, both the Darcy equation and the Forchheimer equation are implemented in Amazon. A detailed description of the model can be found in Hu (2000) and in Reis *et al.* (2008, 2009a).

The physical model overtopping data collected at LNEC for solutions 1 and 2 have been used to check the applicability of Amazon to porous structures. Reis *et al.* (2009b) describe in detail the application of Amazon to estimate mean overtopping discharges for solutions 1 and 2, for the 12 test conditions

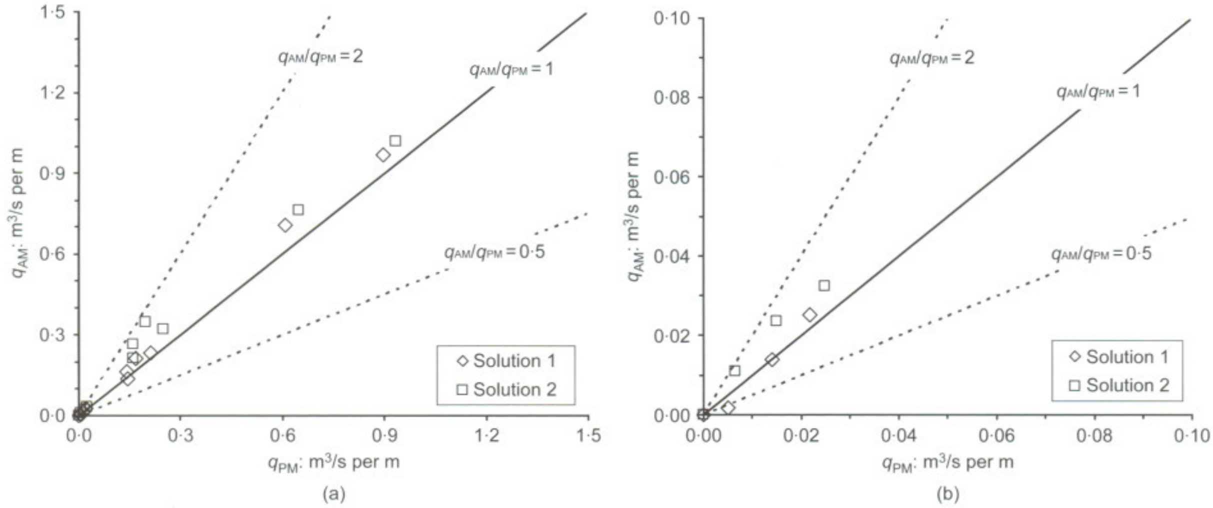


Figure 10. Comparison between the Amazon, $q_{AM}$, and the physical model, $q_{PM}$, mean overtopping discharges obtained for solutions 1 and 2 (all values scaled up to prototype) using the Darcy equation with IP = 0·125 m/s and $n$ = 0·4: (a) all results; (b) results for discharges less than 0·1 m$^3$/s per m

shown in bold in Table 4. 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.

Figure 10 presents the Amazon results versus the physical model results, all scaled up to prototype values, 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 figure shows that there is a good agreement between the Amazon results and the physical model data, although Amazon tends to slightly overpredict the discharges, especially for solution 2.

### 4.2 Neural network and empirical methods

The physical model overtopping data collected at LNEC for solutions 1 to 3 for the 12 test conditions shown in bold type in Table 4 were also 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. The methodologies/tools were the Clash neural network (NN_Overtopping) and the empirical methods calculation tool, both available online (Pullen *et al.*, 2007).

NN_Overtopping, a prediction tool based on neural network (NN) modelling, was developed as part of the Clash European project (Coeveld *et al.*, 2005; Van Gent *et al.*, 2005) to predict Froude-scaled mean wave overtopping discharges and the associated confidence intervals for a wide range of coastal structure types (such as dikes, rubble mound breakwaters, and caisson structures). In addition, prototype mean overtopping estimations, allowing for scale and model effects, are provided. This tool is based on a database of about 8400 test conditions, which originate from small-scale tests at many different laboratories (Steendam *et al.*, 2004), and it uses 15 hydraulic and structure input parameters. Nevertheless, Coeveld *et al.* (2005) suggest that the reliability of the predictions should be verified using dedicated physical model tests for the particular wave conditions and structure geometry under consideration.

The empirical methods calculation tool was also developed as part of the Clash European project for prediction of overtopping. The tool is based on the application of equations that relate the overtopping response (usually mean overtopping discharges) to key wave and structure parameters. The form and coefficients of the equations are adjusted to reproduce results from physical model tests or field measurements. The structures with governing overtopping equations are smooth sloping structures, rubble mound structures and vertical structures.

In order to apply the methodologies/tools mentioned above, each of the three solutions had 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 into any of the structural types/configurations available, several of these representations were tested under the conditions listed here (Table 5).

(*a*) Solutions 1 and 2.

(i) Armoured slope with a berm at −19·85 m CD.
(ii) Armoured slope with a toe at −19·85 m CD (no berm).
(iii) Armoured slope with a zero-width berm at −7·00 m CD and a toe at −19·85 m CD.

| 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 and neural network |
| | 3C | Armoured composite slope with wall (zero berm width) | Empirical methods and neural network |

Table 5. Adopted representations of solutions 1 to 3 and corresponding recommended methods for overtopping estimation according to Pullen *et al.* (2007)

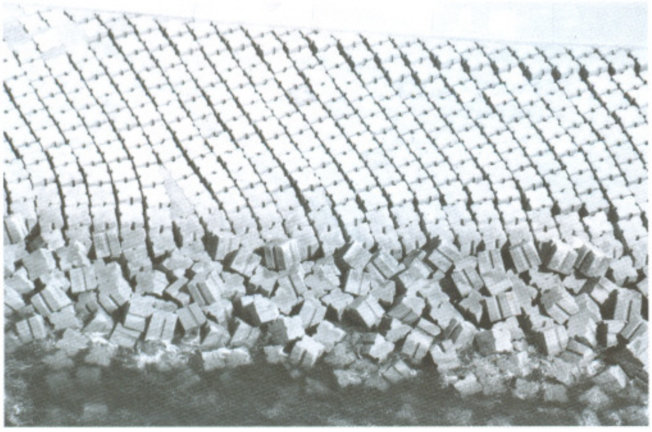

Figure 11. Sines west breakwater: detail of Antifer placement at the upper part of the armour slope and at the crest berm

(*b*) Solution 3.

(i) Bermed armour (berm at −19·85 m CD) with wave wall.
(ii) Armoured composite slope with wall and with a toe at −19·85 m CD (no berm).
(iii) 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, $\gamma_f$, of 0·47 or 0·5 for Antifers, but they have been based on physical model tests carried out with two layers of Antifer blocks placed randomly on a 1 : 1·5 slope (Pearson *et al.*, 2004). For solutions 1 and 2, the Antifers are placed in one layer only and for solutions 1 to 3, they have a regular placement at the upper part of the slope and at the crest berm (Figures 8 and 11), which increases run-up and overtopping considerably. A sensitivity analysis was performed on $\gamma_f$, considering values between 0·5 and 0·8 for the different slope sections of the breakwater.

For each of the above representations, the applied overtopping methods are described in Table 5.

The mean overtopping discharges obtained with the NN ($q_{NN}$) for solutions 1 to 3 using the representations shown in Table 5 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 $\gamma_f$, showing that, as expected, $\gamma_f$ has a major influence on the results. Consequently, the NN predictions for the 12 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. This method provides a simple estimate of a combined roughness/permeability factor, which varies with the test conditions considered and is calculated by weighting the different roughness/permeability factors and by including the lengths of the corresponding sections between SWL − $0{\cdot}25R_{u2\%smooth}$ and SWL + $0{\cdot}50R_{u2\%smooth}$, where SWL is the still-water level and $R_{u2\%smooth}$ is the run-up level exceeded by 2% of the waves incident on a smooth slope. The combined $\gamma_f$ values obtained for solutions 1 to 3 varied between about 0·51 and 0·70. The mean overtopping discharges obtained with the NN for solutions 1 to 3 using representation B are presented in Figure 12, 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 underpredict 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 mean overtopping predictions for the 12 test conditions using the empirical methods were carried out, as for the NN, with $\gamma_f$ 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 $\gamma_f$ has a major influence on the predicted discharges. The mean overtopping discharges obtained with the empirical methods for solution 3 are presented in Figure 13. Note that as far as the empirical methods are concerned, representations 3B and 3C are the same.

The results show that the empirical methods overpredict the physical model results to a large extent, warning of the fact that direct application of these methods is limited to particular structural configurations and wave conditions, and even for these cases there are gaps in data, particularly for the low overtopping discharges for which structures are usually designed. This overprediction is more noticeable for deterministic calculations, since they include safety margins to account for the uncertainty of the predictions, whereas probabilistic calculations are based on average trends.

## 5. 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

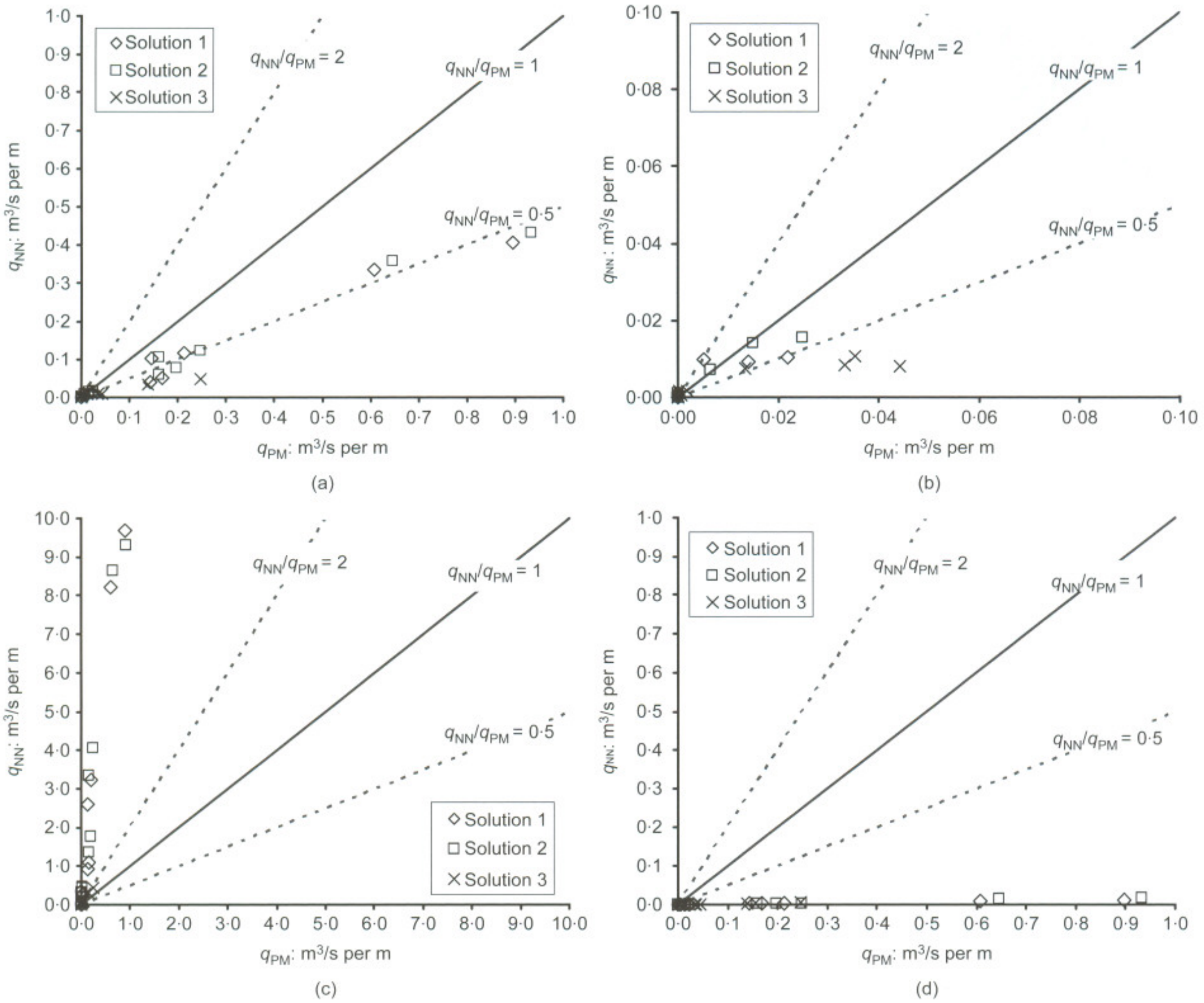


Figure 12. Comparison between the neural network, $q_{NN}$, and the physical model, $q_{PM}$, mean overtopping discharges obtained for solutions 1 to 3 using representation B (all values scaled up to prototype): (a) mean values of $q_{NN}$, all results; (b) mean values of $q_{NN}$, results for discharges less than 0·1 m$^3$/s per m; (c) upper limits of the 95% confidence intervals; (d) lower limits of the 95% confidence intervals

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 and a brief description of the 2-D 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 structurally 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. The Clash neural network was applied to the three solutions, whereas the empirical methods were applicable

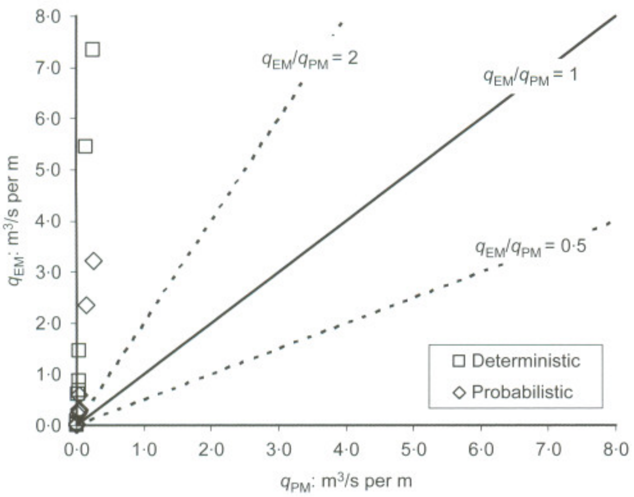


Figure 13. Comparison between the empirical method, $q_{EM}$, and the physical model, $q_{PM}$, mean overtopping discharges obtained for solution 3 (all values scaled up to prototype)

to solution 3 only. Amazon was applied to solutions 1 and 2; it was not applied to solution 3 as 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 for solutions 1 and 2, although Amazon tended to slightly overpredict the discharges, especially for solution 2. The NN mean overtopping discharges tended to underpredict 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 empirical methods over-predicted the mean overtopping discharges to a large extent, warning of the fact that direct application of these methods is limited to particular structural configurations and wave conditions.

## Acknowledgements

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