

Breakwater control system and structural analysis: physical and numerical modelling (Port of Funchal, Madeira Island, Portugal)

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Abstract The objective of this work was to analyze the behavior of the Funchal Port (Madeira Island, Portugal) breakwater using physical and numerical models in order to analyze the evolution of the underwater layers, combining information between the two models as well as simulating storm conditions and atypical docking/undocking impacts of cruise ships. Regarding the physical model using a wave generation system, the data collected and the test conditions were addressed considering different scenarios (return periods of 20, 50 and 100 years). It was found that for return periods of 20 years, the behavior of the infrastructure as a whole was stable. For return periods of 50 years, wave overtopping became more frequent and intense, and began to impact the superstructure. For the 100-year return period the behavior of the infrastructure was found to be unstable, with persistent wave overtopping. The morphology of the submerged layers also changed. The main objectives of the numerical model approach were to estimate the natural frequencies of vibration (when materials suffer a considerable modification, a change in their frequencies of vibration can be detected), and to calculate the expected displacements corresponding to cruise ship docking processes

(agitation in a protected area) and the associated maximum maritime (agitation in an exposed area) forces. The maximum displacements obtained for the two cases were about 0.03 m, which is a high value. Considering the results obtained from this work, the Port authorities decided to maintain the monitoring program and perform an underwater video inspection.

Keywords Port infrastructures · Physical model · Numerical model · GNSS

Introduction

Port infrastructures require special attention due to their high exposure to extreme weather conditions and their location along the sea. These installations are of great importance to economic and social development, and provide the fundamental infrastructure for foreign trade and internal market exchanges.

It is essential that port infrastructures have rigorous and precise monitoring, using the appropriate tools and procedures for loss control, and where possible to predict any inherent risks to these types of structure in order to schedule interventions and provide more efficient maintenance procedures.

Several approaches to breakwater monitoring have been reported in recent years. The methodologies employed in these studies include several surveying techniques such as Global Navigation Satellite Systems (GNSS), remote sensing data and underwater video inspections (Pilarczyk and Zeidler 1996).

Non-destructive approaches such as Global Positional System (GPS) techniques can be used to detect geometric deformation. However, these techniques do not allow any conclusions related to fatigue and corrosion of the infrastructure materials (Meo et al. 2002). These types of deterioration

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develop internally and are extremely complex to quantify. Control systems with warnings or alarms using the carrier phase of a GPS signal through single frequency receivers have been used for monitoring different types of infrastructures, and are able to determine displacements within sub-centimeter accuracy (Knecht and Manetti 2001). Physical modelling has also been used in different applications. Physical two-dimensional (2D) or three-dimensional (3D) breakwater modeling is a common practice, targeting a variety of studies such as structural stability, wave overtopping and sediment dynamics. New experimental techniques, automated data acquisition and analysis systems, rapid processing and increased data storage capabilities also provide useful information for the validation of numerical models (Frostick et al. 2011). Other advantages of physical models include study at a lower level of simplification to confirm theoretical results through numerical measurements, to obtain measurements from complex phenomena which are inaccessible from theory, to test extreme conditions, to examine a wide variety of environmental conditions, and to monitor immediate visual feedback. Despite all these advantages, there are still some problems to solve regarding physical modelling, such as scale effects, incomplete modelling, laboratory effects and the cost of the installation and maintenance. For a physical model to be reliable it is important that the model is accurate and realistic, which is achieved through calibration. Ideally, the calibration of the whole model should be carried out from known field conditions, which is seldom possible in practice (Taveira-Pinto et al. 2008). Numerical modelling also has great applicability in several fields in regard to coastal studies. Nowadays, with the rapid development of numerical models, hydrodynamic and morphodynamic models are becoming more and more widespread in the coastal engineering community. A variety of numerical models are available, including multi-dimensional (1D, 2D and 3D) models. These models are mostly based in simplified Navier–Stokes equations. The most widely-used numerical techniques are based on the finite difference method, finite element method, finite volume method, etc. (Sharifahmadian 2015).

Safety control of the structures is based on the comparison of the observed results (in situ measurements) with the results obtained from physical and numerical models. These results allow the researcher to: i) understand the observed behavior; and ii) predict future behavior for usual and extreme conditions.

The main objective of this work was to analyze the behavior of the Funchal Port (Madeira Island, Portugal) breakwater, using physical and numerical models in order to study the evolution of the underwater layers by combining information between these two types of models (physical and numerical). The simulation of storm conditions and atypical docking/undocking impacts of cruise ships were also performed. The results of in situ campaigns (Pereira et al. 2015) were used to calibrate these models.

Study area

Today the Funchal Port is exclusively dedicated to tourism (docking cruise ships), whereas in the past it was used for commercial purposes. It is located in the southwestern part of Madeira Island (Fig. 1) in a small bay between Ponta da Cruz and Barreirinha beach. The Funchal Port was built in a stepwise manner, and continuous improvements were made, ending with the construction of the Funchal Maritime Station in 2010 (Pereira 2014).

The Funchal Port breakwater is a mixed structure characterized by a superstructure protected with a tetrapod armor layer 1100 m in length. The depths range up to -30 m (CD), while the breakwater crest elevation is about +11 m (CD). The breakwater has shown some damage in recent years, mainly in the most extreme part – its head (Fig. 2). This damage resulted from adverse weather conditions and from the docking/undocking impacts of cruise ships, whose weight and size have increased in the recent years. High winds, waves, storms and micro-earthquakes are other possible phenomena responsible for the breakwater damage.

Until now, the breakwater monitoring of Funchal Port has only been done by visual inspection, which is clearly insufficient because it does not allow quantifying any displacements suffered by the structure. In light of this, combined with the fact that cruise ships are becoming larger (with more surface exposed to wind), it is important to ensure that this infrastructure is prepared to meet the new functional requirements.

Methodology

A physical model, by definition, is an approximate representation of all or part of something that exists or will exist, in a reduced or amplified form in order to collect useful information, thereby offering a deeper understanding. Physical models are often used as guidance in defining infrastructure characteristics in projects, as well as to study and understand the initial conditions and apply further adjustments that are essential for the protection of these infrastructures. Physical models are also used to supplement numerical models or together with them, and are often a vital tool for the calibration of these models. Physical models can be 2D or 3D, depending on the study purpose. There are various strengths and limitations in studies conducted with physical models. Additional information is essential in order to understand infrastructure behavior, obliging the creation of realistic scenarios and responses to expected boundary conditions. Due to their scale reduction, physical models allow performing various tests and creating different scenarios and then returned to their initial condition. They also allow performing an analysis within shorter times and in continuous modes if necessary.

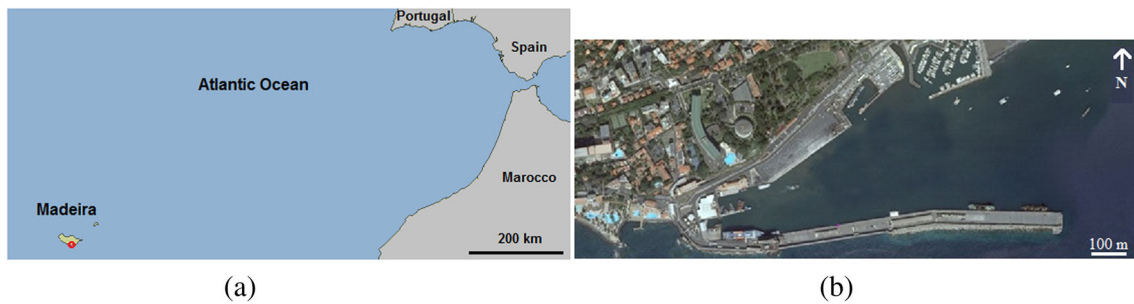


Fig. 1 a Madeira island location b Aerial photo of Funchal Port

During recent years, in order to overcome some of the physical model limitations several numerical models have been developed, based mainly on nonlinear shallow water equations (Brocchini and Dodd 2008), and more recently on Boussinesq (Engsig-Karup et al. 2008), or Navier–Stokes type models (e.g., Li et al. 2004). Most of them have been primarily applied to predicting hydraulic response design parameters associated with wave reflection, transmission, run-up and overtopping (Guanche et al. 2009).

In this work a physical and a numerical model were implemented in order to analyze the Funchal breakwater behavior. In Fig. 3 a flowchart is presented with the methodology used in this study.

In situ campaigns

The topic of this study relies on a breakwater monitoring system and structural analysis through physical and numerical modelling. Therefore, it is important to describe the employed monitoring system (in situ campaigns). The applied monitoring system was divided into:

- Impact of actions on the breakwater superstructure—which aims to analyze the impact on the breakwater behavior from the docking and undocking of cruise ships;
- Wave impacts on tetrapods—in order to quantify the tetrapod macro-movements so as to better understand which areas of the breakwater are most exposed to wave action.

The displacements were evaluated through different techniques/methods:

- Geodetic techniques using GNSS;
- Classical survey techniques;



Fig. 2 Damages visually identified

- Vibration measurements using high precision accelerometers (in order to identify the main natural breakwater vibration frequencies).

To correctly implement the GNSS methodology and to assure that the results are consistent with the expected accuracy (7.5 mm), several test campaigns were performed in order to simulate all the conditions and parameters involved (Pereira et al. 2015). Based on the results obtained we selected 2-hour observation periods, 10° of elevation mask, a 1 s observation rate, precise GPS ephemerides using the GPS system only and the use of troposphere models (Pereira et al. 2015).

Considering the ten absolute stability breakwater monitoring campaigns performed (a network constituted of 12 object points in the breakwater cap—see Fig. 4a) for almost 3 years, the displacements identified were less than 7.5 mm. It was possible to conclude, despite the visible damage, that the breakwater superstructure did not suffer a significant degree of damage during the observation period. Considering the results of the cruise ship docking/undocking monitoring campaigns, it was concluded that more observation is needed in order to correctly quantify the influence of these processes in terms of permanent structure displacements.

The natural breakwater vibration frequencies found were the same in both the February 2012 and September 2012 campaigns. This allowed concluding that during this period no

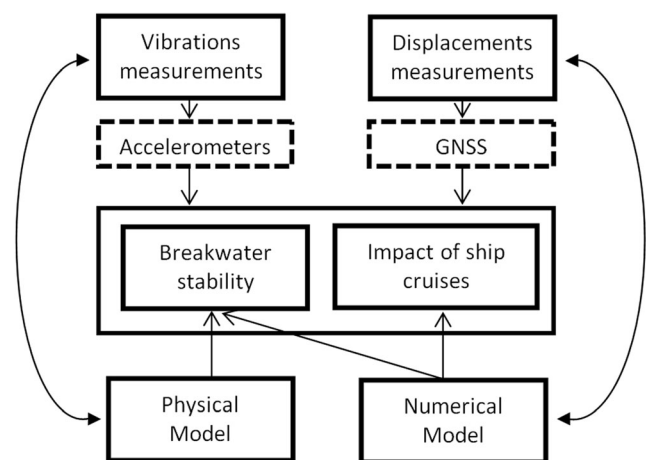


Fig. 3 Flow chart with the methodology adopted

Fig. 4 **a** Breakwater superstructure stability physical marks; **b** Tetrapods



major internal changes occurred in the blocks and layers that constitute the breakwater.

In the tetrapod monitoring campaigns, oscillations and individual block movements ranging from a few centimeters to several meters were found, indicating that wave actions are very intensive on the exposed breakwater armour layer, which is of extreme importance in the defense against direct wave impacts. Figure 4 (b) presents the Funchal breakwater tetrapods. More details about this matter can be found in Pereira et al. (2011, 2013, 2014, 2015).

Structural safety monitoring should be based on the comparison of in situ observations with the results of physical and numerical models. The simulation of different scenarios allows the results obtained to be transferred into reality. Based on the physical model, when deviations are detected between the observed and predicted behavior it is necessary to propose the maintenance/rehabilitation of these structures for the prevention of accidents/incidents—a critical factor in adequate port infrastructure management. Numerical models can contribute to a deeper understanding of the phenomena, and also to improving the characterization and parametric configuration of physical models.

Physical model

This section describes the adopted methodology and the implementation steps regarding the physical model. The aim of using this model was to understand the evolution of the various layers which constitute the breakwater, from its base to the

superstructure top, identifying the external factors and hypothetical actions that contribute to the structural evolution. This work also tried to identify the main reasons for the visible vulnerabilities found in the breakwater superstructure (Fig. 2).

The experimental conditions, equipment used and its characteristics, the software employed, the steps of model construction and the scale adopted will be described. The wave generation system, the data collection, as well the test conditions for the different scenarios will be discussed considering return periods of 20, 50 and 100 years.

General specifications

The tests were done in the wave tank of the Hydraulics Laboratory at the University of Porto Faculty of Engineering, Department of Civil Engineering. The wave tank is 28 m in length, 12 m wide and 1.3 m deep. The wave tank has a wave generation system and a data collection and analysis system.

The wave generation system is an HR Wallingford piston type, multi-element system, consisting of 16 glass-fiber coated blades, each 75 cm wide and grouped into units of 8 paddles in two separate control modules, with an active wave reflection system. The system is monitored in a control room by two terminals that manage the whole system associated with the tests as shown in Fig. 5. In order to control the wave generation, the HR Wave Generation System was applied and the HR-DAQ-Data Acquisition and Analysis software program was used for the data collection and analysis. To measure the water elevation, for the purpose of ensuring that the



Fig. 5 Physical model monitoring system

Table 1 Maximum (HMAX) and significant wave height (Hs) for each of the return periods already considered (Ramos 2009)

Return Period (years)	HMAX (m)	Hs (m)
5	9.38	5.31
10	10.66	6.03
15	11.37	6.43
20	11.87	6.72
25	12.26	6.93
30	12.58	7.11
50	13.45	7.61
75	14.14	8.00
100	14.63	8.28

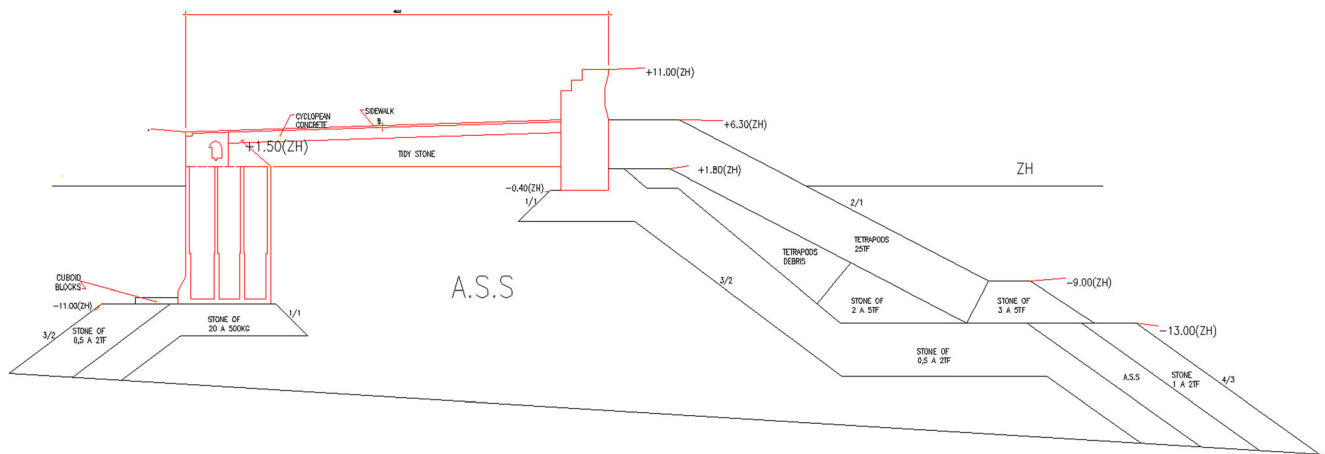


Fig. 6 Port of Funchal breakwater profile

effective wave heights in the guiding channel are the same as those that reach the physical model, hydrodynamic probes were used. These were also used to measure wave heights generated by the agitation system for the purpose of ensuring that the effective wave heights in the guiding channel are the same as those that reach the physical model.

The tests were intended to simulate the wave boundary conditions for well-defined return periods in order to understand the structural behavior during the tests and to confirm the visible and identified weaknesses in the in situ campaigns. The return periods (20, 50 and 100 years) were chosen according to the wave data available from previous work (Ramos 2009) presented in Table 1.

Geometric scale definition and model construction

The geometric scale chosen for the breakwater model was defined according to the experimental conditions and armour materials. The core was built with coarse sand in order to minimize scale effects due to viscosity. Of all the materials needed for the model construction, the tetrapods were those that most conditioned the scale model. Since the tetrapods that were available for the model weighed 156.6 g and the



Fig. 7 Physical model constructed

tetrapods in the breakwater weighed 245 kN the determination of scales were estimated according to Eqs. (1) to (4).

$$\lambda_w = \lambda_\gamma * \lambda_l^3 \tag{1}$$

$$\lambda_w = \lambda_l^3 (\lambda_\gamma = 1) \tag{2}$$

$$\lambda_w = \lambda_l^3 = \frac{0.1566}{25000} = \frac{1}{159642.4} \Rightarrow \lambda_l = \frac{1}{54.25} \tag{3}$$

$$\lambda_t = \sqrt{\lambda_l} = 7.365 \tag{4}$$

where, λ_w is the weights scale, λ_l is the geometric scale and λ_t is the time scale, λ_γ is the specific weight scale.

Thus, the length scale and the time scale were fixed in 1:7.365 and 1:54.25, respectively. The model was built according to the breakwater profile (Fig. 6) obtained from the final project delivered by the Administration of the Madeira Ports (APRAM). The breakwater profile consists of several layers of different materials, as shown in Fig. 6.

The breakwater layers are composed of:

- i) rock of many sizes in the core rocks, ranging from 4.9 to 49.0 kN in the intermediate layer;
- ii) rocks ranging from 29.4 to 49.0 kN in the rock fill toe berm;
- iii) pieces of broken tetrapods in the outer layers, and
- iv) tetrapods (245 kN) in the resistant mantle or armor layer.

The superstructure profile consists of a deflector curtain wall. The 2D model was built close to the tank wall so that it was possible to follow its evolution through the wave tank glass windows. The weights considered varied between 2.2 and 156.6 g. Figure 7 presents the physical model construction.

Table 2 Reynolds number considered

Wave height (m)	4	5	6	7	8	9
Re ($\times 10^4$)	5.13	5.74	6.29	6.79	7.26	7.70

Table 3 Duration for each test considering different return periods

Return period (years)	Duration(s)
20	700
50	770
100	840

Scale and laboratory effects

It is always necessary to take scale effects into consideration when physical model tests are conducted on a different scale than the prototype. To minimize these effects, the largest possible geometric scale compatible with the dimensions and characteristics of the wave tank should be used. In this case, since the gravitational actions are dominant, the Froude criterion was considered (Hughes 1993) to build the model. The main scale effects related to hydrodynamic processes come from the differences associated with viscosity and surface tension forces when the physical model is defined through Froude criterion (Hughes 1993). According to the adopted scales, the scale effects were considered to be negligible, in particular the scale effects due to viscosity, since it is accepted in the scientific community that for Reynolds number values higher than 6×10^4 the scale effects are minimal. In the present study, the Reynolds number was estimated through Eq. 5. The values obtained (Table 2) confirm that scale effects due to viscosity can be disregarded for the present study.

$$\text{Re} = \frac{UL}{\nu} \quad (5)$$

where U is the characteristic orbital flow velocity (m/s), L is the characteristic length of an armour block (m) and ν the water kinematic viscosity (m^2/s).

The laboratory effects that can influence the results obtained are related to reflections at the wall (near the model) and the screens that separate the channel from the rest of the tank, as well as the effects introduced by the wave generation system. The nonlinear interaction of generated waves can give rise to higher order harmonics in regular waves and false irregular long waves. The wave generation system used includes an active wave absorption system which minimizes the effects

of reflection along the generation paddles and a method to compensate the nonlinear effects. From past laboratory experience and visual assessment it was assumed that the techniques used were the most appropriate and these effects were minimized (Silva 2010).

Test conditions

Since the main objective of this work was to study the evolution in the configuration of the different layers of the breakwater profile from its submerged superstructure to its base, it was necessary to define the initial conditions. Based on the available data related to significant wave heights for each of the return periods presented in Table 1, the periods with the maximum significant wave height were selected. Accordingly, return periods of 20, 50 and 100 years were chosen. The water level used was 2.5 m (CD). The sea wave conditions were generated using JONSWAP spectra irregular agitation, according to the significant wave heights and respective periods already defined. The JONSWAP spectrum is effectively a fetch-limited version of the Pierson-Moskowitz spectrum, except that the wave spectrum is never fully developed and may continue to develop due to nonlinear wave-wave interactions for a very long time. More details can be found in Hasselmann et al. (1973).

Table 3 presents the duration for each test, considering different return periods. The duration criterion was to end each test when there was no evidence of further instability of the breakwater.

Five hydrodynamic probes were used. The first one was located at the entrance of the channel to provide information about the incident wave characteristics, and the other four were placed at specific distances in reference to the model (4.57, 4.45, 4.26 and 3.47 m, respectively), as shown in Fig. 8.

Tests performed

The tests were performed on two consecutive days: four tests on the first day and the remaining eleven tests on the second day. After each of the 15 tests, the statistical results of measured significant wave heights that effectively reached the

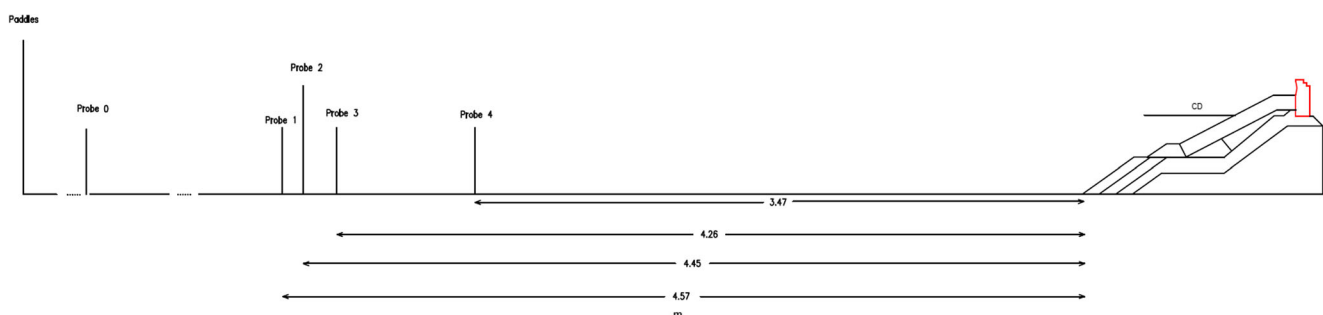
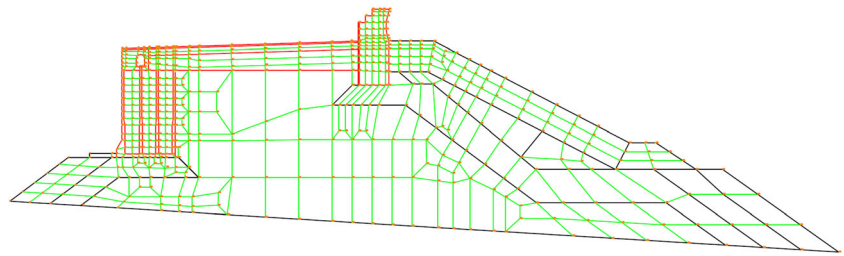
**Fig. 8** Location of the five hydrodynamic probes in the physical model

Fig. 9 Four nodal point finite element mesh



channel at each hydrodynamic probe in the wave tank were analyzed. In the case of the measured model the wave heights were higher or lower than expected (they were always lower than desired) and an increment or decrement, respectively, in the gain input of the generation control system was given. The gain values considered were 0, 1.10 and 1.15. After the 12th test the initial model setup was always considered.

For the last test, it was decided to hold a 2-hour test in order to simulate the evolution of the lower breakwater layers. This last test was repeated only for a return period of 50 years.

Numerical model

Numerical models used as reference elements can contribute to a deeper understanding of breakwater structural response,

and may also contribute to improving the characterization and parametric configuration of physical models. This relationship is bi-directional because knowledge generated in either case can be used as a supplement for the other one in several applications. There are several advantages to using numerical models, either for ease of re-testing, and for the simplicity of adjusting the test conditions at any stage of the study, compared to what this would involve in physical models.

Materials have natural frequencies of vibration. When they suffer any considerable modification, a change in their frequencies of vibration can be detected. This internal property of each material allows for temporal monitoring of materials and infrastructures, adding to a better understanding of their evolution, since any change in their vibration frequencies will indicate a change in the material constituents.

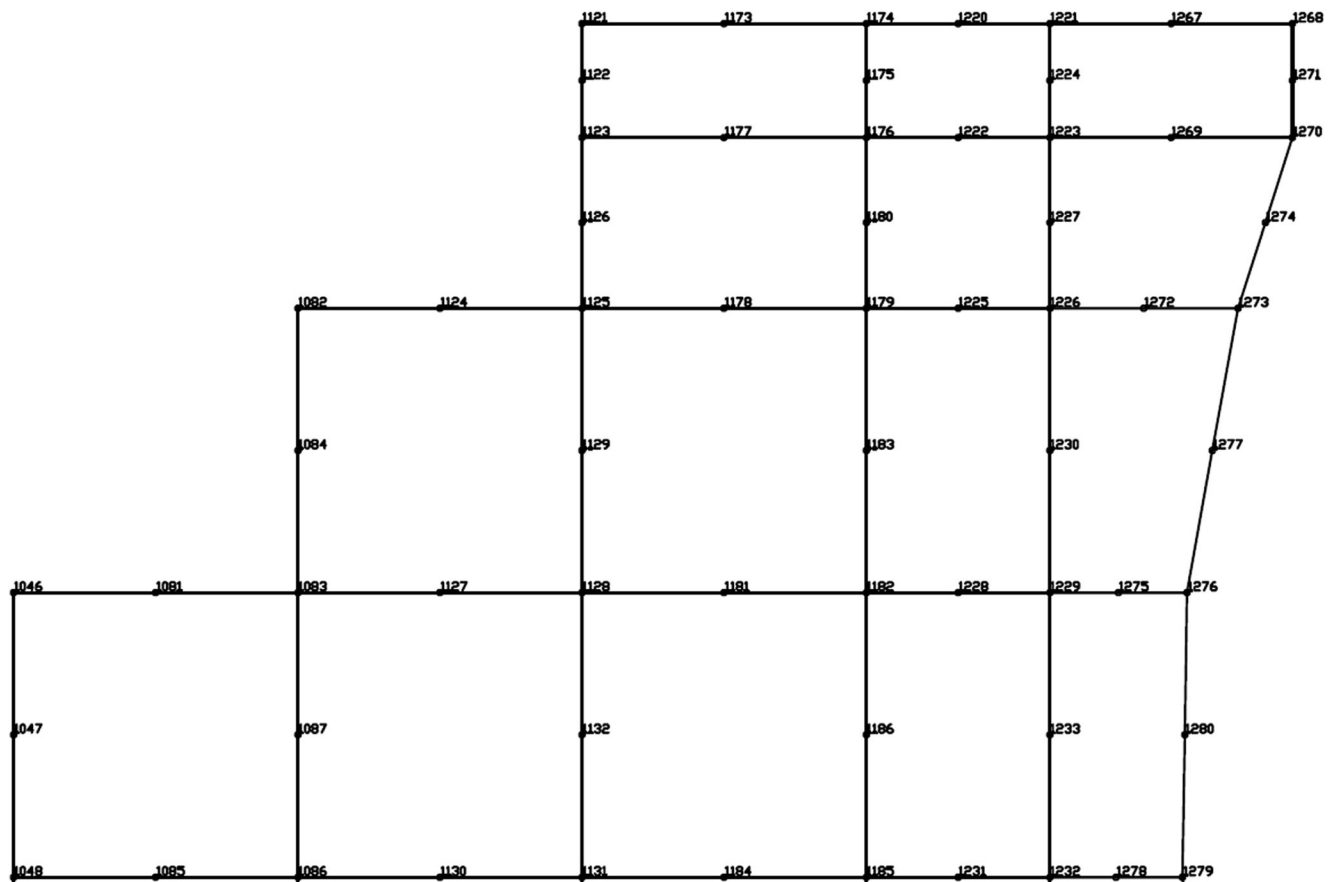


Fig. 10 Eight nodal finite element mesh detail (upper superstructure)

Table 4 Material layers characteristics

Material	Type	E (kN/m ²)	ν	γ (kN/m ³)
1	Concrete	2.00E + 07	0.20	–
2	Concrete with large aggregate	1.00E + 07	0.25	–
3	Concrete ducts	2.00E + 07	0.20	–
4	Walk / Bituminous	4.00E + 06	0.40	–
5	Broken stone	1.00E + 05	0.30	–
6	Stone 0.5 to 2 tf	5.00E + 04	0.30	–
7	Stone 1 to 2 tf	2.50E + 04	0.30	–
8	Stone 2 to 5 tf	2.50E + 04	0.30	–
9	Tetrapods	1.00E + 05	0.30	–
10	Pieces of broken tetrapods	1.00E + 05	0.30	–
11	All size	1.00E + 05	0.30	–
12	Stone 20 to 500 Kg	5.00E + 04	0.30	–
13	Parallelepiped Blocks	2.00E + 07	0.20	–
14	Other Concrete	2.00E + 07	0.20	–
15	Superstructure Concrete	2.00E + 07	0.2	24

E elasticity modulus, ν poisson's coefficient, γ self weight

The main objectives regarding this approach were to:

- i) Estimate the main natural frequencies of vibration in the infrastructure;
- ii) Calculate the expected displacements corresponding to cruise ship docking processes (waves in the protected area) and maximum maritime incident wave forces.

Further, it was intended to associate these forces with the damage identified in the breakwater superstructure, as already described. This characterization would allow a deeper understanding of the configuration evolution of the infrastructure and its behavior.

General specifications

For the numerical modeling of the breakwater of Funchal Port, an algorithm of 2D finite element (Zienkiewicz and Taylor 2000)

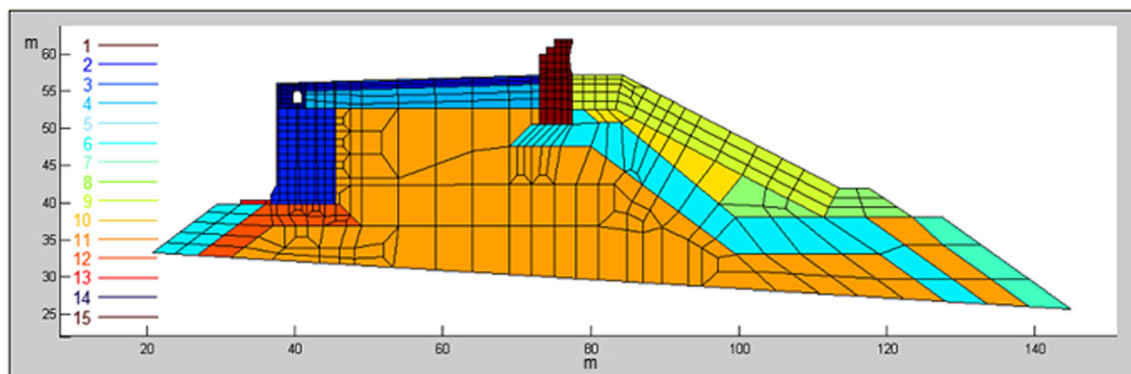
Table 5 Natural vibration frequencies calculated for the base conditions considered and for the calibrated model

Mode	Natural Vibration Frequency for the base conditions (Hz)	Natural Vibration Frequency for the calibrated model (Hz)
1	4.60	3.81
2	4.86	4.31
3	10.14	8.69
4	79.53	79.12
5	132.02	131.93
6	146.81	146.49

developed in Matlab® was used to simulate the breakwater's structural behavior under static and dynamic loads, admitting the hypothesis of isotropic materials with linear elastic behavior. The numerical model is based on the classical boundary value problem of solid mechanics involving the Navier equations, to be verified on the breakwater superstructure and at the foundation layers. The body forces include not only gravitational forces, but also inertia and damping forces in the case of dynamic behavior. Using the fundamental lemma of Variational Calculus and the Green-Gauss theorem, the weak formulation is obtained. Then, considering the finite element approach for a chosen domain discretization of N degrees of freedom (DOF), we obtain the well-known system of ordinary differential equations ($m\ddot{u} + c\dot{u} + ku = 0$), where m , c and k are the mass, damping and stiffness matrices of the discretized structure and u'' , u' and u are the vectors of nodal accelerations, velocities and displacements. The natural frequencies and mode shapes are computed solving the eigenproblem: $(k - \lambda m)\phi = 0$.

The dynamic analysis was performed in order to estimate the natural vibration frequencies of the breakwater superstructure and the corresponding modal configurations. The deformability parameters of the materials used in the landfills and in the superstructure were calibrated in order to ensure that the frequencies calculated are consistent with the material changes identified in the in situ campaigns.

To numerically simulate the breakwater response to the forces of cruise ship docking and sea wave impacts, the

**Fig. 11** Breakwater profile (the legend colors 1 to 15 correspond to the materials described in Table 4)

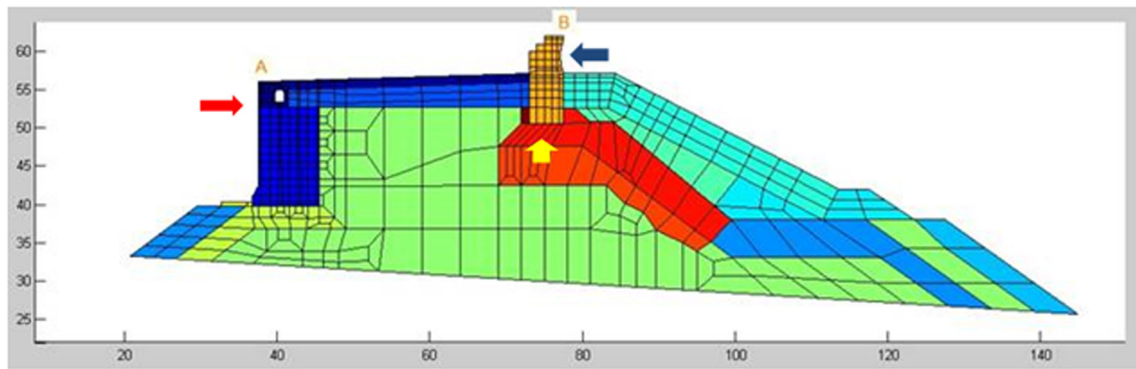


Fig. 12 Applied forces in finite element mesh

equivalent static forces were applied in the model. The model deformability parameters were calibrated in order to have a good agreement between the numerical and experimental values of the main natural frequencies.

Finite element mesh of the port of funchal breakwater and characterization model

To implement the numerical model it was necessary to create a representative finite element mesh of the breakwater superstructure and its foundation. The discretization process used square finite elements with 8 nodal points (isoparametric finite elements of 2nd degree). The superstructure and docking areas used a tighter mesh, while for the tetrapod layers and embankments larger finite elements were used. For the inner layers of smaller sized material a wider mesh was used.

First, a mesh of finite elements of 4 nodal points was implemented (Fig. 9) which was later converted to a mesh using the finite elements of 8 nodes referred to above (Fig. 10).

The assignment of features for each element was performed according to the constituent materials of the breakwater profile, as shown in Table 4.

In the analysis of the dynamic response of the breakwater superstructure the hypothesis of a massless foundation with elastic behavior was considered, as usual. Since the displacement estimations are not considered in association with their own weight forces (only those associated with the docking and agitation forces are considered) it was decided to consider only the values of the specific weights of the superstructure presented in Table 4 (Type 15).

Parametric study. Model calibration based on adjustment materials layers according to the in situ campaigns

After the finite element mesh implementation, a parametric study was performed in order to calibrate the deformability parameters of the model and to ensure that the calculated

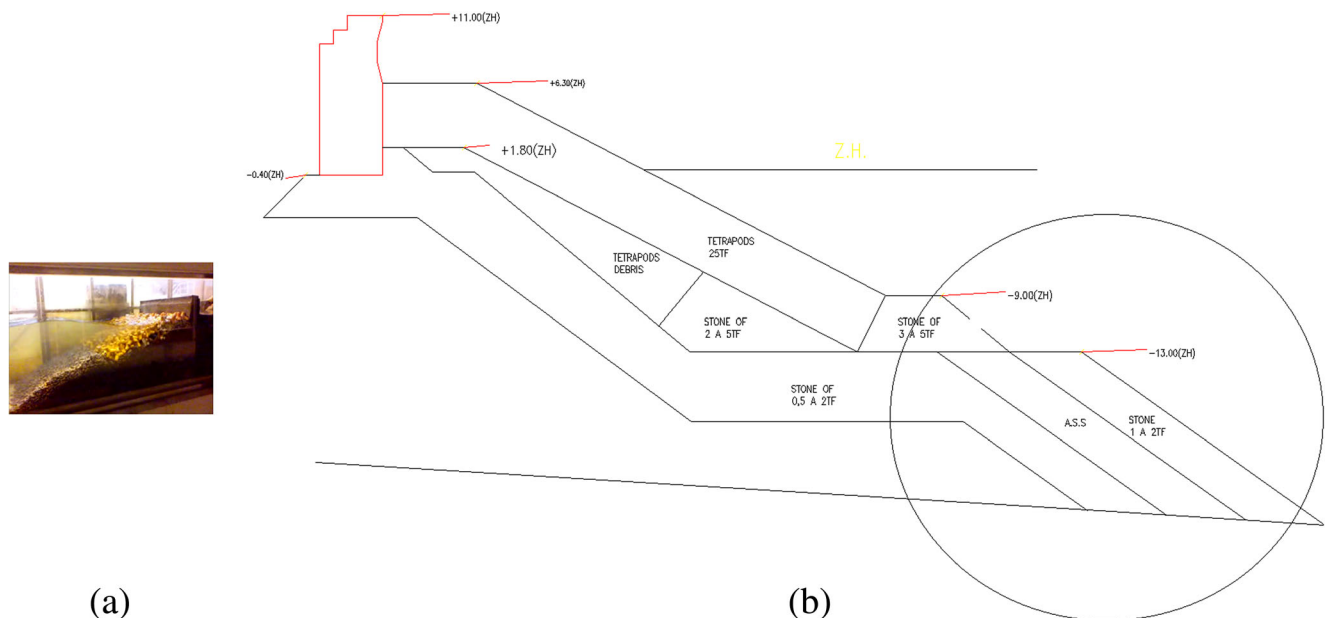
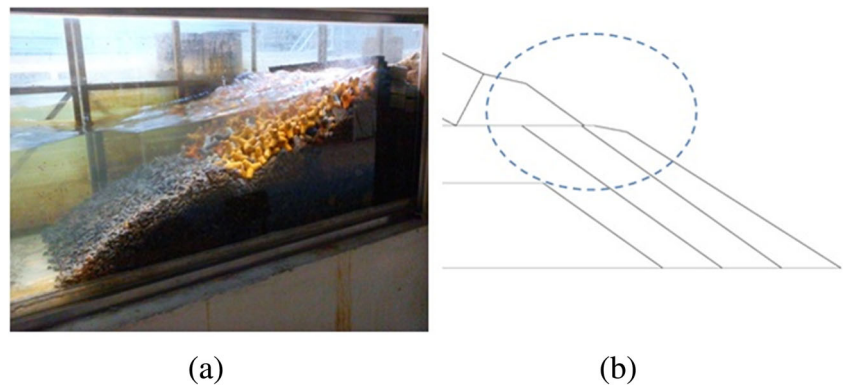


Fig. 13 Photograph (a) and b and drawing after a return period of 20 years test

Fig. 14 Photograph (a) and b and drawing after a return period of 50 years test



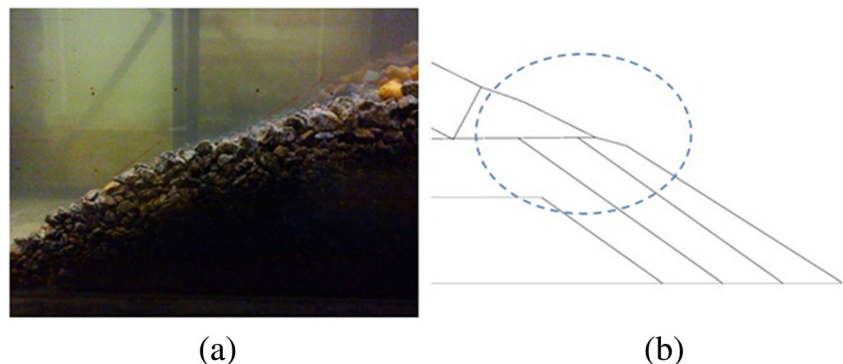
frequencies were consistent with the natural frequencies identified by the accelerometers in the in situ campaigns.

The first attempt considered the material characteristics and the base layers design presented in Table 4. Figure 11 shows the breakwater profile, with the representation of the different materials considered. Table 5 presents the first six natural vibration frequencies estimated considering the elastic properties listed in Table 4.

In the results obtained in the in situ campaigns using accelerometers, the conditions of greater deformability were observed in the layers which are more exposed to sea waves (armour layer and pieces of broken tetrapods), in the layers beneath the superstructure and in the area under the pieces of broken tetrapods.

The frequencies obtained using the parameters E (elasticity modulus) and ν (Poisson's coefficient) values presented in Table 4 do not correspond to those obtained in the in situ campaigns. To determine the E and ν values of the layers, a parametric study was conducted in order to adjust the deformability parameters of the model. The frequencies obtained in situ were compared with the frequencies obtained with the model (parametric study). The main frequencies identified were: 4.3 and 8.6 Hz (Table 5). After calibration, the numerical model was used to study different scenarios considering the impact from the docking of cruise ships and sea waves. The frequencies calculated for the calibrated model are presented in Table 5.

Fig. 15 Photograph (a) and b and drawing after a return period of 100 years test



For material types (see Table 4) between 7 and 10, 17, 18 and 21, modifications were needed in the numerical model inputs, regarding the elasticity modulus and Poisson's coefficient parameters. For the other materials no changes were needed.

Cruise ship docking and wave equivalents forces

The calibrated model (which is assumed to adequately represent the existing conditions at the breakwater) was used to study the expected maximum displacement due to cruise ship docking and consider the total force due to wave pressure propagation in storm episodes. According to Ramos (2009), the reaction force distributed by the fenders in a typical docking process is 2786 kN, while the horizontal forces and vertical forces due to sea waves in a storm episode are 4632 and 1322 kN / m, respectively.

To determine the forces to be applied, the Goda method was employed (Goda 2000). Figure 12 indicates the areas where the forces were applied. The static force equivalent to the maximum dynamic force estimated for docking (red arrow) is applied as a surface force distributed into two finite elements in the horizontal direction. The equivalent static force from sea waves (blue arrow) is applied as a normal distributed force in 13 finite elements at the right side of the superstructure. Uplift forces were also considered at the base of the superstructure. The maximum displacements are not achieved simultaneously in all the parts of the structure.

Table 6 X and Y maximum displacements

Type of Force	Maximum Displacement X (m)	Maximum Displacement Y (m)
Cruise Ship	0.0311 (Point A)	0.0086 (Point A)
Sea Waves	0.0200 (Point B)	0.0297 (Point B)
Joint	0.0165	0.0019

Results

Physical model

Using the initial settings, it was found that the significant incident wave heights were not reached. The waves that entered the channel had a smaller height than the corresponding wave measured in the wave tank. This variation is probably due to the friction created by the wave tank wall. Consequently, it was necessary to increase the input data to reach the expected local incident wave heights.

Tests 1 to 4 were conducted with a reduced number of tetrapods at the top of the armour layer (which is the part most exposed to incident waves). The number of tetrapods was lower than in the prototype, resulting in breaking waves occurring on the superstructure from the low energy dissipation in the tetrapod armour layer. The impact was violent and the wave overtopping caused some instabilities. It was necessary to strengthen the superstructure and replace some tetrapods in the upper armour layer. The next eleven tests included these changes. In these tests the impacts on the superstructure for the 20 year return period were not so strong. Wave energy was largely dissipated in the tetrapod protection layer. The exposed underwater layers, the toe, and tetrapod armour layer showed no changes, and to be stable. Therefore, considering a return period of 20 years, the behavior of the infrastructure as a whole was stable (Fig. 13).

Considering the 50-year return period, although there was already a reinforcement of tetrapods on the top of the armour

layer, the waves began to impact the superstructure, but not on the tetrapods. Wave overtopping became more frequent and more intense, and some small instability of the superstructure was observed. The submerged layers and the toe became more exposed to wave action and suffered progressive settlements, aligning themselves with the exposed underwater armor layer.

The tetrapod armor layer showed more pronounced movements in situations of strong agitation, as well as some landslides, perhaps facilitated by movements in the toe berm, and various instabilities began to become more visible. Therefore, the behavior of the infrastructure became slightly unstable as a whole, particularly in the exposed armour layer and in the toe berm (Fig. 14).

In the 100-year return period tests there was a large variation in the toe berm with upward and downward movements, and in the exposed underwater armor layer that tended toward a structure with no distinctive layers. An evolution towards a uniform slope in these two layers was clearly shown, losing the clear separation between them, and an extended “S” form was observed. The tetrapod armour layer also tended to prolong the downward direction, clearly being filled with lighter material and moved in both directions (Fig. 15).

Wave overtopping became very frequent and the energy and turbulence impacting on the superstructure increased, resulting in damage. It was found that the behavior of the infrastructure was quite unstable for a return period of 100 years, with frequent overtopping, movement of armour layers and displacement of armour units and toe erosion.

Numerical model

With the numerical model the following conditions were simulated:

- i) the cruise ship docking conditions in the protected part of the breakwater;
- ii) the equivalent forces due to wave action;
- iii) the above two actions together.

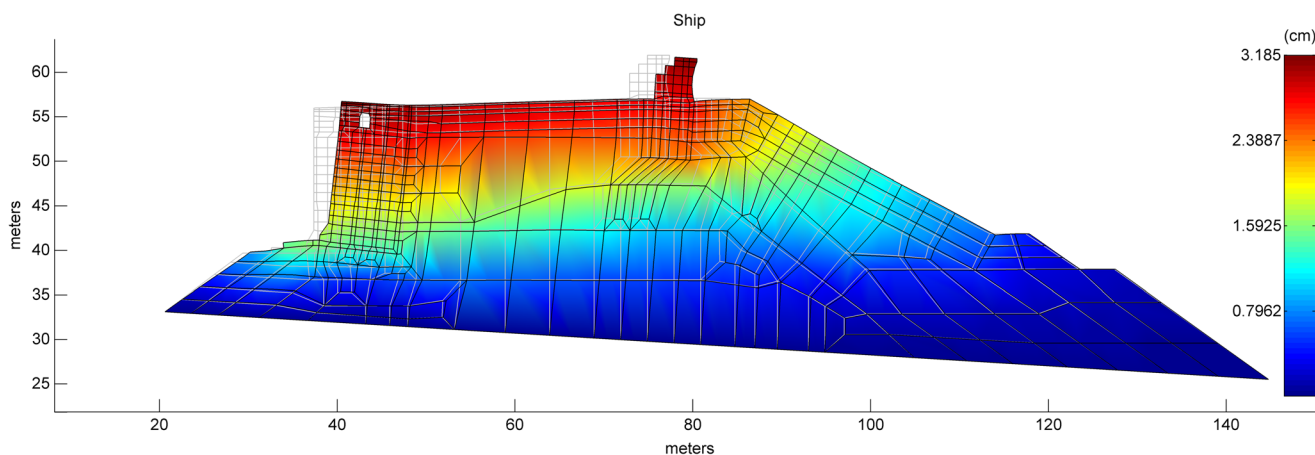


Fig. 16 Displacements considering the docking of cruise ships maximum applied forces

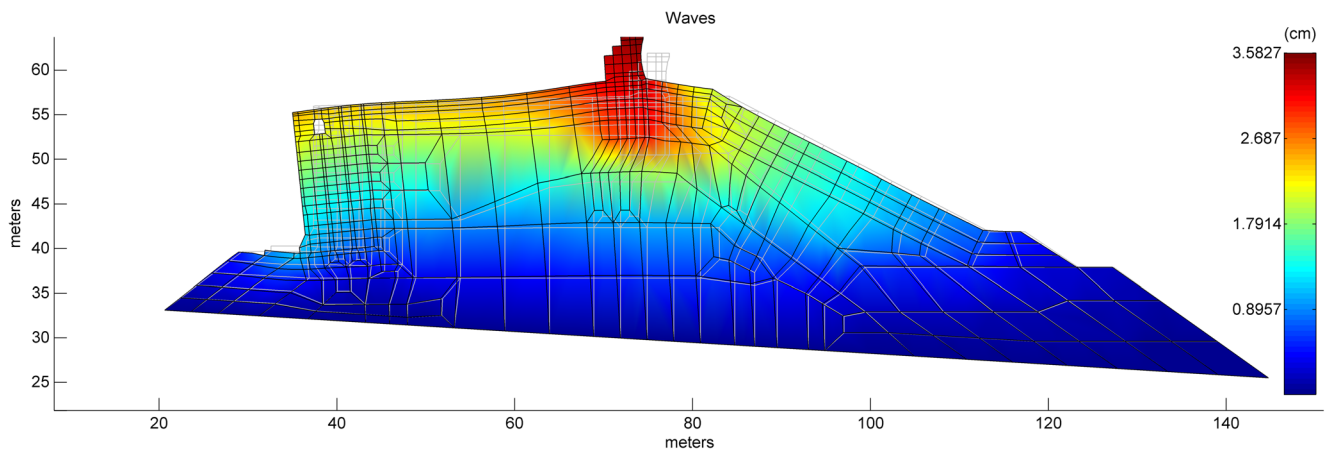


Fig. 17 Displacements considering sea wave maximum applied forces

The possibility of instability occurring in the foundation of the submerged layers, particularly in the armour layer and in the broken tetrapods, the toe and underlayer below the superstructure (this was assumed earlier) were also taken into account. The maximum displacements obtained for the two cases are presented in Table 6.

Figures 16, 17 and 18 show the displacement for each of the cases considered: i) ii) and iii).

The values presented in Table 6 and also in Figs. 16, 17 and 18 consider the maximum values applied to each type of force described, verifying a natural displacement balance when the forces are applied together (the displacements are lower in both axes).

Discussion and conclusions

In the physical model of the Funchal Port breakwater the hypothetical causes for weaknesses observed in the superstructure were verified. Through the analysis of incident waves on the tetrapods and in the cap it was possible to verify the effects in the prototype when violent storms occur. Return periods of 20, 50 and 100 years were considered. It was found that the

outermost layers of the breakwater foundations tend toward a slope reduction, involving slippage in the upper layers and thus putting the stability of the infrastructure at risk. The main limitations are related to some uncertainty in the transposition of results and conclusions obtained from the model due to scale and laboratory effects. Based on the results obtained it can be concluded that it is important to implement regular underwater video monitoring campaigns, to design a 3D model of the breakwater “head” and also to investigate the magnitude of the docking/undocking processes. The 3D modeling was not conducted due to a lack of funding.

Considering the numerical model, the main natural frequencies were computed. Consistency between the values of natural frequencies determined experimentally (the measured accelerations series were analyzed using the Fourier analysis technique in order to obtain the amplitude spectra whose peaks indicate natural breakwater frequencies) and the numerical results was verified—the numerical model parameters were adjusted with high accuracy, informed by the evolution of the breakwater elements obtained with the physical model. The expected displacement values corresponding to extreme sea waves in storm events and extreme docking/undocking

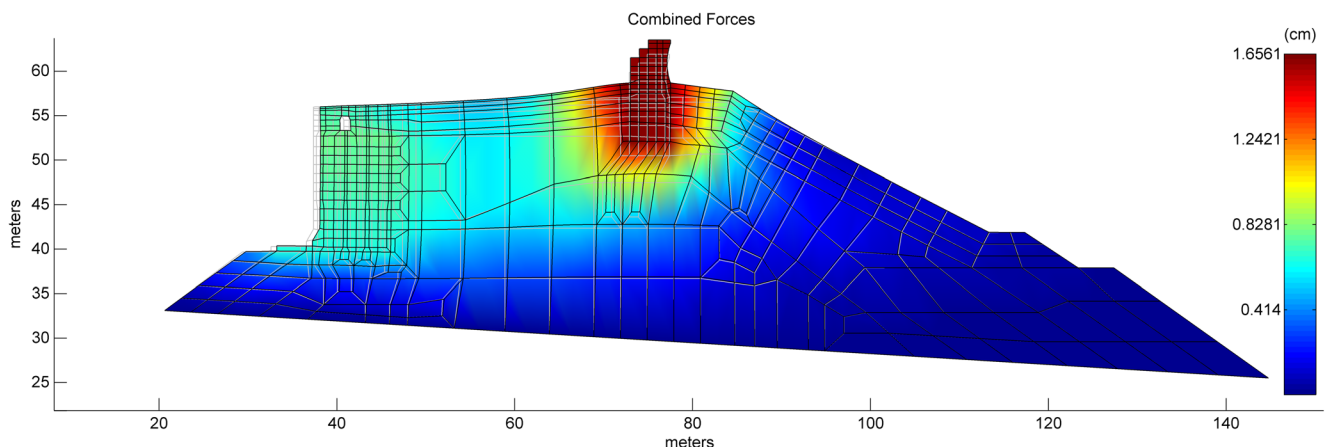


Fig. 18 Displacements considering the cruise ship docking and sea wave maximum forces applied together

processes were calculated. These displacement values could explain the visually identified damage in the breakwater.

The sustainable development and conservation of coastal resources involves insights from various studies and approaches. Several works which incorporate different monitoring methodologies (e.g. in situ data, satellite data, GNSS data) are already reported in the literature (e.g. Teodoro and Veloso-Gomes 2007; Teodoro et al. 2007, 2014; Taveira-Pinto 2008; Teodoro and Gonçalves 2012; das Neves et al. 2015; Williams 2015). Coastal infrastructures such as ports are an important issue in coastal management. Ports are a vital component of coastal zones and their impact on the broader environment can be extensive during infrastructure development and ongoing operations. Therefore, the establishment of maintenance programs in order to analyze the stability of port infrastructures is critical. The development, operation and maintenance of coastal infrastructures have the potential to impact the coastal/marine environment through the potential contamination of water quality, sediment quality and disruption of coastal processes, habitats and public recreation areas. Management and support of port infrastructures is a core element of coastal management. This study presents some innovative contributions in which we highlight the development of an integrated system using GNSS, accelerometers, physical and numerical models. Innovative integrated analyses applied to the Funchal Port breakwater were also presented.

The multifunctional integrated system for monitoring and controlling the safety of port infrastructure is divided into two distinct parts:

- i) Structural evaluation;
- ii) Configuration evaluation.

The structural evaluation took elements relating to in situ campaigns, and the configuration evaluation used information concerning the conditions of development of inner and outer breakwater layers according to input data in physical and numerical models. The data from in situ campaigns were used for the configuration of the physical and numerical models through the natural frequencies of vibration and the displacements found. The data obtained from in situ campaigns were used for calibration of the numerical model; the data obtained through physical modeling were used in the calibration of the numerical model and the results obtained in numerical modeling thus proved the weaknesses identified in situ.

The integrated methodology developed in this work can be applied to other breakwaters, taking into account their specific characteristics.

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