ADAPTATION STRATEGIES IN MARITIME PORTS: THE CASE OF LEIXÕES PORT

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Aos meus Pais e Irmãos

Cada dia sabemos mais e entendemos menos.

Albert Einstein

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Resumo

As alterações climáticas têm vindo a condicionar a tomada de decisões em vários campos que vão desde o urbanismo à política. O mesmo se passa com a gestão portuária, estando este caso ainda mais agravado com a previsível subida do nível médio da água do mar e com a expectativa do aumento tanto do número como da intensidade das tempestades, as quais contribuem para a imprevisibilidade e agravamento das condições marítimas.

Com a presente dissertação pretende-se avaliar a influência das alterações climáticas na segurança portuária, utilizando-se metodologia implementada noutros projetos que avaliam o risco e possíveis adaptações a galgamentos de estruturas portuárias.

Esta metodologia caracteriza a agitação gerada localmente, quer pela ação do vento, quer pela passagem de navios, e a agitação marítima que chega ao porto, analisando consequentemente o seu impacto nas estruturas marítimas e os valores de caudal médio galgado por unidade de comprimento. Termina com a análise e discussão de possíveis medidas que possam mitigar tais riscos.

O trabalho é iniciado com uma revisão bibliográfica onde são explorados outros casos em que já foram tomadas medidas para minimizar a ação marítima e a subida do nível médio das águas do mar, passando por um processo de caracterização do caso de estudo, o Porto de Leixões, bem como as condições marítimas a que este está sujeito. De forma a introduzir o risco de galgamento no porto foi feita uma breve análise às estruturas de defesa exteriores através da ferramenta XGB-Overtopping. Para o cálculo da agitação marítima incidente nas estruturas interiores do porto foram aplicados os métodos de Hasselman e Hochenstein para as ondas geradas localmente (ondas de vento e de esteira, respetivamente) e para a avaliação da agitação exterior incidente foi considerada informação fornecida pelo Laboratório Nacional de Engenharia Civil (LNEC). Posteriormente, é utilizada uma ferramenta que, através das fórmulas empíricas de cálculo de galgamento, analisa para diferentes subidas do nível médio da água do mar o caudal médio galgado e que cota de coroamento seria necessária para impor limites a esses galgamentos.

Finalmente, são apresentadas medidas que podem ser aplicadas no caso de estudo, nas zonas de maior risco. Para tal as docas e terminais do porto foram divididas em 30 zonas e analisadas individualmente.

Esta avaliação torna possível uma primeira análise ao risco futuro a que o porto poderá estar submetido identificando as estruturas mais sensíveis e desenvolvendo medidas de adaptação que serão eficazes na gestão e defesa do porto contra as alterações climáticas.

PALAVRAS-CHAVE: Alterações Climáticas, Galgamento, Medidas de adaptação, Porto de Leixões, Subida do nível médio da água do mar.

Abstract

Climate change has been conditioning decision making in various fields, ranging from urban planning to politics. The same is true for port management, and in this case is further aggravated by the predicted sea level rise and the expected increase in both the number and intensity of storms that further drive the unpredictability of maritime conditions.

This dissertation aims to evaluate the influence of climate change on port safety, using a methodology implemented in other projects to assess the risks and possible adaptation measures to the overtopping of port structures.

This methodology characterizes waves generated locally, either by the action of wind or by the passage of vessels, and the penetration of offshore waves in the port, consequently analyzing its incidence on maritime structures and the values of the mean overtopping discharge value per unit of length. It ends with the analysis and discussion of possible measures that can mitigate such risks.

The Dissertation begins with a literature review where other cases are explored in which some measures have already been taken to minimize sea action and the sea level rise effects, passing through a process of characterization of the case study, the port of Leixões, as well as the maritime conditions to which it is exposed. In order to introduce the risk of overtopping in the port, a brief analysis of the outer defense structures was made using the XGB-Overtopping tool. The Hasselman and Hochenstein methods for the locally generated waves (wind waves and wake waves respectively) were applied to evaluate the wave conditions at the inner structures of the port. To evaluate the outer waves incident on the port information provided by the National Laboratory for Civil Engineering (LNEC) was considered. Subsequently, a tool is used, which, through empirical formulas for calculation of overtopping, analyzes the mean overtopping discharge for different sea level rise values and which crest elevation would be necessary to counteract such overtopping.

Finally, measures are presented that can be applied in the case study at the highest risk areas. For this purpose, the docks and terminals of the port were divided into 30 zones and analyzed individually.

This assessment makes it possible to analyze the future risk that the port may be subject to, identifying the most sensitive structures and developing adaptation measures that will be effective in managing and defending the port against climate change.

KEYWORDS: Climate Change, Overtopping, Adaptation measures, Leixões port, Sea level rise.

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SYMBOLS, ACRONYMS AND ABBREVIATIONS

APDL - Administração dos Portos do Douro, Leixões e Viana do Castelo S.A.

EWS - Early Warning System

GHG - Greenhouse gases

SLR - Sea Level Rise

SSP - Shared socioeconomical pathways

A_c - Cross sectional area

α_b - Angle between the berm and the horizontal

α_d - Angle that the slope immediately below the berm makes with the horizontal

α_u - Angle that the slope immediately above the berm makes with the horizontal

β - Angle between the direction of the incident wave and the normal to the structure

B - Width of the structure berm

B_t - Width of the structure toe berm

F - Fetch

g - Acceleration of gravity

G_c - Width of the crowning structure

γ_f - Parameter of roughness and permeability of the structure

H_s - Significant wave height

$H_{máx}$ - Maximum wave height

H_{m0} - Spectral significant wave height

$H_{m0,\ Toe}$ - Spectral significative wave height at the tow of the structure

H_i - Wave height of waves generated by vessels

H - Wave height at a certain point

H_0 - Offshore wave height

h - Water depth at the structure toe

h_b - Depth of the structure berm

h_t - Water depth of the toe berm

K_d - Coefficient of agitation

q - Mean overtopping discharge

R_c - Distance between the mean water level and the crest of the crown wall

T_m - Mean wave period

$T_{m-1.0\ Toe}$ - Spectral wave mean period at the toe of the structure, defined by m_{-1}/m_0

T_p - Peak wave period

θ_m - Mean wave direction

U - Wind speed

U_A - Pressure applied by the wind on the water surface

U_{10} - Wind speed at a height of 10 meters

U - Wind speed

U_A - Pressure applied by the wind on the water surface

1
INTRODUCTION

1.1. INTRODUCTION

Ports and waterways are the core of the system that provides supplies over water. They are part of a chain where their functions and operations influence the rest of the transport system as a whole.

Ports are usually defined as a complex of infrastructures that enables vessels to load and unload their cargo so it can be transferred from one mean of transportation to another. Ports may contain different cargo-specific terminals and facilities that allow the handling and storage of certain cargo. Apart from these terminals, a port complex usually also includes fuel supply facilities, repair services, customs, etc.

Although ports and breakwaters are designed to be exposed to climate risks and to extreme maritime events, it is not always possible to accomplish that since there is an economic limit where the investment can only guarantee a certain safety level. Due to the unpredictability of the ocean and the fact that breakwaters are not designed to withstand every possible loading value but instead a specific design wave, damages or failures in these structures are always a possibility. Climate change is expected to aggravate the exposure to climate risks, with more storms, bigger swells, and increased sea levels. The possibility of overtopping in ports is therefore expected to increase through this century. Without adaptation, ports will be even more vulnerable to storms and extremes of climate that will likely be more frequent and will jeopardize every aspect of the port, safety (when the deterioration, breaking or collapse of the structure becomes imminent), functionality (the failures that reduce or condition the use of the port and can have an impact on the useful life of the structure), and operability (when the exploitation of the port area is reduced/suspended).

Although overtopping on Port of Leixões main outer breakwaters has been extensively studied, the consequences of climate change on the interior area are yet to be analyzed. In this sense, in order to understand how these changes will affect the inner structures of the port, the methodology followed in this study will start by briefly analyzing the outer breakwaters through the tool XGB-Overtopping. Next, the factors that will influence the agitation within the harbor will be explained, *e.g.*, which physical effects will have consequences for the study, such as wind and vessel waves, and the agitation created by offshore wave penetration. These are obviously very complex effects that for the purpose of the study will be simplified so that they can be studied together.

After this initial definition of the physical components that will be involved in the final study, it will be necessary to prepare the inputs and apply them to an overtopping calculation tool. This tool is a MATLAB® program that compiles the empirical formulas, for calculating the overtopping in terminals and quays, provided by the guidelines in the EurOtop manual. This MATLAB® tool was used in a similar study in the port of Zebrugge (Belgium) where the purpose was also to analyze the effects of sea level rise inside the harbour basin along the outer port quay-walls (Bolle *et al*., 2018).

Finally, the results are divided into 4 sections that represent the mean overtopping discharge limits allowed for the analysis of the terminals and the required crest elevation to counteract these overflows. The results obtained in each section are separated by different values of sea level rise and are presented through graphs and maps that represent areas of greatest risk and / or areas most affected by the agitation inside the port.

With this being said, once Leixões stands out as a place of great interest that always needs to be evaluated and due to the worsening of climate change and consequently the sea level rise, it is important to study and understand the impact and what changes can be made to minimize these effects on infrastructure, goods and port safety.

1.2. OBJECTIVES

This dissertation aims to apply a methodology to define the risk level to climate change in the Port of Leixões focusing on rising sea water levels and extreme storm events and to evaluate possible adaptation strategies. Specific objectives of the dissertation include:

(1) Literature review on the topic;

(2) Defining possible strategies for adapting to sea-level rise in the Port of Leixões;

(3) Preliminary design of these strategies for different water levels selecting specific case studies to be investigated, *e.g*., increasing terminal crest elevation, adding storm walls;

(4) Assessment of the effectiveness of such strategies taking into account aspects such as adaptability, impact on port operability and acceptance;

(5) Cost-benefit analysis of adaptation alternatives.

1.3. STRUCTURE

Following this introductory chapter, the document is divided into 4 core chapters and conclusions and future developments.

Chapter 2 focuses on examples of adaptation strategies already applied in ports worldwide and from literature.

Chapter 3 presents an in-depth overview of the Port of Leixões, including a characterization of its outer breakwaters, quays and terminals as well as environmental conditions. The methodology used in the dissertation is presented in chapter 4. In chapter 5 the simulations are presented, and results are discussed.

Finally, conclusions and future developments are presented in chapter 6

2
CLIMATE CHANGE ADAPTATION IN MARITIME PORTS

As the climate has been changing at an alarming rate, the infrastructures most exposed to nature become extremely vulnerable and the question arises: can they endure these changes? In this case, being a maritime port, it is subject to the various possible events that can be exponentially worsened by climate change effects that likely involves the aggravation, both in quantity and severity, of storms correlated to the size of the waves that hit the breakwaters of the port. Making matters worse, as sea level rises, the quay, the terminals and breakwater crest levels initially designed for a given storm surge can eventually be "out of date" and it is this possibility that motivates this study. What measures can be taken? At what sea level rise will it be necessary to take measures in ports? With the current rate of sea level rise, in what year will it be necessary to take these measures? These important questions are the main motivation for the present dissertation.

2.1. Climate Change Scenarios and Sea Level Rise

According to the new global estimates, the ocean is rising around 3.4 mm/yr (IPCC, 2019), however, this varies considering the location where it is measured. The sea level rise and climate changes are directly correlated with green-house-gas (GHG) emissions. Since GHG emissions can vary throughout time so can the predictions of future sea level rise. In 2021, the IPCC presented its sixth assessment report where new inputs were considered, referred as SSP (Shared Socioeconomic Pathways) scenarios, it includes factors such as population, economic growth, urbanization, education, and technological development (IPCC, 2021). This model relies on the fact that these factors lead to different future emissions and climate outcomes. In that manner, five different pathways were designed to describe plausible futures as shown on Table 1.

These 5 different narratives have different ranges regarding sea level rise, not only there are five SSP's but also they vary the level of confidence and plausibility of each possibility.

Medium confidence scenarios refer to a higher probability of occurrence and a more realistic situation, which corresponds to a lower sea level rise. On the other hand, low confidence scenarios although improbable are also important to pay attention to. On Figure 1 it is possible to compare medium and low confidence scenarios on both ends (SSP1 and SSP5).

Table 1 - Characterization of different socioeconomical pathways (IPCC, 2021).

SSP	Socioeconomical Development	Challenges
1	Sustainability-focused growth and equality	Low challenges to mitigation and adaptation
2	Trends follow the current pattern	Medium challenges to mitigation and adaptation
3	Fragmented World (concerning nationalism, security and conflicts)	High challenges to mitigation and adaptation
4	Increasing Inequality	Low challenges to mitigation and high challenges to adaptation
5	Unconstrained growth economically and in energy usage	High challenges to mitigation and low challenge to adaptation

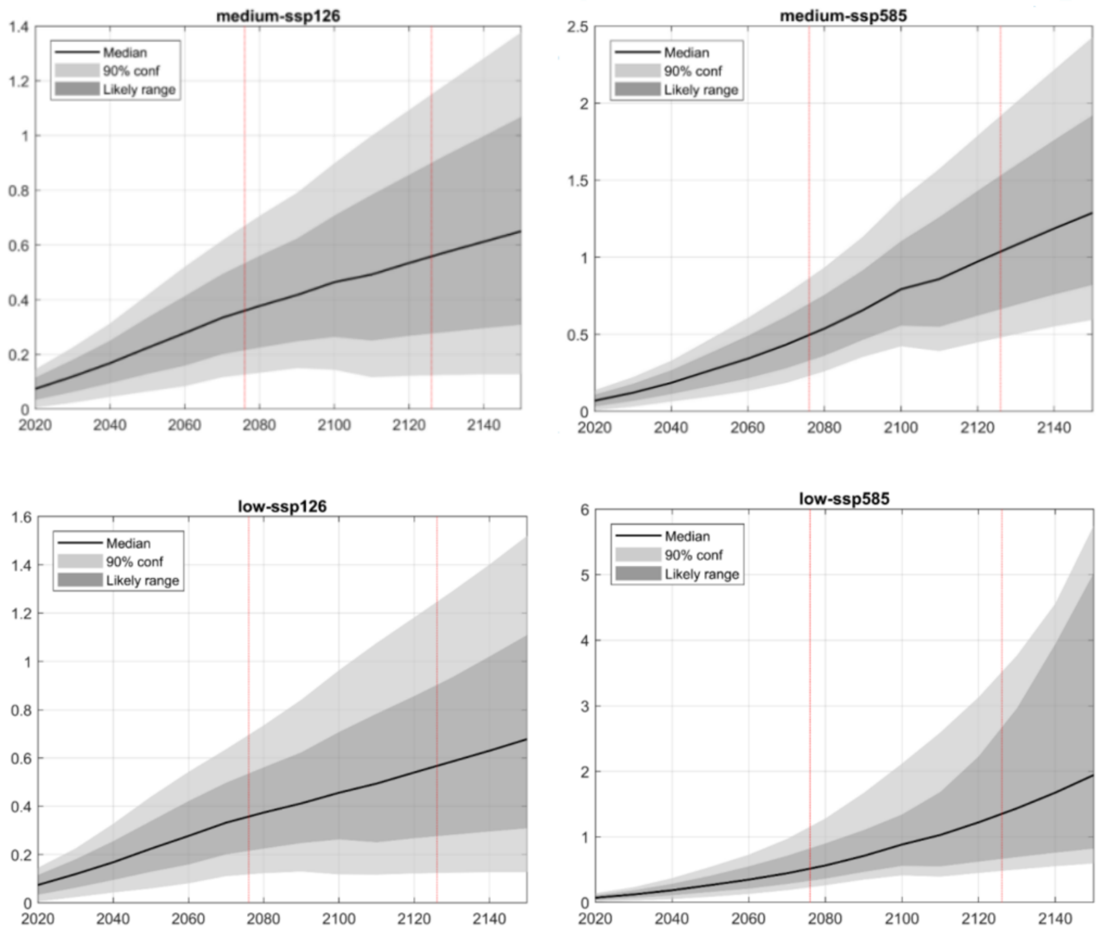


Figure 1 - Medium confidence scenarios for SSP 1 and 5 (IPCC, 2021).

Usually, when designing, the medium confidence SSP5-8.5 and median estimate is used. It is important to bear in mind that the SSP model focuses as much on possible socioeconomical development, as concentration levels of GHG. So, when it comes to the rising sea level, the worst-case scenario socioeconomically also corresponds to intensive use of fossil fuels and therefore higher greenhouse gas emissions.

Global sea level rise varies greatly at regional to local scales therefore is important to consider existing local projections rather than global climate projections. For instance, on the Portuguese coast it is expected that throughout the years of 2025, 2050 and 2100 the sea level rises 23 cm, 44 cm and 1.15m respectively (Antunes *et al.*, 2017).

2.2. Adaptation Measures

When talking about adaptation measures in Ports, it is necessary to bear in mind that their effectiveness varies from one port and area to another; what is a successful action in one Port may be a liability in another with slightly different characteristics. In the end, these adaptations are in fact investments that can provide the Port with better operability which is associated with bigger costs. Mitigation measures should start at the early stages when considering new plans within a port system, where port designers can feature climate change into their plans so to prepare for an uncertain future (Becker *et al.*, 2013). In the end, the costs, and benefits of an "adaptable terminal" design must be weighed against the construction of a stronger but less adaptable design (Headland *et al.*, 2011).

First of all, these adaptation measures go hand in hand with the component's ports have to deal with, weather and maritime threats, exposure and vulnerability, *i.e.*, which physical conditions may threat and harm the port structures, what elements are more exposed to these threats (for example, people, goods, houses…) and how vulnerable they are, as in, how impactful can the damages be, both economically and how hard it will be to recover from it. The last one is very important to be taken into account since it is mostly defined by a country's ability of recovery. A developed and resilient country with means of recuperation can be subjected to some difficulties but will have more options and the ability to turn it around and recover.

With this being said, there are measures that can mitigate these threats. These measures can fall into 3 categories of adaptation strategies: protection, accommodation, and retreat, and each one can alleviate differently the threatening components. Protection is focused on reducing the impact of maritime conditions threats, the accommodation will prevent the exposure and vulnerability effects and retreating will reduce maritime threats and decrease exposure. Each one of them is linked with various options that have obviously different purposes, costs, and effectiveness. Hard coastal defenses and retreating should be considered as a last resource since ports can apply many low-cost intervention measures that can reduce climate risks and build a port's resilience before they resort to hard engineering works (Becker *et al.*, 2013). Main adaptation steps include storm defenses and elevation to compensate for projected sea levels (Christodoulou *et al.*, 2019).

The construction of breakwaters, seawalls, and other elements (associated with the protection phase of defense against the ocean), although being an effective method, they are also an expensive one since it involves the improvement of existing structures or the construction of new ones and can cause environmental problems, e.g., coastal erosion and habitat degradation (Airoldi *et al.*, 2005). As an estimate, the cost of a sea wall and a bulkhead construction varies between three quarters of a million and two million Euros per kilometer, and the construction cost of dikes or levees to protect against 1 m water level rise varies between less than a million and four million Euros per kilometer (Hippe, 2015).

To accommodate, in this case, refers to the creation of, for example, an Early Warning System (EWS) a flood hazard map, or evacuation plan in order to give a warning to the port, making it possible to move goods, vehicles and people out of the hazardous area. It is certainly a cheap option (compared with the other) but can become unsuitable if the number of times the warning system is activated makes it unbearable to maintain a minimum operability degree. Another solution inserted in the accommodation category is elevation, and it entails raising the port above the predesigned height, accommodating the difference in heights between the water level and the port structure (Becker *et al*., 2018).

The previously mentioned EWSs are a key component of maritime defense, reducing the risk of possible disasters. This measure offers different options of usage since it is used alongside other protective methods for example evacuation plans or the deployment of temporary barriers (Bolle *et al*., 2018). Essentially, an early warning system offers the possibility to take action and prepare to avoid or reduce the disaster risk, as an example, an early warning system can enhance the success of evacuation plans and even lead to the early deployment of emergency responders that will further protect the area (Becker *et al*., 2013).

The retreat category its often used as a last resource, since it refers to the setback of the coast, that is, it involves the realignment of the coast, the port would retreat from its original position moving inwards into the land. This implies that surrounding areas must be vacant and available for relocation which usually is not the case. Most ports would not be able to apply this measure and continue to occupy their current location or be abandoned (Becker *et al*., 2018).

After this explanation on what type of options exist, on the following Figure 2 it is possible to understand the impact that different climate parameters have on the port as a system as an all.

Figure 2 shows which climate parameters affect each area of the port and the impacts that can possibly occur. For example, on the loading/unloading area almost all climate parameters can influence it and operability can become complicated and get delayed. Some impacts are less concerning than others and can be solved somewhat easily whereas the others involve more costly and demanding solutions.

With all of the information already presented, it is important to emphasize how the operability of a port has can influence the decision making and that these adaptation measures are often looked at as investments. The knowledge of what types of measures exist and their advantages and disadvantages although important, have no meaning if the case isn't evaluated accordingly. For this, it is necessary to bear in mind the different possible scenarios that can occur. Exploring a range of climate change scenarios during the planning and design processes allows an asset or operation's sensitivity and tolerance to possible future climates to be tested (PTGCC, 2022). This is important so decisions that focus in to a single predetermined or 'presumed' climate change scenario can be averted, and hence avoid potentially significant investment risks.

It is established that using more scenarios will be beneficial to make a final decision on the problem, but it also brings up another question, how many and which scenarios are going to be necessary. This is determined by the exposure and vulnerability of the structure, asset, or operation the more susceptible it is to weather or climate-related damage or disruption, or the greater the magnitude of investment involved, and the longer the intended design or operational life, the more important it becomes to explore a full range of scenarios (PTGCC, 2022).

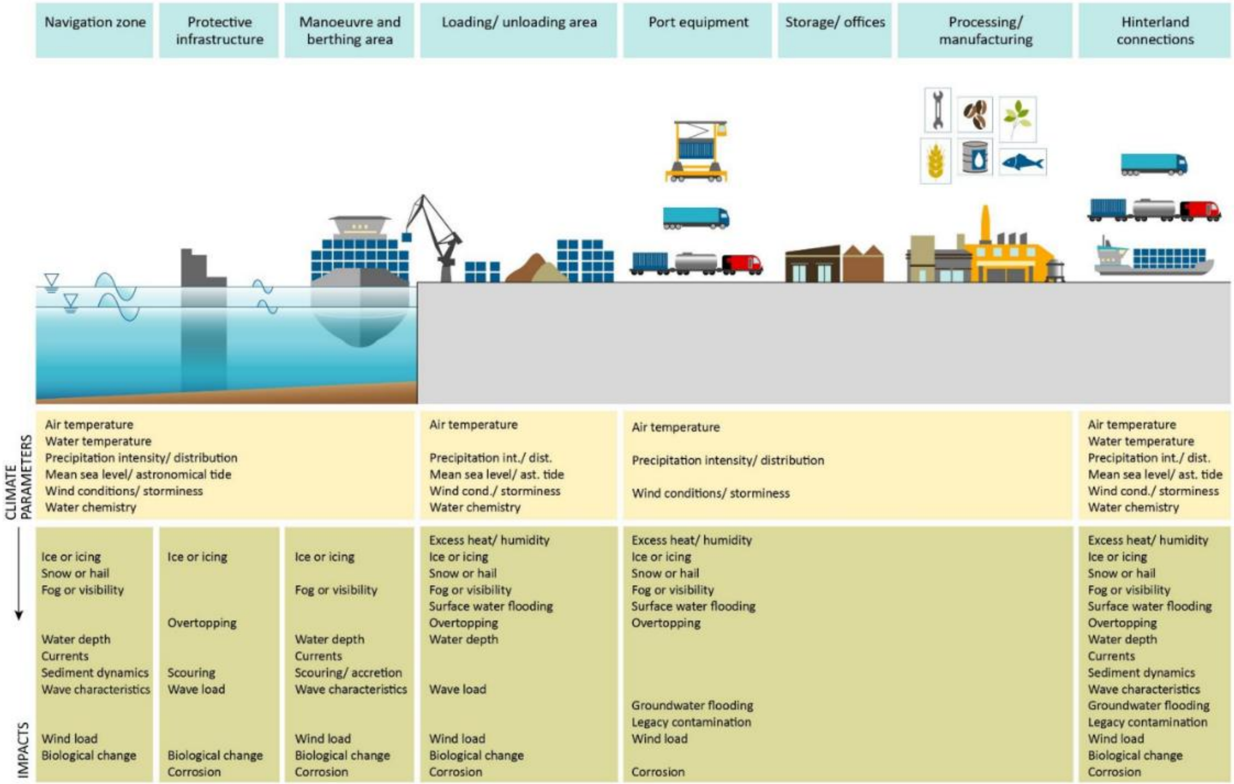


Figure 2 - Relationship between climate parameters and impacts on port infrastructure/operations (PIANC WG 178, 2020).

This fact implies that if the investment is made as a long-term objective, then extra attention must be paid to the low chance but highly impactful scenarios and how they are defined by those designing or investing in such asset. Indeed, IPCC (2019) suggests that stakeholders with a low risk tolerance (for example, making long-term investments in critical infrastructure) consider the possibility of sea level rise above the likely range scenario.

2.3. International Examples

These mentioned measures have been already applied in some cases where the rising sea waters and climate change are already starting to have negative effects and begin to impact the social economy of a port, city and even country.

One of the most famous cases of "fight against the ocean" is in one of the most avant-garde countries in port and coastal engineering, the Netherlands. The project known as "Delta Project" was started in 1958 in order to counter the catastrophic floods of 1953 and was completed in 1986. However, at that time, the rising sea levels related to climate change were an irrelevant or almost non-existent problem, and this project was just another way to safeguard the coastline of this country, but with the worsening of global warming, the government was forced to resort again to an investment in coastal protection (Menteth, 2006). This is a special case, in which the country depended on these measures, the investment was almost unlimited, and the important thing was the perseverance and functioning of these structures for a long time.

The MOSE project was designed to protect the city of Venice, which is known for its annually flooding disasters. It was precipitated by the great flood of 1966, and it can protect the venetian lagoon against projected climate induced sea level of at least 60 cm (Menteth, 2006). Although it was a high-profile engineering project, it ended up not being as efficient as expected and turned out to be an unsuccessful solution.

Another high-profile example of flood protection is located in London where the Thames barrier protects the British capital from surges. It consists of a retractable barrier system and is operable since 1982. Even more interesting for this study is the management plan created in 2002 in order to complement the barrier defense. Known as the Thames Estuary 2100, it consists of a strategic flood risk management plan and sets out the recommendations and actions that are needed to manage flood risk through this century. During the development of the plan, it was investigated the present flood risk as well as how it can change in the future and the different possibilities to manage and adapt to those changes (British Environment Agency, 2012) .

In Belgium, the city of Nieuwpoort is building a new storm surge barrier to protect the Belgium coast and hinterland from heavy storm surges. The barrier will be completed in 2025 and will be able to cope with water levels up to 8 m high. This movable barrier is an important measure for the Belgium country and the important commercial Nieuwpoort port and it is only possible since the port entrance is composed by a canal, which provides the perfect conditions for a barrier of this kind. The construction of this structure will cost 58 million euros (Agency MDK website).

In Texas, United States, after the devastation caused by Hurricane Ike several proposals were presented to protect the Galveston Bay Area region and one of them is to create a coastal spine projected to be 160 km long, which is already a challenge itself, but to make matters worse it is also necessary to build a 2.8-kilometer-long storm surge barrier to prevent hurricane surges from penetrating into the Galveston Bay. This concept certainly is a challenge, where it needs to balance the sufficient water exchange between the Gulf of Mexico and the Galveston Bay and reduce the effect of storm surges with a probability of occurrence of once in ten thousand years. Not only there are many problems that this structural wonder will face, but also it will have estimated costs ranging from $2.7 to $4.0 billion (de Vries, 2014).

These are special cases, but a number of seaports have already taken measures towards adaptation and offer valuable lessons on the development of best practices. For instance, in The Netherlands the Port of Rotterdam has developed a strategy that focuses on flood safety, better accessibility for ships and their passengers, building, urban water system and city climate (Becker *et al*., 2013), transforming the city of Rotterdam and making it “fully” resilient to climate change impacts by 2025 (Rotterdam Climate Initiative 2013). In fact, the Rotterdam protection measures are of the highest level globally, consisting of different storm surge barriers two of which are the largest in the world making it one of safest port cities in the world (Christodoulou *et al*., 2019).

The Port of San Diego (California) led a multi-stakeholder effort, The Climate Mitigation and Adaptation Plan, with the surrounding communities that share responsibility for emergency response, critical utilities protection, and stormwater drainage (Port of San Diego, 2013; Messner and Moran, 2013). The assessment methodology included a risk evaluation framework that considered likelihood and consequences of impacts and prioritized according to risk classification (very high, high, medium, low, and N/A or no benefit). The International Finance Corporation and the Port of Muelles el Bosque (Cartagena, Colombia) elaborated a climate risk study for the port and quantitatively assessed financial, and environmental and social impacts that are projected to result from the changing climate (Stenek *et al.,* 2011). The study analyzed projected changes in sea level rise, storm surge height, precipitation, temperature, and wind patterns, and direct and indirect effects of these on port assets, operations inside and outside of the port, surrounding environment and communities, and on the trade of the goods transported through the port. Based on the conclusions of the study, the company announced plans for USD 30 million adaptation investments in two ports (Portafolio, 2011), of which USD 12 million have already been invested (Stenek *et al.*, 2011).

Finally, in Australia, the National Climate Change Adaptation Research Facility and RMIT University have developed a basis for studying the risk, vulnerability, and resilience of the country's ports (McEvoy *et al*., 2013), which highlights that seaport adaptation to climate change involves the development of practices in a range of areas. These include technology and engineering (e.g. cranes that operate safely under stronger winds, raising of connecting roadways/rail lines to respond to flooding); design and maintenance (e.g. accommodation of projected precipitation changes into future building designs); planning (e.g. planning of resilient logistic hubs in partnership with supply chain logistics infrastructure providers/local authorities); insurance (e.g. quantitative determination of unavoidable climate risks for appropriate insurance); and systems management (e.g. introduction of climate change considerations into environmental, logistics and risk/emergency management systems). Other studies have suggested specific practical steps, based on modeling of the wind regime inside a port, such as the reduction of wind loads through the building of permanent wind walls or transient wind protection by stacking containers to heights of up to 13–15 m, which can decrease wind velocity for Ro-Ro vessels up to 25 % and wind loads up to 30% (Paulauskas *et al*., 2009). These studies and initiatives demonstrate integrated and collaborative approaches to climate change responses for ports and port regions. However, much more remains to be done and different approaches will have to be devised to accommodate for the variety of conditions facing ports at a global level.

Table 2 provides a summary of different measures, their advantages, disadvantages, and costs.

Table 2 - Summary table of the type of measures and their characteristics.

Category	Measure Type	Advantages	Disadvantages	Cost
Protection	Breakwaters, dikes	High Protection against different climate change scenarios	Environmental Problems (coastal erosion and habitat degradation)	High
Accommodation	Early Warning System, Elevate	Very versatile options	Can become ineffective	Low/Medium
Retreat	Relocation	Can prevent almost any threat	Highly inconvenient and improbable	Very High

2.4. CONCLUDING REMARKS

Nowadays, designing and taking climate change into account when making decisions is already considered an important pillar of managing and planning strategies even in port reconstruction are designed to be climate-proof. Climate-change assessments are integrated into the port's spatial planning to allow for dealing with uncertainties (Vellinga and De Jong, 2012; Rotterdam Climate Initiative, 2013).

Regardless of which decision and strategy, whether is through elevation, defense or relocation, it will need a wide variety of engineering, design, technological, planning and management options and involve many stakeholders in the process (Becker *et al*., 2018).

Many countries already started implementing measures, some because it was a necessity, others as a precaution, but either way, it is important to start thinking and studying options in case it is necessary to apply some adaptive measures in Leixões port. Although, right now might not seem an emergency to take measures, the northern part of Portugal is the largest exporter of the entire country, most of which is done through the Leixões port. This means that if the port is not prepared, damaging events can occur and have a huge negative impact on the Portuguese economy.

3
CASE STUDY CHARACTERIZATION

3.1. PORT DEVELOPMENT

Located at the mouth of the Leça river, the Leixões port is the most important port on the northern region of Portugal. Consisting of 5 cargo docks is one key economic point of the municipality of Matosinhos and district of Porto, receiving annually almost 3 000 vessels (APDL, 2022).

In the beginning, the city of Porto was only supported by a port on the Douro River, which, despite its privileged location in the epicenter of the city, always brought obstacles to the entry of ships due to both maritime conditions at the river mouth and frequent floods (Cruz, 1993).

In this line of reasoning, the creation of an alternative to a port with many flaws has been present since the mid-19th century. However, it would be difficult for the merchants and inhabitants of Porto to give up a port with so much history and connection to the city. The Leixões port did not immediately replace the port on the Douro River, but started out as a shelter for ships bound for the port of Porto (APDL, 2022).

Only on the 13 of July of 1884 the construction of the port began extending until 1892. First, two breakwaters were built on the North and South sides of the port, but also a submerged breakwater was created (between 1932 and 1940) to extend the north pier by a few hundred meters.

One important fact to take into account is how they were built, since one of the bigger problems was how the heavy granite blocks, that make up the structure, would be moved and placed in their location. To overcome this obstacle, two cranes, called “titãs”, were used and are still standing today as symbols of this historic landmark in Portuguese history and the city of Porto (APDL, 2022).

With the port of Leixões becoming more and more used and preferred by ships that wish to dock in the district of Porto, the shift from a safe haven to a commercial port was imminent. However, the port always had problems with the maritime conditions that often forced long stops and the undocking of ships. Since a rapid solution was needed, the answer was to "conquer the land" and not the sea, so, in 1955 the Commercial Port Enlargement Plan was approved being one of the biggest milestones in the port development, using the river to expand the port complex (APDL, 2022).

After that, the port continued to grow, being the biggest changes the oil tankers terminal construction (during the 60’s), a new and long-awaited fishing port (finished on 1968), the creation of new container terminals between 1974 and the 90’s, the enlargement of the breakwater on the 1980s and finally a new marina built for leisure on the first half of the 1990s. In Figure 3 is represented the evolution of the port with all the mentioned changes.

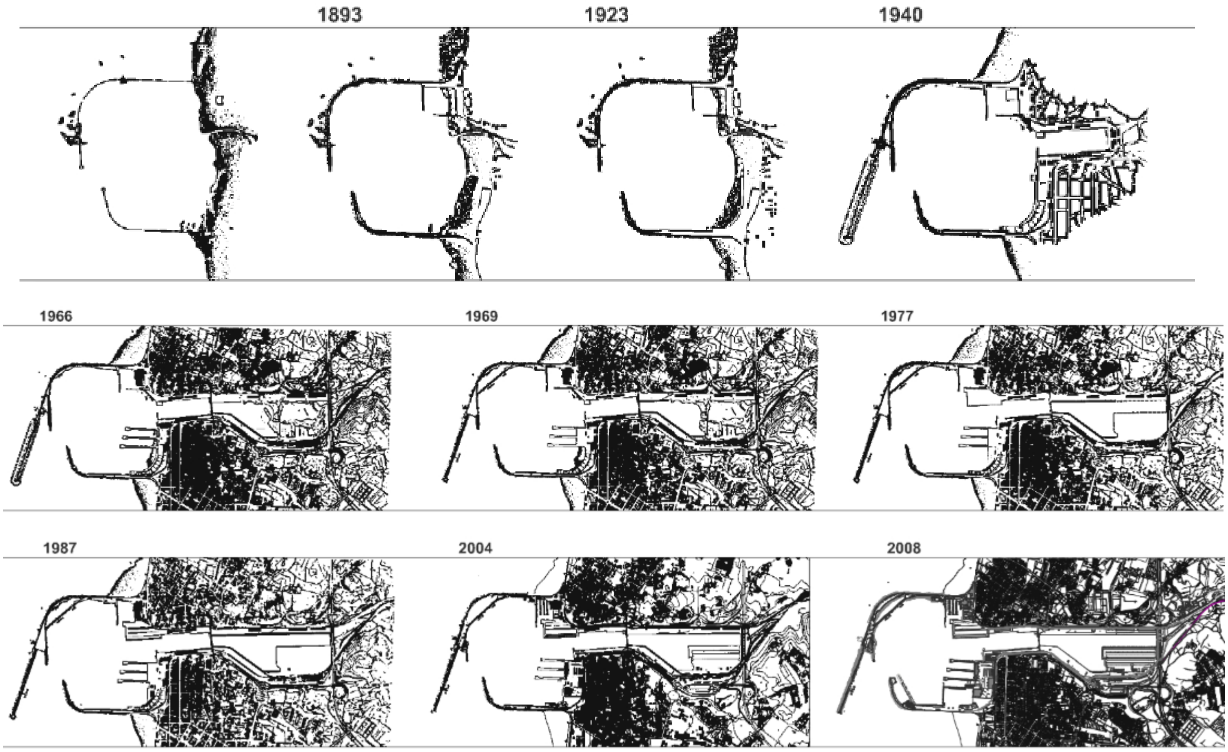


Figure 3 - Evolution of the Leixões Port (APDL, 2012).

More recently, changes have been made on the southern breakwater including the construction of a multipurpose terminal and the construction of the cruise terminal that aims to increase the flow of passengers allowing the docking of larger cruise ships. It is one of the biggest projects since the opening of the port receiving annually around 80 000 passengers (APDL, 2022), Figure 4.

It is also important to note that there are already developments on course, such as the extension of the outer breakwater, the deepening of the water depth throughout the harbor and (more importantly for this study) the enlargement of the multipurpose terminal.

Figure 4 - Leixões port nowadays (APDL, 2013).

3.2. DOCKS AND QUAYS

In this chapter, the different quays and terminals that constitute the port system are characterized, giving a brief description on their purpose and what type of goods they can store, what equipment's are associated with different terminals and other structural characteristics.

Oil tanker terminal – this terminal was built during the 1960s and is essentially used for loading and unloading of oil and supply the Leça refinery. It consists of 3 berths (A, B and C) along the breakwater of about 1.5 km (Figure 5). This terminal is under concession to Petrogal - Petróleos de Portugal, S.A.

Figure 5 - Oil tanker terminal (APDL, 2013).

Berth A is the one with the highest service drought with a water depth of -15 m ZH and a capacity for vessels until 100 000 DWT. It is located at the entrance of the port and is protected from the fierce Atlantic Ocean maritime conditions by a rubble mound breakwater. It mainly deals with oil goods (APDL, 2022).

The berths B and C are located in a more sheltered area than post A since they are located in a more inner area. With a water depth of -10 and -6 m respectively, these berths can dock ships with a capacity of 27 000 DWT on the deepest one and 5 000 DWT on the shallowest one.

Figure 5 shows that berth B and C are located further from the breakwater providing more protection against the ocean. This structure also contains oil pipelines that can transfer oil directly to Leça refinery.

Cruise ships terminal – This terminal is located on the south part of the port and it is the biggest project since the opening of the port. It is an infrastructure of various valences, being not only a terminal for embarkation and disembarkation of passengers but also used as a hub for CIIMAR and other scientific departments. The structures that make up the terminal are the main building and a dockable quay with 340 m in length, Figure 6 (APDL, 2022),

Figure 6 - Cruise Ship Terminal (APDL, 2013).

Multipurpose terminal and fishing port - The multipurpose terminal focuses essentially on the roll on-roll off traffic and has a support area of 8 ha. The fishing port will be transformed into an expansion of the multipurpose terminal making it possible to at least double the amount of cargo that the terminal can withhold. Both the terminal and the fishing port are represented in Figure 7.

Figure 7 - Fishing Port (on top) and Multipurpose terminal (APDL, 2013).

Table 3 and Table 4 summarize the characteristics of the docks, terminals, their storage, the type of structure, crowning elevation among others.

Table 3 - Leixões port docks and equipment's (APDL, 2022).

Docks	**Dockable Quay (m)**	**Water Depth m ZH**	**Crowning Elevation m ZH**	**Equipment's**	**Storage Capacity (m^2)**	**Type**
Dock 1 North	455	-10	+6	1 crane of 45 to 90 t	17 850 (uncovered)	Blockwork gravity structure and diaphragm wall structure
Dock 1 South	520	-10	+6	5 cranes of 6,2 t and 2 cranes of 16 to 40 t	16 663 (uncovered)	Blockwork gravity structure and diaphragm wall structure
Dock 2 North	670	-11	+6	9 cranes of 6.2 t, 2 cranes of 5 to 15 t and 1 crane of 42 to 104 t	34 693 (uncovered)	Open piled structure
Dock 2 South	690	-11	+6	9 6.2-ton cranes, 4 cranes from 12 to 18 t, and 1 crane from 29 to 104 t	53 414 (uncovered)	Underwater concrete
Dock 4 North	400	-12	+6	2 ecological 15-t cranes with anti-pollution system	22 448 (uncovered) and 4 000 (covered)	Diaphragm wall structure with unloading platform

Table 4 - Leixões port terminals (APDL, 2022).

Terminals	**Water Depth m ZH**	**Crowning Elevation m ZH**	**Type**
Berth A	-15	+6.20	Underwater concrete gravity structure
Berth B	-10	+6	Open piled structure
Berth C	-6	+6	Open piled structure
North Container Terminal	-6 and -10	+6	Blockwork gravity structure
Southern Container Terminal (Dock 4 South)	-12	+6	Blockwork gravity structure

3.3. Environmental Conditions

3.3.1. Wave Climate

Since it is intended to study the effect of climate and maritime changes on the port of Leixões coastal structures, it is necessary to analyze the sea conditions such as tides, winds, and swell, so the overtopping can be evaluated. For this, using the data registered in the northern part of the Portuguese coast, it is possible to evaluate the characteristics of the maritime conditions.

The parameters that will be taken in consideration in this analysis will be the significant wave height (H_s), the average of the highest wave heights registered in the period of analysis, the maximum wave height (H_{max}) observed, the mean wave period (T_m), the peak wave period (T_p) and the mean wave direction (θ_m). The data was collected in the span of 17 years, since 2004 until 2020.

Analyzing the data obtained during this period it is possible to see that the significative wave height varies between values of 0.26 and 9.21 m and presents an average value of 2.02 m. In relation to the peak period, it ranges between 3.5 and 18.4 s and has an average value of 9.3 s (Figure 8).

The average wave direction varies between 0º and 360º, where 85% of the values are higher than 270º, showing the predominance of waves coming from the northwest quadrant (Figure 9).

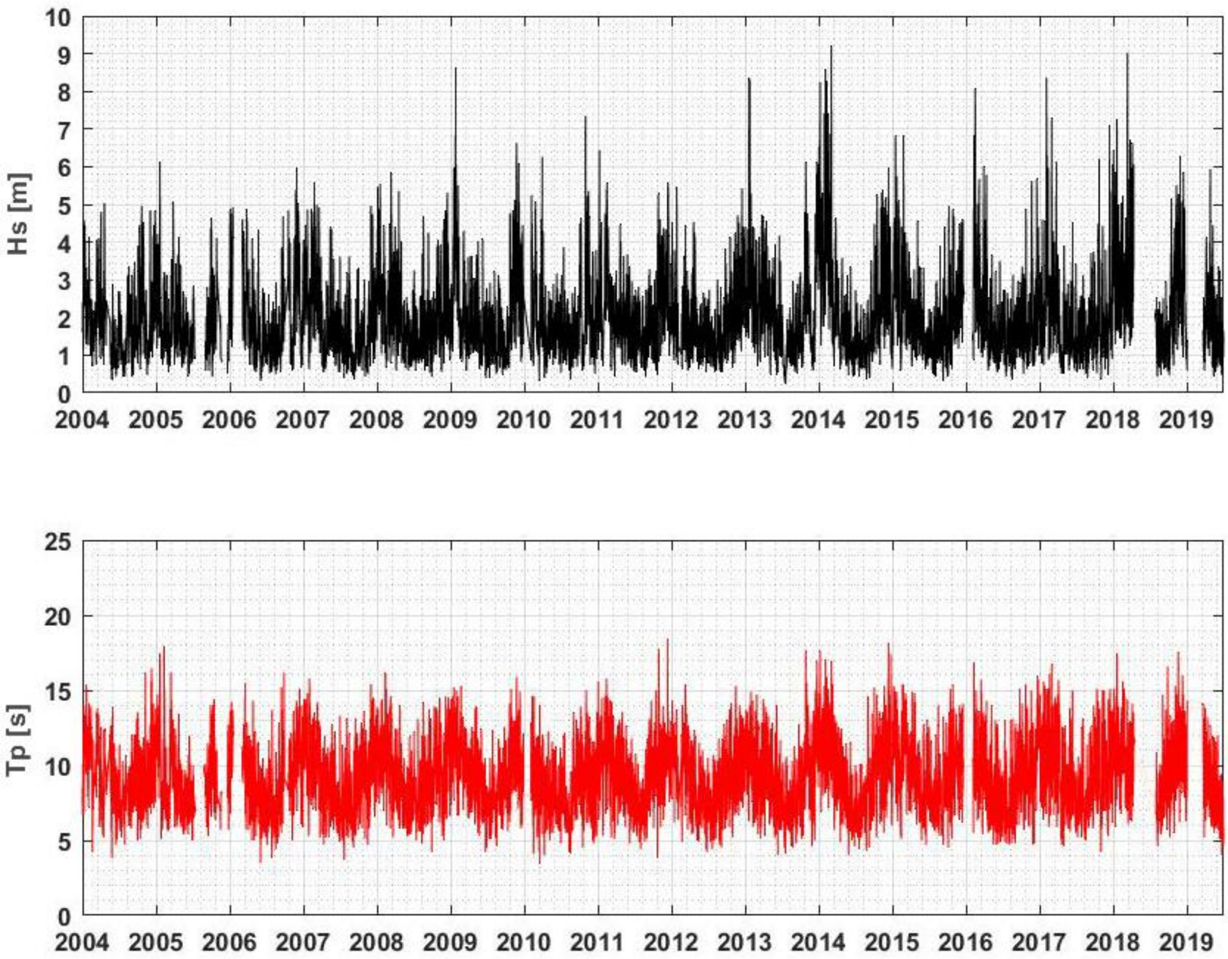


Figure 8 - Time series of significative wave height and peak wave period.

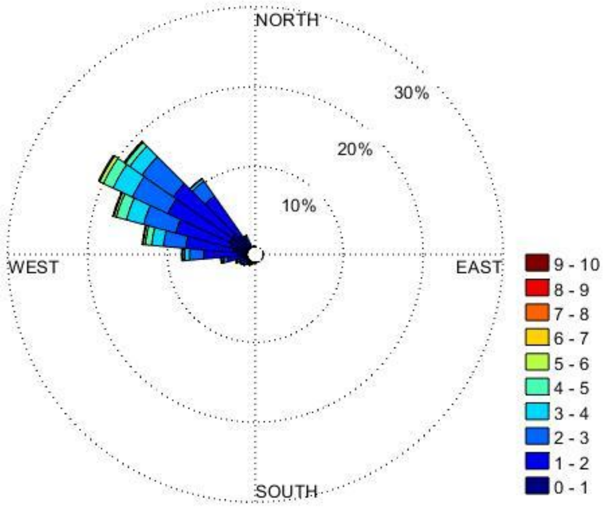


Figure 9 - Rose of wave directions and significative wave heights.

3.3.2. WATER LEVELS

The tide is characterized by two components: astronomical tide and meteorological tide. The first one is easily estimated resorting to harmonical analysis, and it can be obtained through the software T-tide (Pawlowicz *et al*., 2002). The meteorological component varies, as the name suggests, accordingly to meteorological conditions, usually caused by storm surges where there is a sea level rise due to pressure variations or caused by the action of strong winds and, although the wind shear stress is usually small, its effect when integrated over a large body of water can be accentuated immensely (Dalrymple, 1991). The meteorological component is harder to estimate then the astronomical one since it is very unpredictable.

With this being said, the mean sea level can be obtained through data collection or through the prediction of the astronomical tide and then add a rough prediction of the meteorological component (Munk, 1950). In this case, there are data values obtained by the buoy located in the port collected since 1994 until middle 2020, but it is important to note that from the year 1996 until middle 2002 there were no values registered. In Figure 10 it is possible to understand the tidal evolution.

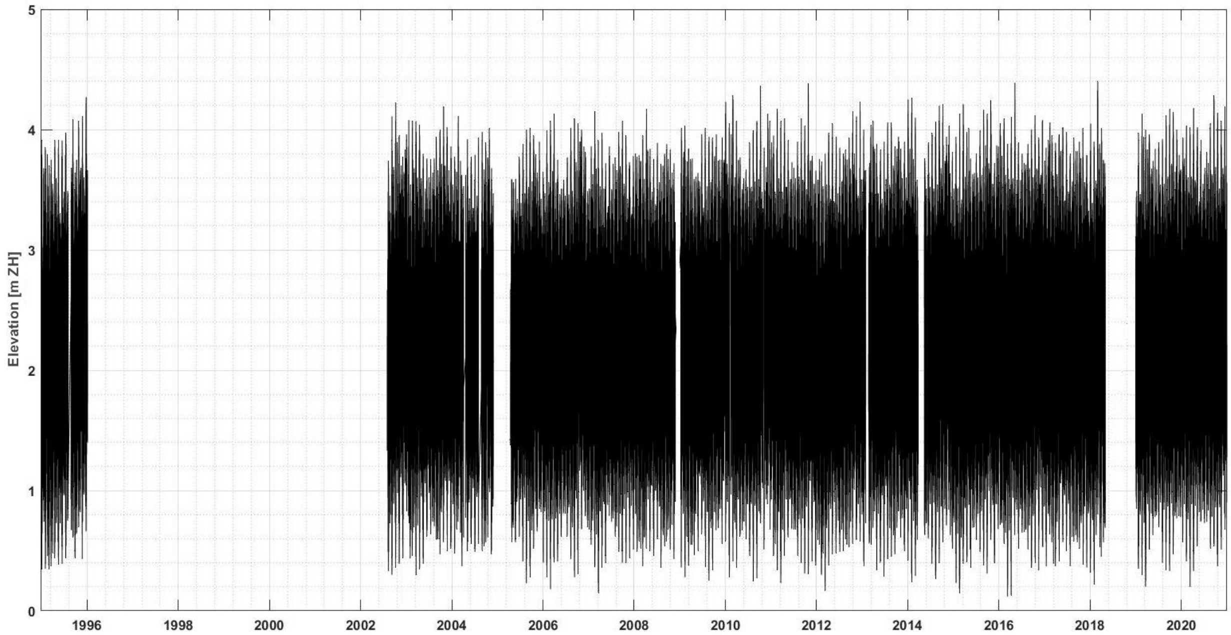


Figure 10 - Tidal levels recorded by Leixões buoy throughout the years.

As it was previously stated, the meteorological component of tidal analysis is quite variable and can't be precisely estimated or calculated, so, in order to have an idea of how it has changed through the years it is necessary to separate the two components. To do this, the T-tide software was used in order to calculate the astronomical component, thus obtaining the 'Predicted Tide'. Then, through the measured data obtain the 'Registered Tide', and by the subtraction between the two, will lead to the meteorological component (Almeida *et al*., 2009).

When calculating it is important to emphasize that the astronomical tide is calculated using the harmonical constants that are used in Portugal, and which can define the accuracy of the estimation of this component (Almeida *et al*., 2009). In this case, five harmonical elements were used which have corresponding amplitudes and wave phases (Table 5), although these are considered the most important components when defining the astronomical tide, it may still be possible to expect some imprecisions in the estimations. The adopted water level during the study was +2.00 m (ZH), which is the reference value for the Leixões port.

Table 5 - Harmonical constants of the main components of the astronomical tide for the Leixões port (Almeida *et al*., 2009).

Astronomical tide components	Amplitude, H (m)	Wave Phase (º)
M2	1.05	74.2
S2	0.36	102.5
K2	0.10	100.3
K1	0.07	59.3
O1	0.06	318

After the subtraction of the two parameters, measured tides and calculated astronomical tides, the values of the meteorological components are obtained and can be analyzed. However, it is important to note that, although there is information for more comparisons, the graphic that is calculated is very hard to analyze because of the high amount of data, so, for better understanding of the difference between astronomical calculated tide and the real registered one, the following figures were obtained corresponding to the years of 2003, 2005, 2017, 2019 and 2020 (Figure 11). These specific years were chosen not only to compare the development of the meteorological tide component throughout the years but also because of the amount of missing data in previous years that could lead to estimation errors.

Although the graphics give a visual representation of the difference between the measured tides and the calculated astronomical tide, it is also important to further analyze the data and obtain more information regarding the full collection of values. For that matter, Table 6 provides the values of the maximum meteorological tide annually since 2002. These values will be used in an analysis of extremes, that will further characterize the meteorological tide for different return periods with confidence intervals of 90%. The method applied was the generalized extreme value (GEV) distribution and analyzing Figure 12 it is possible to conclude that for a return period of 10 years the elevation induced by the meteorological component is around 0.9 m, for a 50 year return period the elevation is 1.3 m and finally a 500 year return period presents a 1.7 m elevation.

Table 6 - Annual maximum value of meteorological tides.

Year	Maximum Annual Value
2002	0.88
2003	0.78
2004	1.10
2005	0.64
2006	0.82
2008	0.86
2009	0.81
2010	0.94
2011	0.90
2012	0.86
2013	0.81
2014	0.84
2015	0.75
2016	0.81
2017	0.62
2018	0.82
2019	0.65
2020	0.61

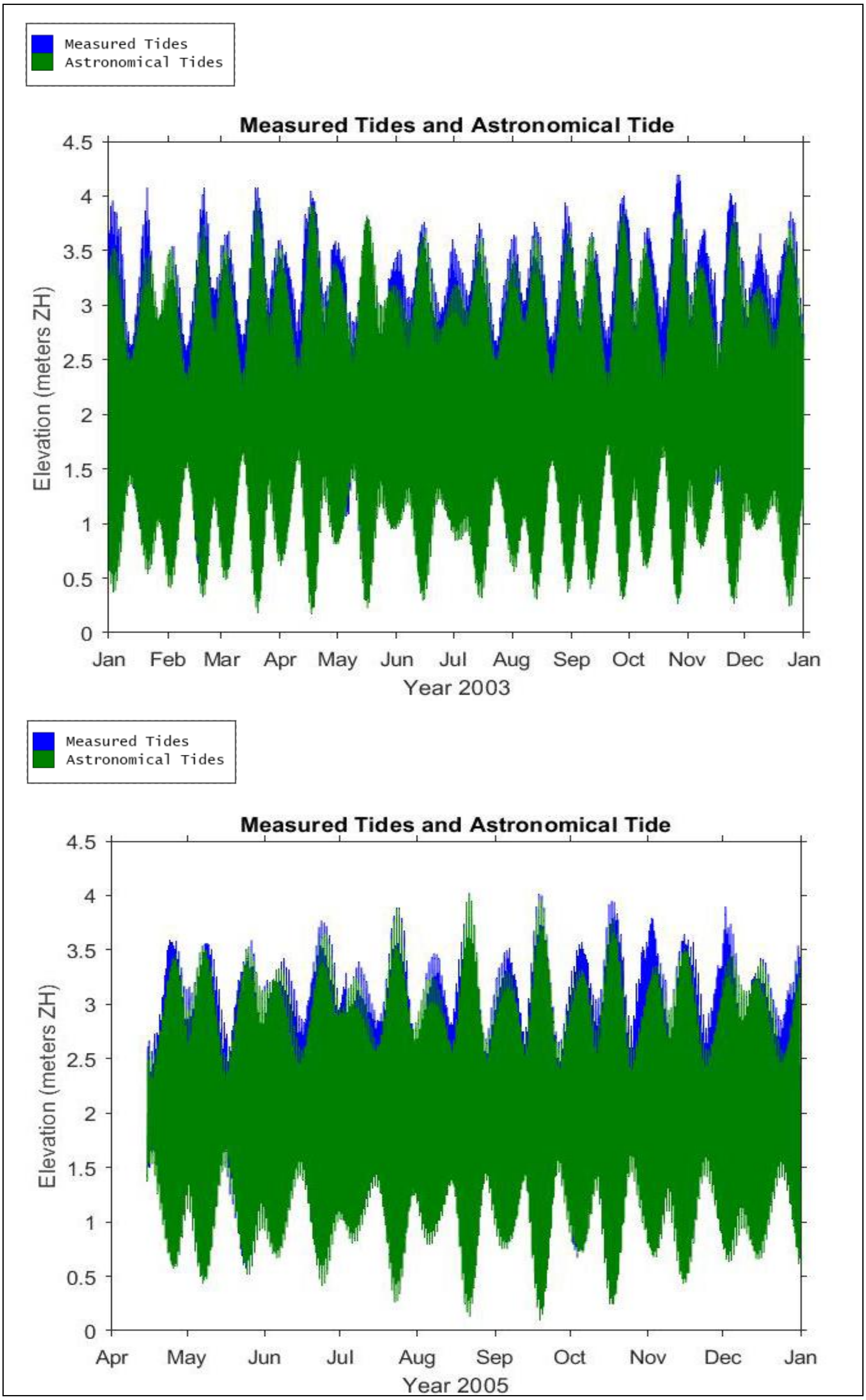


Figure 11 - Comparison between the meteorological and astronomical components in different years

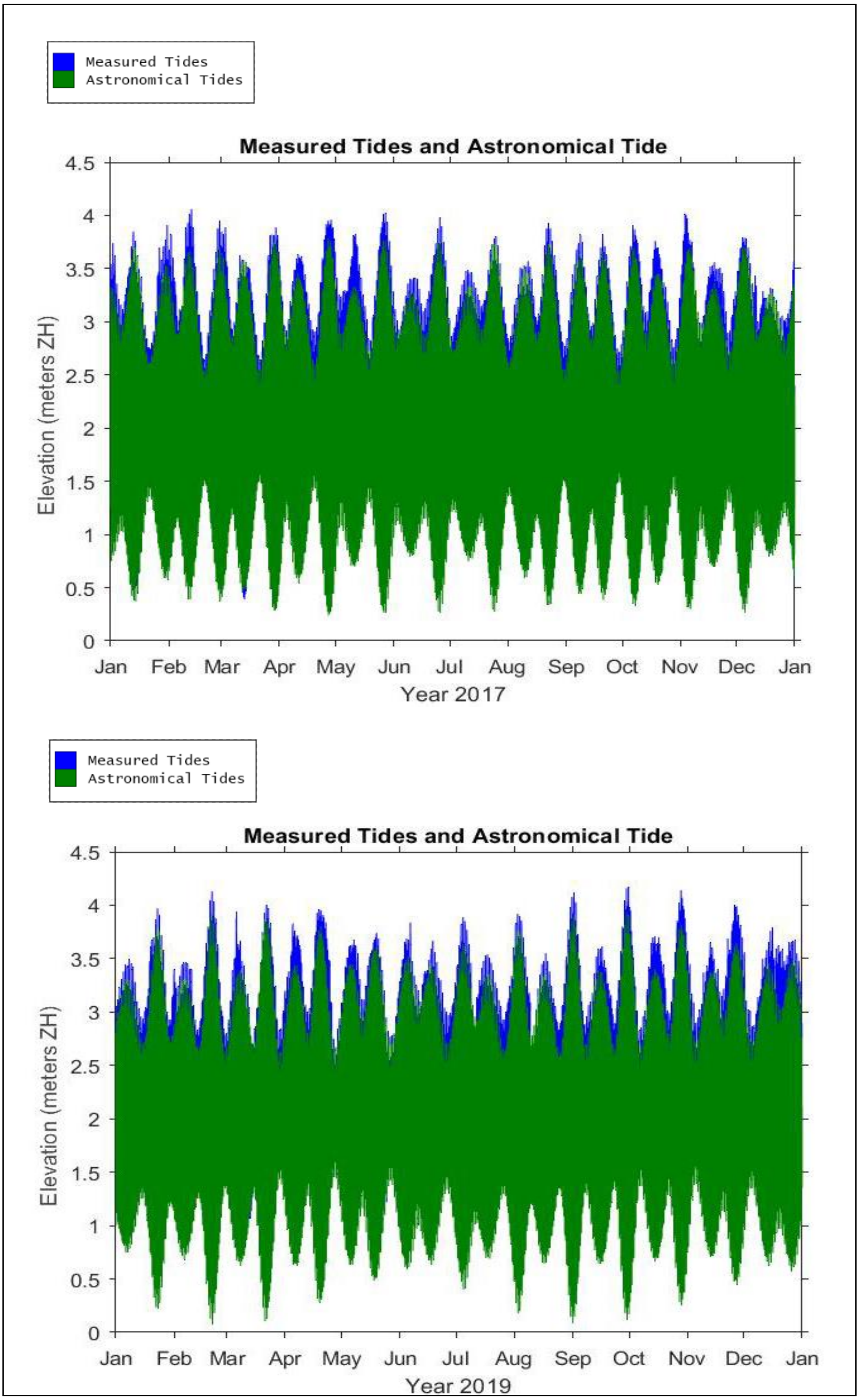


Figure 11 - Comparison between the meteorological and astronomical components in different years (Cont.)

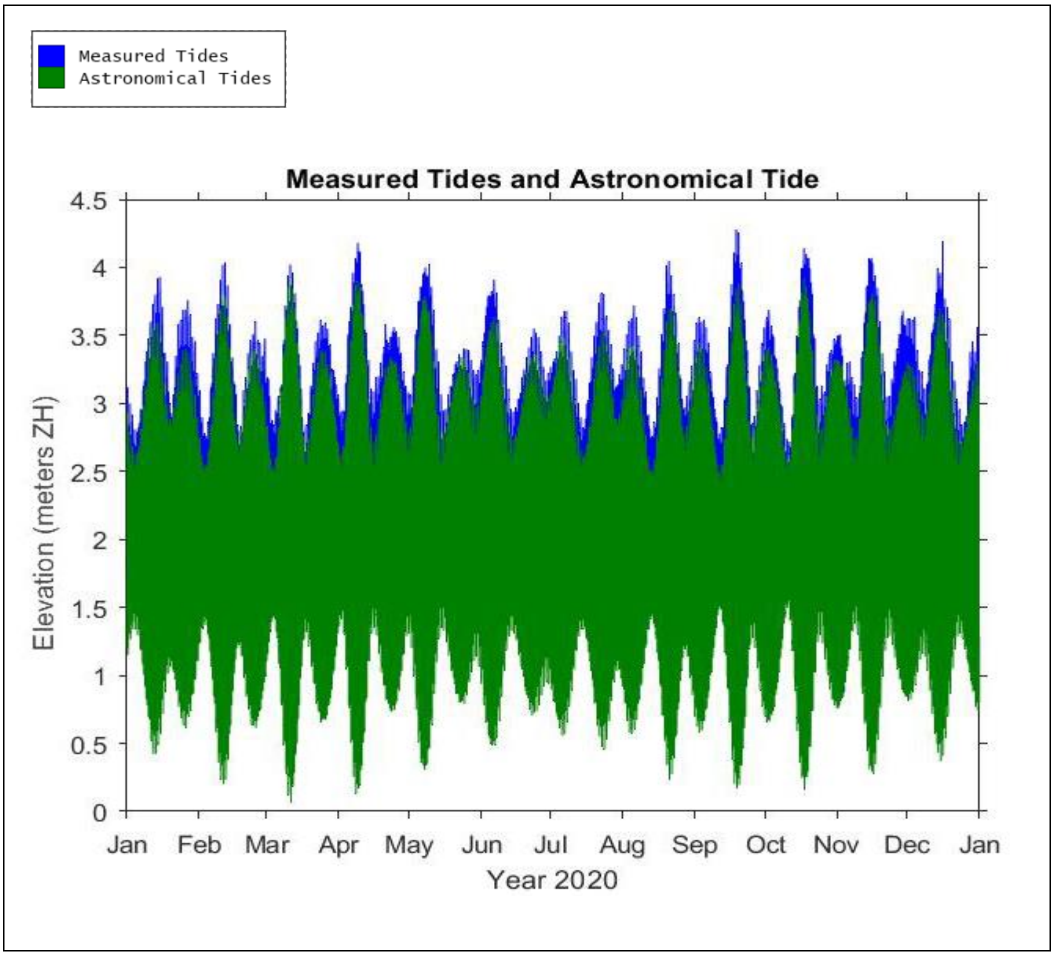


Figure 11 - Comparison between the meteorological and astronomical components in different years.

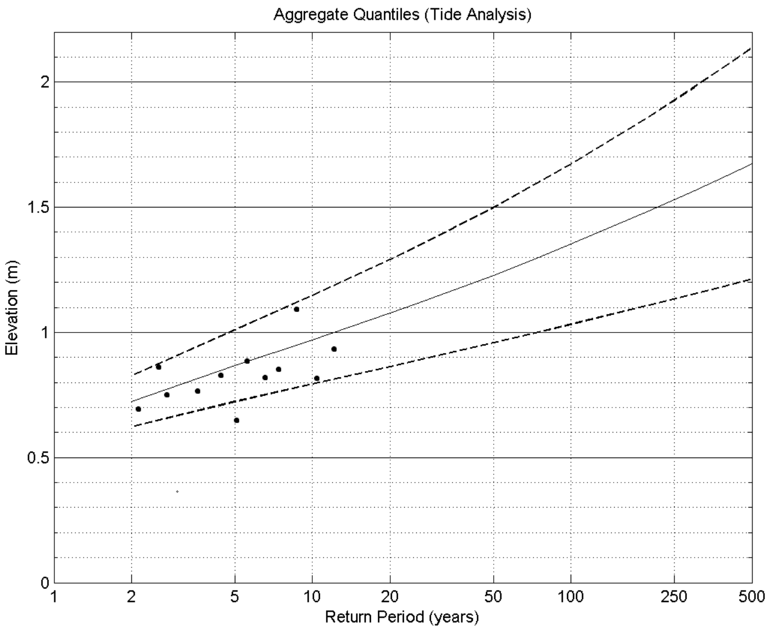


Figure 12 - Extremes analysis of the meteorological tidal component.

4
ANALYTICAL STUDY METHODOLOGY

In this chapter the methodology used to address the problem in study is presented. Since the aim of the study is to assess the impact of extremes of climate and climate change, specifically sea level rise, on the overtopping of the quays and terminals in the port of Leixões, it will obviously be necessary to take into account different variables and different calculation methods for each structure. Waves inside the harbors are a combination of locally generated wind waves, waves generated by the passing of vessels and offshore wave penetration in the port, and in order to obtain results it is important to analyze which effects will be important to consider in the process and which formulations are going to be used (Bolle *et al.*, 2018).

Figure 13 presents a flow chart of the methodology used in this study.

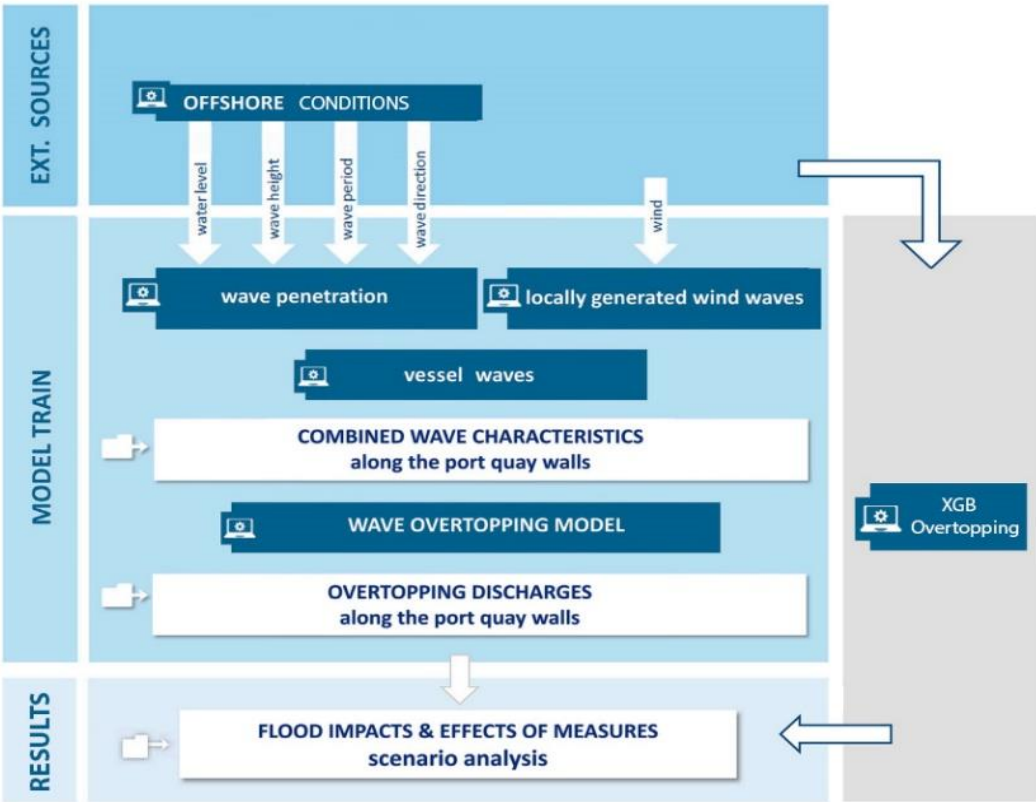


Figure 13 - Flow chart of the methodology.

The methodology focuses on two main structures, the outer breakwaters and the inner terminals and quays. For the outer breakwaters the overtopping is estimated based on the neuronal network tool (XGB-Overtopping) using the offshore conditions data and obtaining results for different scenarios. For the inner terminals and quays, the methodology follows an estimate based on the calculation of input boundary conditions, which are then combined and applied to a wave overtopping model tool that computes these inputs taking into account the different structure characteristics and the interactions between them.

4.1. INPUT BOUNDARY CONDITIONS

4.1.1. LOCALLY GENERATED WIND WAVES

Wind waves generation occurs due to the transfer of energy from the wind to the water, and an increase in wave height and period is observed as the wave travels through the fetch zone (Gomes, 2014).

The effect of the wind acting on a mass of water is correlated with various inputs and there are many different methods to calculate the characteristics of these waves. In this study the method proposed by Hasselman *et al.* (1976) is used. This method is easy to implement and shows good correlation with the expected characteristics of the waves being generated locally by the wind. The method is as indicated in the following equation,

$$\frac{g\ H_{m0}}{U_A^2} = 1.6\ \times\ 10^{-3}\left(\frac{g\ F}{U_A^2}\right)^{\frac{1}{2}} \tag{4.1}$$

where g represents the acceleration due to gravity, H_{m0} the spectral significant wave height, F represents the fetch in meters and U_A the pressure the wind applies on the water surface.

In this case, the fetch was estimated through the measure of the largest length that can originate the most undesired situation. To estimate the most conservative option it was thought that a simple series of measures could overcome the problem. In order to do that, three measurements were executed using the provided port layout that can deliver an accurate fetch value. Figure 14 shows how the fetch varies according to the three measures. The values obtained were approximately 930 m, 1120 m and 2800 m.

Concluding, the worst scenario for the fetch is about 2800 m and it will be a constant in every situation. U_A, can be obtained through the following equations:

$$U_A = 0.71\ U_{10}^{1.23} \tag{4.2}$$

$$U_{10} = U(z)\ \left(\frac{10}{z}\right)^{\frac{1}{7}} \tag{4.3}$$

where U_{10} represents the wind speed at a height of 10 m in relation to the water surface, is a variable used to convert the wind speed (U in m/s) considered at any given height (z in m) in relation to the water surface (Gomes, 2014) and can be calculated applying Resio and Vicent method (1977) whose numerical expression is shown in equation (4.3). To be noted that this equation has an application range between 8 and 12 m, so, the most conservative (8 m) will be used. The parameter U is chosen based on the Beaufort scale, so, it will be separated in 4 categories: windy, strong wind, storm, and hurricane.

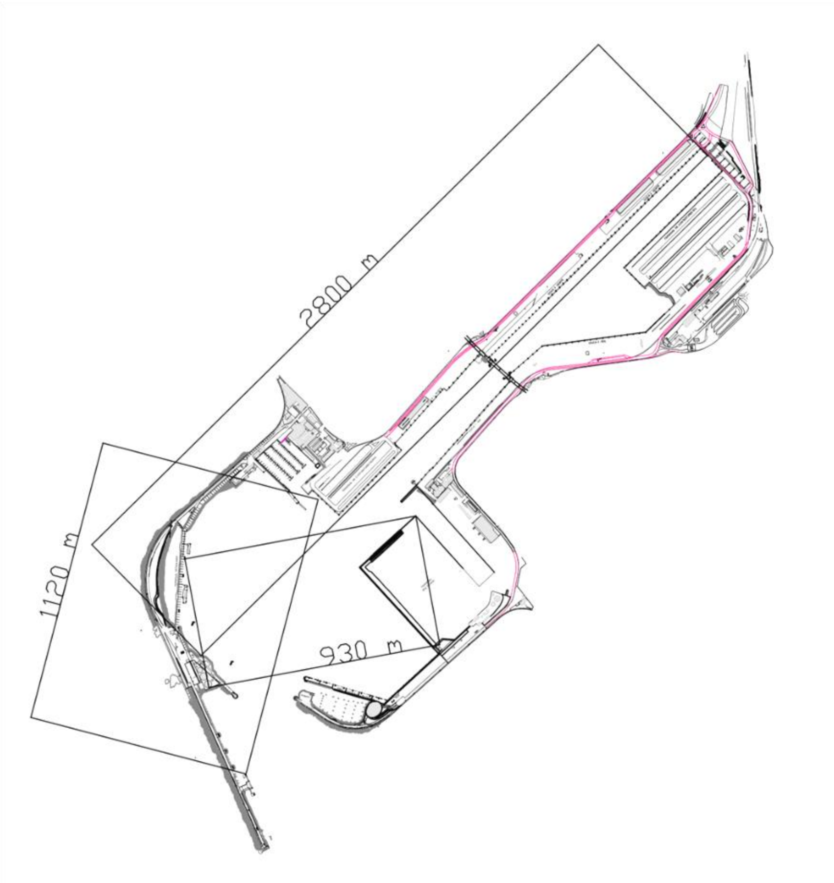


Figure 14 - Fetch measurements at the port's layout (APDL, 2017).

Table 7 presents a summary of derived wind waves and the parameters assumed in the calculation.

Table 7 - Wind waves for each scenario.

Scenario	Fetch (m)	Wind Speed (km/h)	Wave height (cm)
Windy	2800	50	5
Strong Wind	2800	80	10
Storm	2800	110	15
Hurricane	2800	125	18

Following the analysis of the results it is possible to conclude that in the worst-case scenario the wave height from wind waves is 18 cm.

It is important to note that that both the values of the fetch and the wind speed were chosen as being the most unfavorable for the study. However, this combination generates an extremely unlikely scenario, and the environmental conditions would hardly reach these parameters. Nevertheless, attempts were made with the other fetch and wind speed values and the results obtained changed so slightly that it was chosen to assume the highest values for both variables.

4.1.2. Wave agitation inside the port

Wave agitation inside the harbour basin is determined by the amount of wave penetration inside the port. The agitation is a fraction of the offshore wave conditions and depends on the wave height, wave period and wave direction. This fraction may also be influenced by the water level.

The wave agitation was estimated based on a recent numerical study by LNEC (2017), which was done for the purpose of the ongoing works within the Leixões port, namely the extension of the north breakwater, the deepening of the access channel and the creation of a new container's terminal (LNEC, 2017).

The outputs obtained consist of different parameters, namely water level, offshore wave period, offshore wave direction, agitation coefficient H/H_0 (relation between the wave height at the measuring point, H, and the offshore wave height, H_0), wave period and wave direction at the output point.

LNEC's model domain and output points are presented in Figure 15. For the purpose of the present study the points 50 to 77 in Figure 15 are relevant.

Based on the data provided in LNEC's study it is possible to correlate an offshore wave conditions with the conditions inside the port, but it is important to point out that this method is a simplification, and some effects and hydraulic interaction are not taken into account. Such effects can be related to the conditions for which the values of the agitation coefficient (K_d) were determined, which can influence the preciseness of this study. Since the coefficients were extrapolated from a study that involved specific conditions of water level, which will be an important variable throughout the study, and wave direction that influence how the waves penetrate the harbour, it can cause a deviation for this parameter which will impact the results. In addition to this, there are hydraulic interactions that are not applied, such as interactions between the wave and the seabed, the effect of currents, the effect of how wave overtopping on the outer breakwaters can affect the agitation inside the port and other energy dissipation effects.

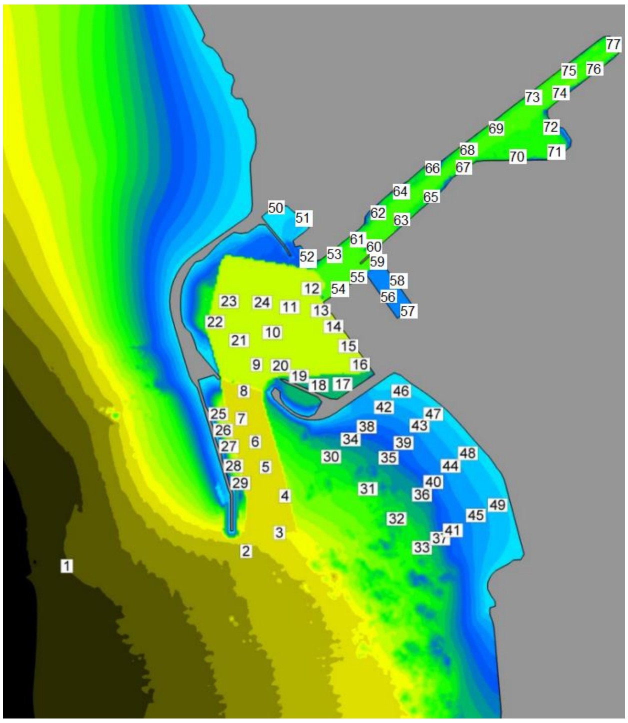


Figure 15 - Model 2 of future Leixões Port layout and output points (based on LNEC's numerical study).

Either way, to calculate the wave height on a certain point, the following equation will be used:

$$H = H_0 \times K_d \tag{4.4}$$

where H corresponds to the wave height at the measuring point, H_0 is the wave height offshore and K_d will assume the value of the agitation coefficient calculated at an output point inside the harbour.

It is important to bear in mind that the offshore waves penetrating inside the port are associated with a specific wave direction. This means that the harshest wave conditions may come from a direction that is sheltered by the outer breakwaters, thereby although higher these may not be as impactful as waves coming from other directions for which the harbour basin is more exposed to. So, not only the offshore wave height is important to consider, but also the direction of the penetrating wave.

With this being said, in this case it is usual to observe that the highest waves come from the northwest/west quadrants. However, these do not impact or influence that much the inner quays and terminals, the ones which do, have a direction coming from the south and southwest quadrants towards which the harbour entrance is oriented to.

For that matter, to obtain the design offshore waves (H_0) an extreme regime analysis was made regarding each direction which will then be applied to the equation (4.4) thus calculating the wave height caused by the penetration of offshore conditions. To calculate the wave height, a generalized extreme value (GEV) distribution was applied and the values obtained were 4 m for southern directions and 7 m for southwestern directions (

Figure *16*) and the wave period for the calculation will be the same as the wave period offshore (15 s). For an analysis purpose, the Table 8 will provide the calculated data for the important points inside the port that will be crucial in the course of the study.

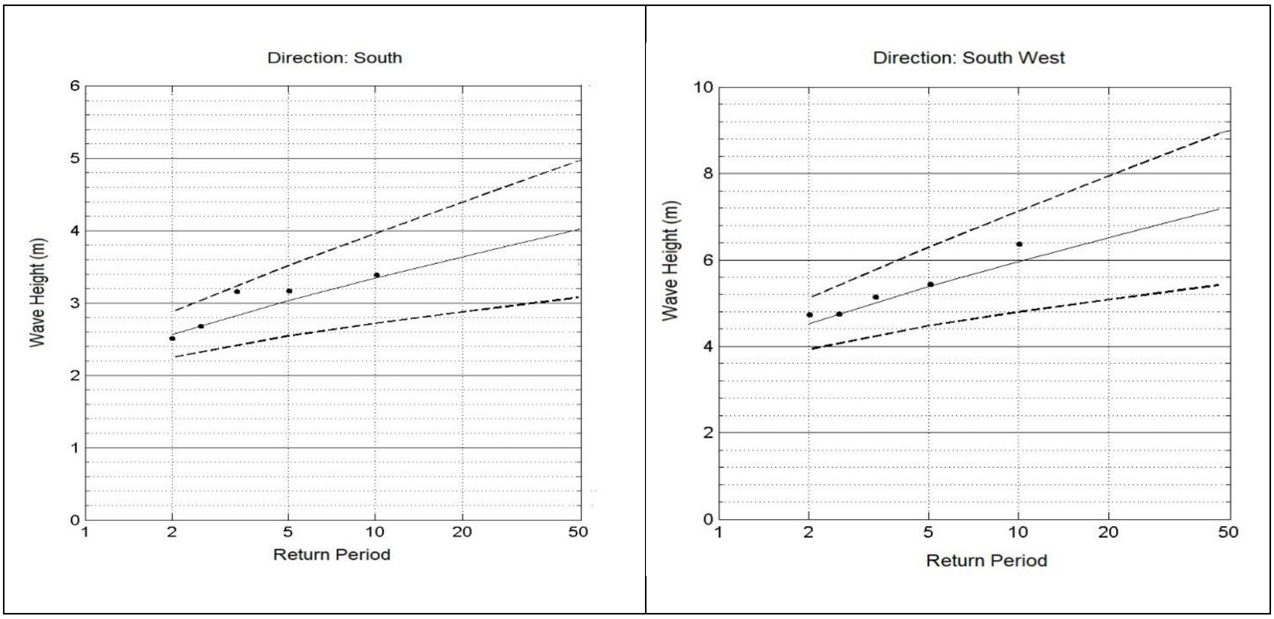


Figure 16 - Extreme analysis for the south and southwest directions.

Table 8 - Wave height generated by the penetration of offshore waves.

Points	Direction	Kd	Wave Height (m)	Wave Period (s)
P52	Southwest	0.39	2.76	15
P53	Southwest	0.31	2.17	16
P62	Southwest	0.19	1.30	18
P64	Southwest	0.16	1.15	16
P66	Southwest	0.23	1.58	18
P68	Southwest	0.26	1.84	16
P69	South	0.13	0.51	5
P75	Southwest	0.08	0.56	18
P77	South	0.10	0.41	16
P76	South	0.10	0.40	16
P74	Southwest	0.09	0.61	18
P72	Southwest	0.12	0.83	18
P71	Southwest	0.24	1.66	16
P70	Southwest	0.14	0.96	18
P67	South	0.21	0.82	5
P65	Southwest	0.16	1.12	16
P63	Southwest	0.32	2.23	16
P60	Southwest	0.20	1.40	16
P59	Southwest	0.15	1.05	17
P58	Southwest	0.12	0.85	17
P57	South	0.07	0.28	12
P56	Southwest	0.15	1.03	17
P54	Southwest	0.21	1.44	17
P14	Southwest	0.20	1.43	17

4.1.3. Vessel Waves

The passing of vessels through a harbour basin (Figure 17 and Figure 18) is capable of creating waves that when superimposed with locally generated wind waves and the wave agitation inside the port may induce an even higher wave height, which can impact quays and terminals operations.

Although there are many methods to estimate vessel waves, each one of them is usually used in different situations and can be more or less precise in that estimation. In the present study, the Hochenstein method (1980) is used. This method has its advantages and disadvantages, on the one hand it does not have any constraints and requires little calculus, because of its generally low estimation precision. Notwithstanding, in the present study, this method is considered appropriate for a preliminary assessment, in which is assumed that the vessel is moving along the channel with a finite section as it happens inside a port and for which there is little information readily available about the type of vessels entering and leaving the port in Leixões.

Figure 17 - Vessel going through Leixões port (APDL, 2013).

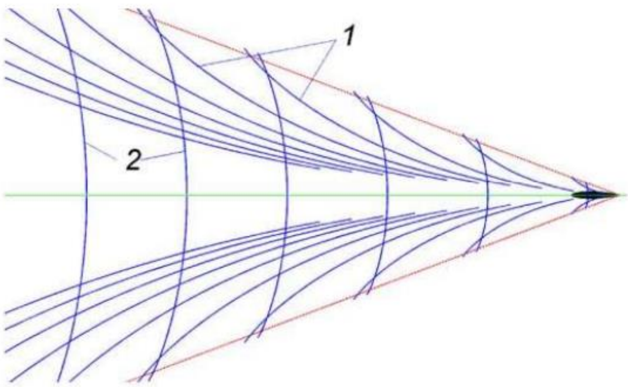


Figure 18 - Divergent waves (1), transverse waves (2) (Adapted from Soomere, 2007).

The method is shown on equation (4.5), where D is the draft, L the length, B the width, A_c the cross-sectional area and V the movement speed of the ship. It is important to note that the value of V will be the maximum allowed inside the port which is 5 knots (9.26 km/h).

$$H_i = 0.0448\, V^2 \left(\frac{D}{L}\right)^{0.5} \left(1 - \frac{BD}{A_c}\right)^{-2.5} \tag{4.5}$$

As mentioned before, there is little information about the vessels that use the Leixões port. Furthermore, there are no measurements available on vessel waves inside the harbour basin. Hence, the approach followed to estimate these waves is based on the PIANC Harbour Approach Channels Design Guidelines manual (PIANC, 2014) which provides guidelines and recommendations for the design of vertical and horizontal dimensions of harbour approach channels and is very often the manual followed in similar studies where limited information is available. To that purpose, the dimensions of the approach channel and rotating basin were used as reference to extrapolate the possible vessel design dimensions. The assumed design vessel dimensions are provided in Table 9. Using these dimensions in equation (4.5) yields a wave height of 0.45 m that is relatively high comparatively to the other components.

Table 9 - Calculated vessel dimensions.

Vessel Dimensions		
D	L	B
13.5 m	300 m	40 m

4.2. Outer Breakwater

For the outer breakwaters, the tool XGB-Overtopping was used to gain understanding on how the wave height affects the structure and the wave transmission over the crest and thereby the overall port basin and operations influence.

The XGB-Overtopping tool needs certain input parameters which were obtained through existing data of the breakwater and its profiles and refer to the structure itself. The remaining inputs refer to hydrodynamic boundary conditions. Specifically, the tool requires 15 input variables where 3 refer to the sea conditions ($H_{m0,\,Toe}$, $T_{m-1.0\ Toe}$ and β) and the rest are associated with the geometrical design of the structure (h, h_t, h_b, B_t, B, γ_f, α_d, α_b, α_u, R_c, A_c, G_c), Figure 19.

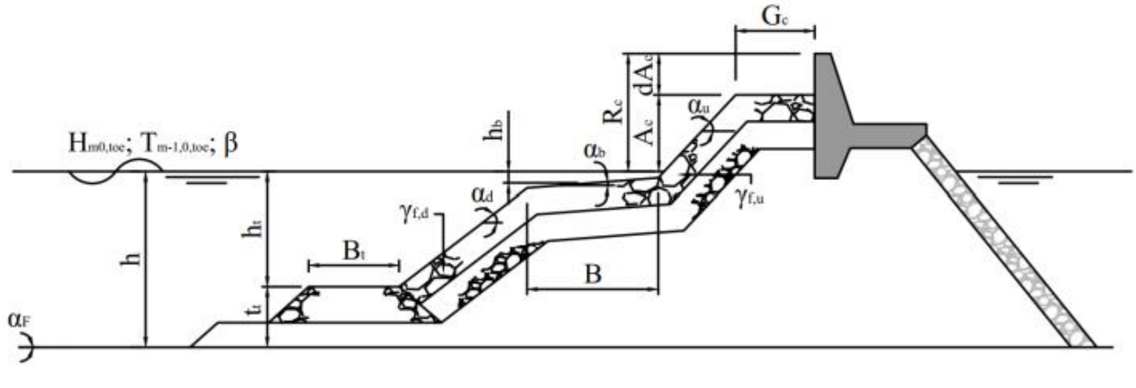


Figure 19 - Schematization of the input parameters (XGB-Overtopping Manual).

Although very useful, there are some limitations of the tool, since it must follow a schematization of the actual structure, which is not straightforward if the breakwater under study varies too much from the model, making the interpretation of the results more difficult. In the case of the port of Leixões, it was required to approximate the typical cross-sections of the breakwaters to the schematization shown on Figure 19. A selection of 4 locations along the breakwater of the Leixões port were chosen, as indicated on Figure 22. The associated cross-sectional profiles are presented on the following Figure 23 to Figure 25.

To characterize the different cross-sections, it was necessary to adapt the tool to this particular cases. Profile 1 (Figure 23) is composed by more than two slopes with different angles (Figure 20). As such, it was chosen to consider a fictional berm (parameter B is null) at the top point where the slope variation occurs and also the existence of a fictional toe berm at the lower slope variation point, where B_t is zero.

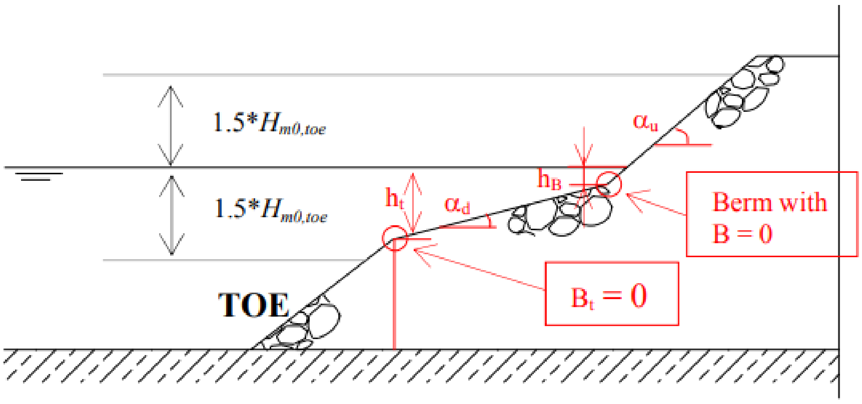


Figure 20 - Structure composed by several slopes (Coeveld *et al*., 2005).

Another situation that needs special attention is the definition of the inputs. In the case the crest isn't horizontal, it is advisable to consider a fictional crest elevation that corresponds to the medium point of the real crest. This will influence the G_c and A_c parameters. A scheme of the solution is shown in Figure 21.

Profile 1 presents two particularities; a wave return wall at the crest and a submerged breakwater adjacent to the main defense structure. To define the first one the solution was to apply a variation to the roughness and permeability parameter (Coeveld *et al*., 2005), this is obtained through the equation:

$$\begin{cases} R/H \geq 0.5 \rightarrow \gamma = \gamma - 0.05 \\ R/H < 0.5 \rightarrow \gamma = \gamma \end{cases} \tag{4.6}$$

To study the behavior of the wave impacts when in the presence of a submerged breakwater it would be necessary to apply other inputs that the XGB-Overtopping tool cannot cover so it had to be despised.

The second profile (Figure 24) is relative to the central part of the outer breakwater and is similar to profile 1 with the exception of the submerged breakwater that does not exist in this part. It is also composed by more than two slopes with distinct angles, which implies the creation of a fictional berm, like in the previous example (Figure 20).

Profile 3 is also similar to the last two profiles in light of the fact that is also composed by two slopes with different angle values which will apply the same solution to define this structure.

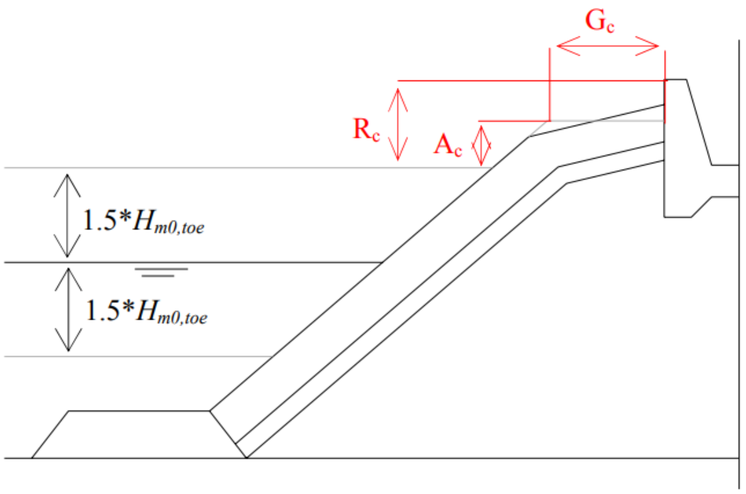


Figure 21 - Case of sloping crest (Coeveld *et al*., 2005).

The considered inputs are presented in Table 10.

Table 10 - Input values for each profile.

Profiles	β (º)	h (m)	H_t (m)	B_t (m)	H_b (m)	B (m)	R_c (m)	A_c (m)	G_c (m)	Cotα_d (-)	Cotα_u (-)	$\gamma_{f,d}$	$\gamma_{f,u}$	tan α_F
P1	0	17	17	0	1	2.72	15	11.5	4.09	2	1.33	0.42	0.33	0
P2	0	16.5	16.5	0	1.77	2.72	15	11.5	4.09	2	1.33	0.42	0.33	0
P3	0	13.8	13.8	0	14	0	12.2	6	10	1	3	0.47	0.47	0
P4	0	13.8	13.8	0	14	0	12.2	6	10	1	3	0.47	0.47	0

To complete all the inputs, it is also necessary to define the wave height and the wave period that will be used to calculate the overtopping discharge on the previously referred profiles. As a matter of comparison different situations were taken into account to further understand how the breakwater works when protecting the port from the waves.

For the first case, it will be evaluated the wave height following an extreme analysis for a return period of 50 years. To proceed with the analysis, it was necessary to collect the data regarding the significant wave height available from 2004 until 2020, then, applying the annual maximum wave method it was possible to carry out with the extreme value analysis and apply a generalized extreme value (GEV) distribution to obtain the values shown on Figure 26 with 90% confidence intervals.

To calculate the wave periods that correspond to the significant wave heights, it was necessary to process the data in such a way that only the bigger waves that are also concentrated on the main direction of wave propagation will be considered. Afterwards, with the data tapered, a fitting was done to calculate the peak wave period for the wave heights considered in the previous demonstrated extreme analysis leading to the values shown on Figure 27, Figure 28 and Figure 29.

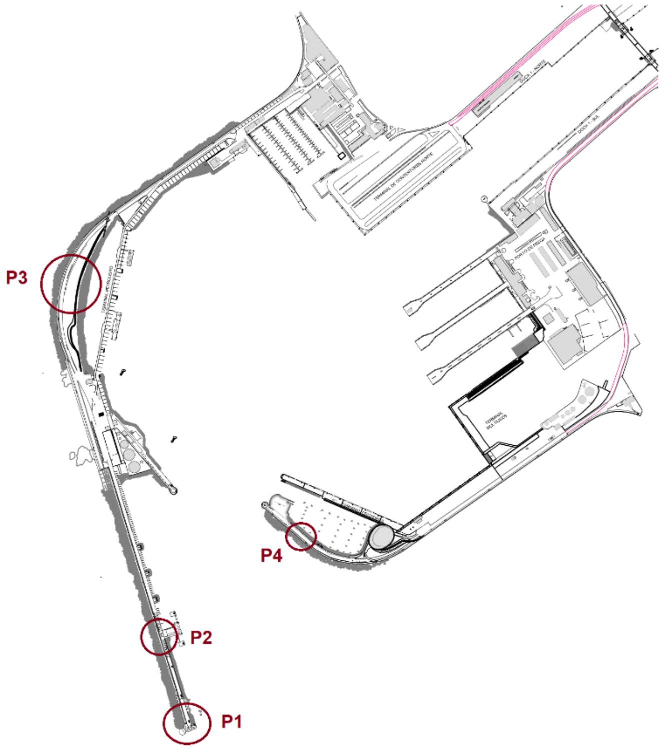


Figure 22 - Port layout (Provided by APDL).

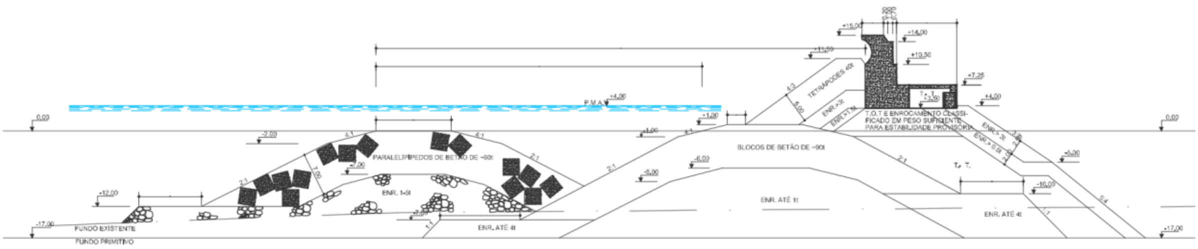


Figure 23 - Breakwater profile 1 (P1) (Provided by APDL).

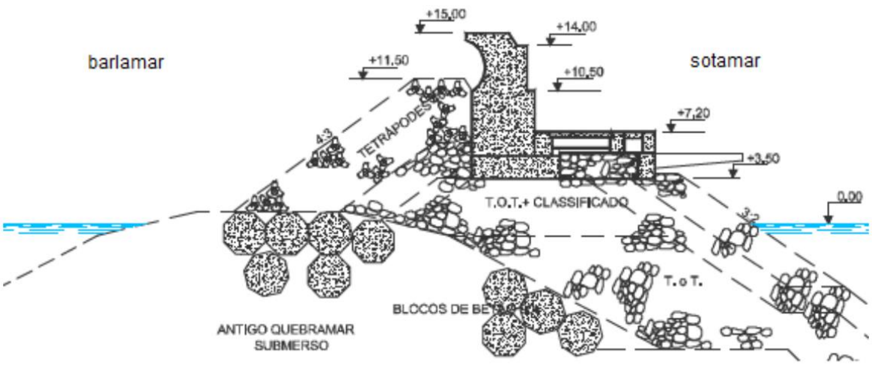


Figure 24 - Breakwater profile 2 (P2) (Provided by APDL).

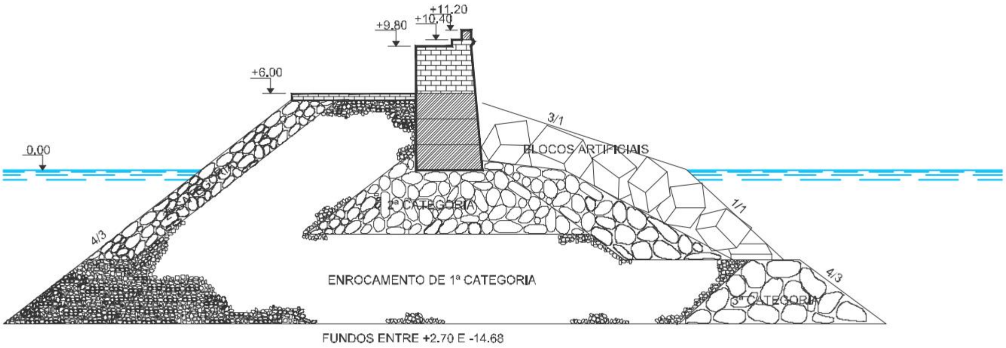


Figure 25 - Northern and Southern breakwater profiles, P3 and P4 respectively (Provided by APDL).

The wave height and peak wave period values that were retrieved from the extreme analysis and correspond to significative return periods are presented in Table 11.

Finally, to calculate the overtopping it was necessary to import all the values corresponding to the breakwater structural characteristics and the maritime conditions into the overtopping calculator which will provide different graphics for different situations for all of the profiles.

Table 11 - Significant wave heights and peak wave periods for different return periods.

Return Period (Years)	Significant Wave Height (m)	Peak Wave Period (s)
2	7	13.5
0	9	15.5
50	11	17

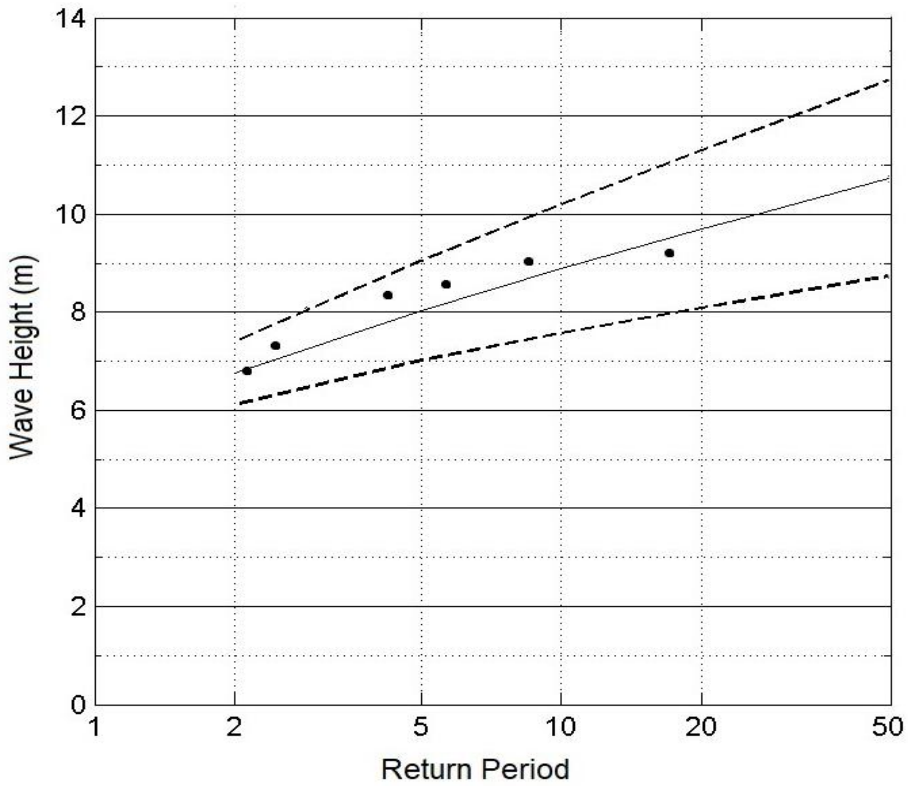


Figure 26 - Wave height extreme analysis

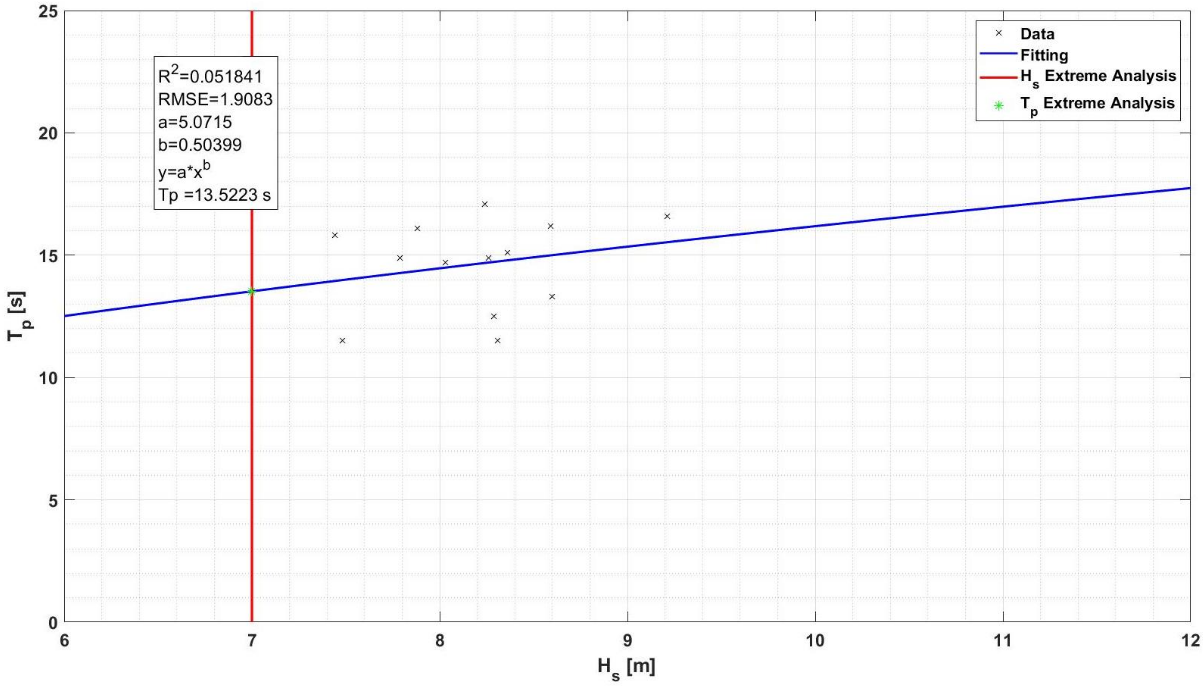


Figure 27 - Peak wave period for a return period of 2 years.

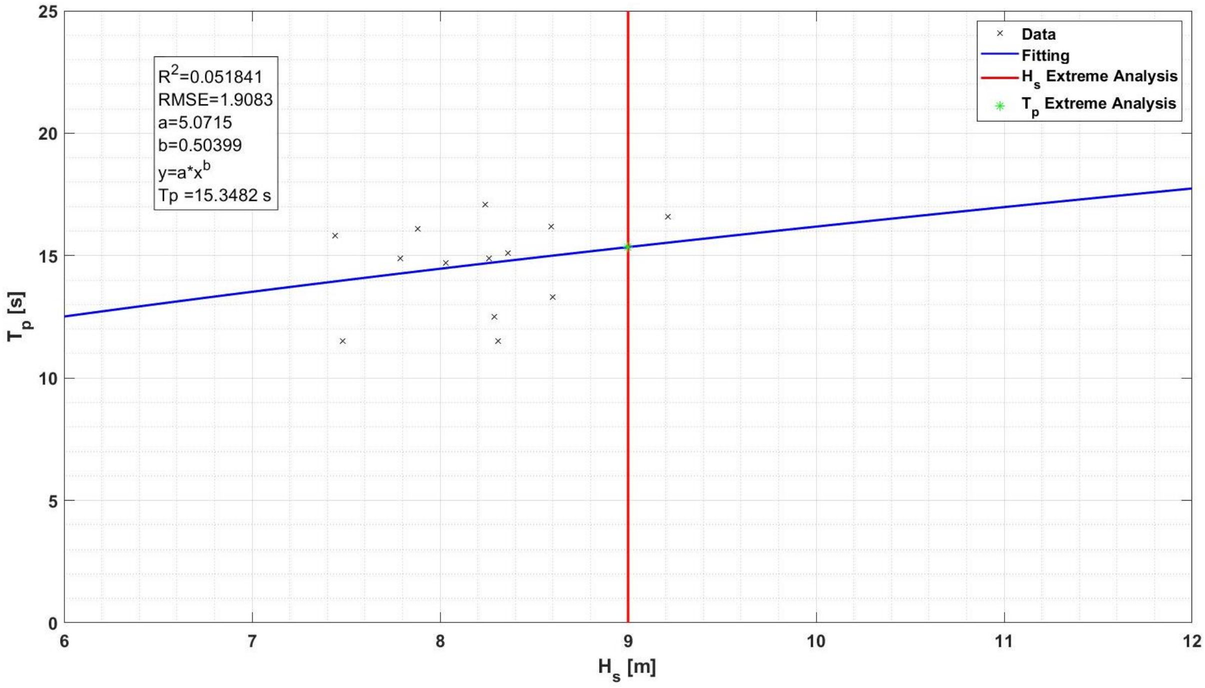


Figure 28 - Peak wave period for a return period of 10 years.

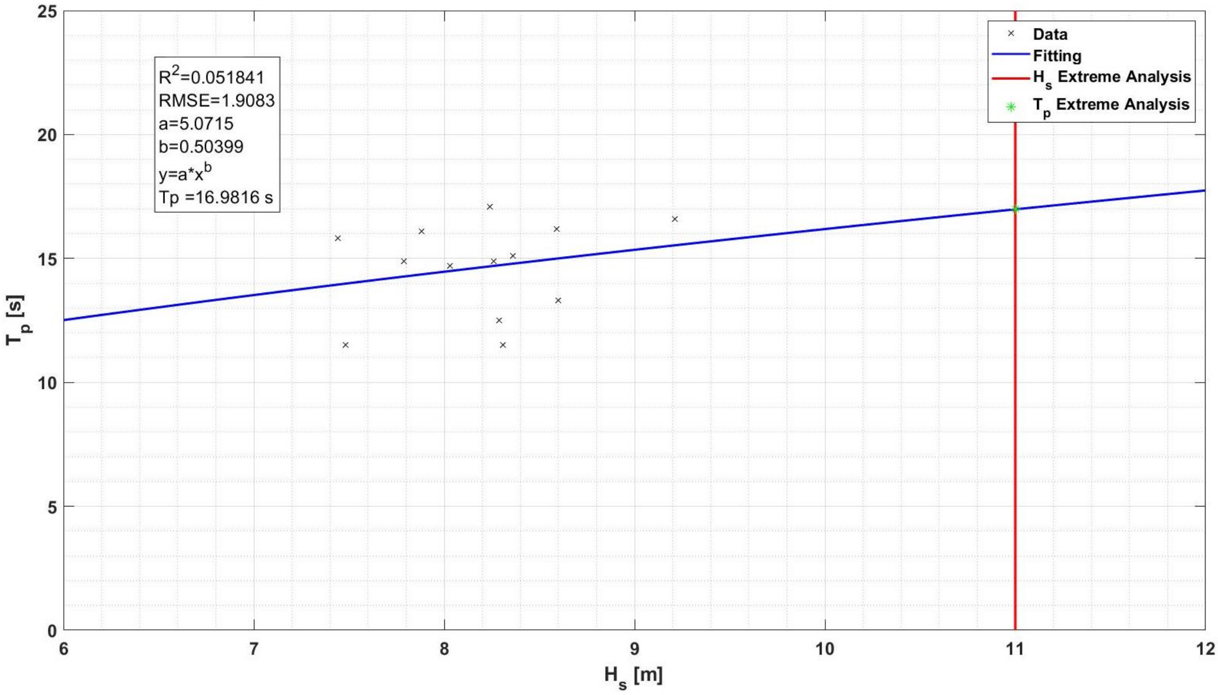


Figure 29 - Peak wave period for a return period of 50 years

4.3. Inner Quays and Terminals

For the methodology to be applied on the overtopping of inner terminals and quay-walls, it was necessary to define local hydrodynamic conditions to be used, as wave height, wave period and wave direction will be used and how the sea level impacts different scenarios. It was also important to characterize the structures, which type of terminals or how the structure type affects the end results and how is the port characterized. To analyze all the terminals, it was necessary to divide the port into zones where the docks have different characteristics. According to the water depth, the type of terminal or any other variation, the port was separated in 30 zones represented on Figure 30. Applying these data into empirical equations for each terminal and dock area it was possible to calculate the wave overtopping. Afterwards, the possible scenarios will include, the present situation, future sea level rise situations to understand how it impacts the terminals, and scenarios with different mean discharge limits. In the end, the different scenarios lead to different results and accordingly to those, compatible measures will be selected and justified. A MATLAB tool that had been developed by Bolle *et al.*, (2018) for the same purpose in an evaluation of the Port of Zeebrugge in Belgium, was adapted to the present case hither go obtaining results for this study.

4.3.1. Hydrodynamic Conditions

The first step to start the analysis is to define the maritime conditions to be used on this analysis. These will be constant throughout the analysis and consist of determining the wave height for a certain return period, the corresponding wave period and wave direction. In order to do so, its necessary do take into account the previous mentioned forms of wave climate inside the port, locally generated wind waves, offshore waves penetrating into the harbor and waves created by the passing vessels, so, following that line of thought to calculate the wave height at each terminal it was used equation (4.7).

$$H_{m0} = \sqrt{{H_{m0\,wave\,agitation}}^2 + {H_{m0\,wind\,waves}}^2 + {H_{m0\,vessel\,waves}}^2} \tag{4.7}$$

The procedure of equation (4.7) is a common simplification of the real physics of the problem that could provide inaccurate predictions of the wave height distribution along the harbor quay walls, although it can lead to lower quality predictions of the overtopping discharges and consequently of the hinterland flooding predictions (Bolle *et al.*, 2018).

The wave period to be used will be the same as the correspondent to the offshore wave applied when calculating the wave agitation inside the port.

The wave direction parameter will take the values that contributed to the worst agitation inside the port at each zone and was used to calculate the wave attack angle (Table 12). Comparing Table 12 with Table 8 it is possible to conclude that the main influence factor is the penetration of offshore waves inside the port.

Table 12 - Wave height and wave direction for each zone.

Zones	Wave Height (m)	Wave Direction (º)	Zones	Wave Height (m)	Wave Direction (º)
1	2.80	198.34	16	0.96	286.40
2	2.22	226.15	17	1.73	250.45
3	1.38	213.97	18	1.07	274.45
4	1.24	284.48	19	0.95	282.15
5	1.25	284.48	20	1.22	232.21
6	1.65	226.75	21	2.28	238.61
7	1.65	226.75	22	2.28	238.61
8	1.90	224.98	23	1.47	238.53
9	0.70	245.78	24	1.48	238.53
10	0.73	139.02	25	1.15	279.70
11	0.63	149.30	26	0.98	317.40
12	0.63	149.30	27	0.55	313.51
13	0.62	247.38	28	1.13	331.22
14	0.77	240.16	29	1.52	226.31
15	0.96	286.39	30	1.51	239.64

4.3.2. Structure Characteristics

In chapter 3, a description of the structures was already made, giving an illustration of the terminals and docks that characterize the port, but it is also necessary to analyze better these structures, since their characteristics are one of the most important inputs to obtain results. The structural inputs that the tool requires are the depth of the seabed, depth at the toe of the structure, the level of the quay wall and influence factors such as roughness, and the type of structure if it is a dike, armored or vertical wall.

All of these characteristics were provided by APDL through a map and cross-sectional profiles. These are demonstrated respectively in Figure 30 and from Figure 31 to Figure 44.

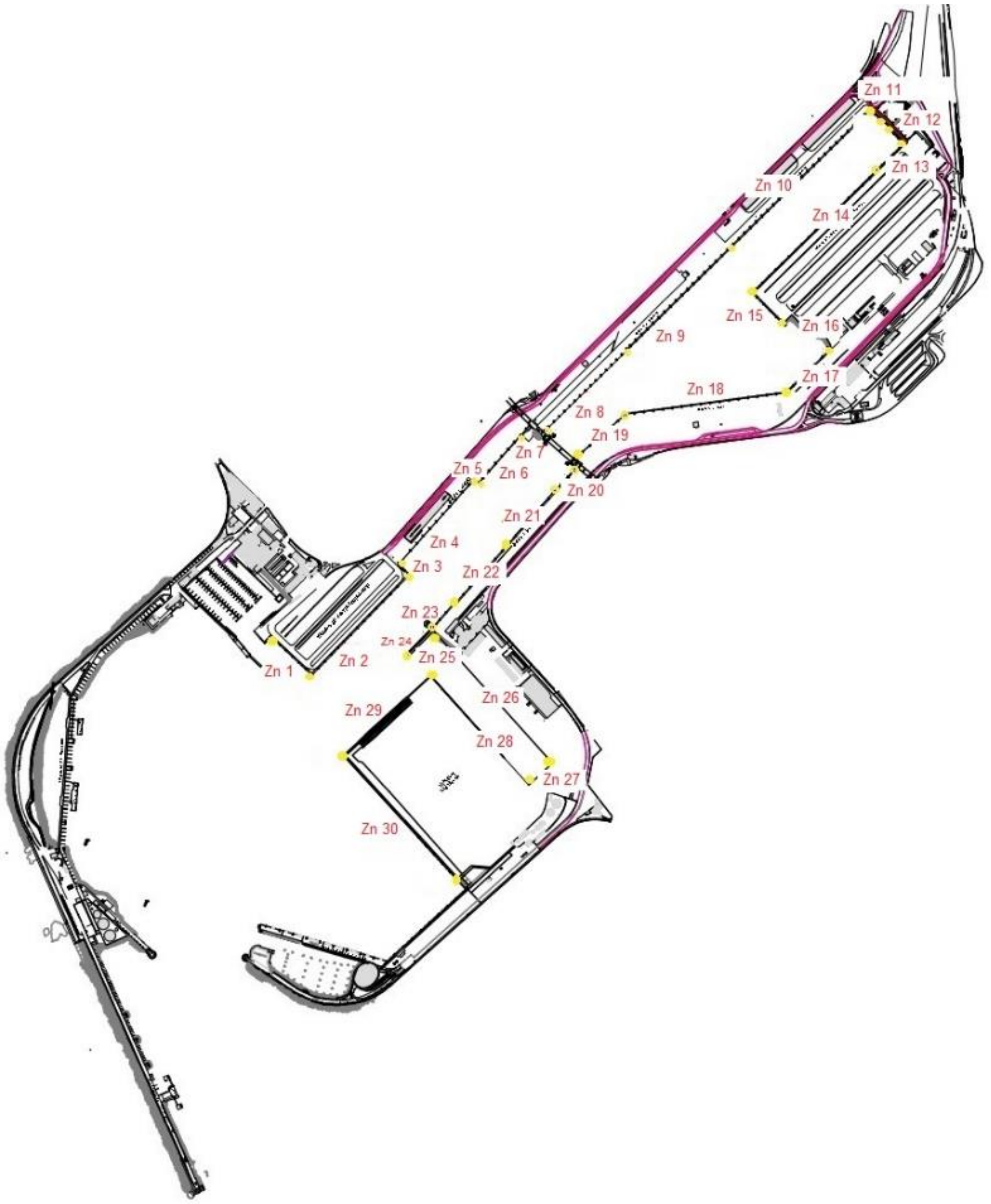


Figure 30 - Defined zones of the port used in the analysis.

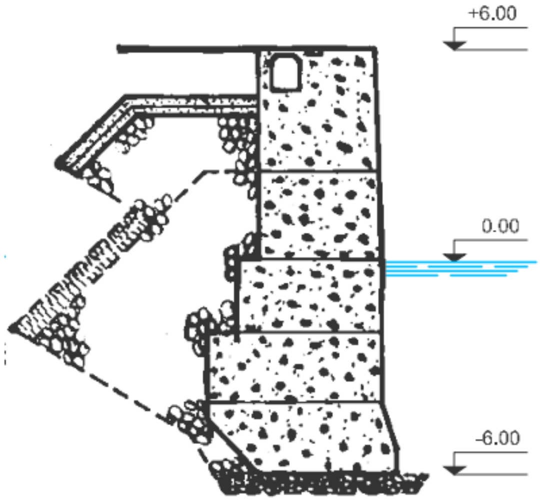


Figure 31 - North Terminal (Zone 1).

Figure 32 - North Terminal (Zone 2 and 3).

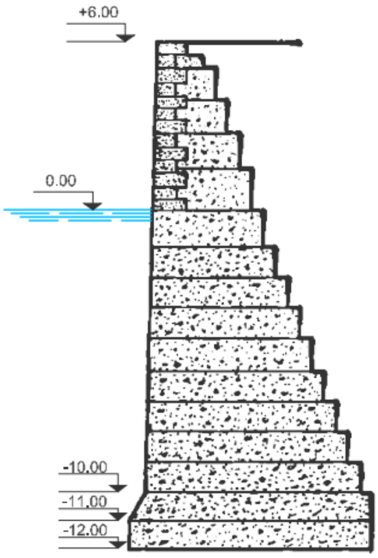


Figure 33 - North and South dock 1 (Zones 4, 5, 20 and 22).

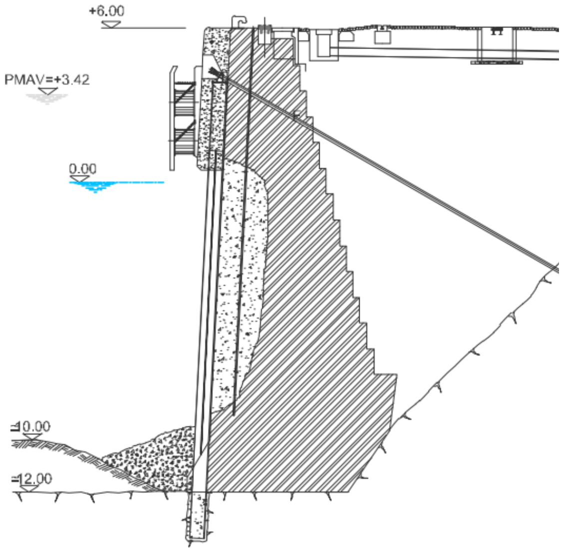


Figure 34 - Dock 1 South (Zone 21).

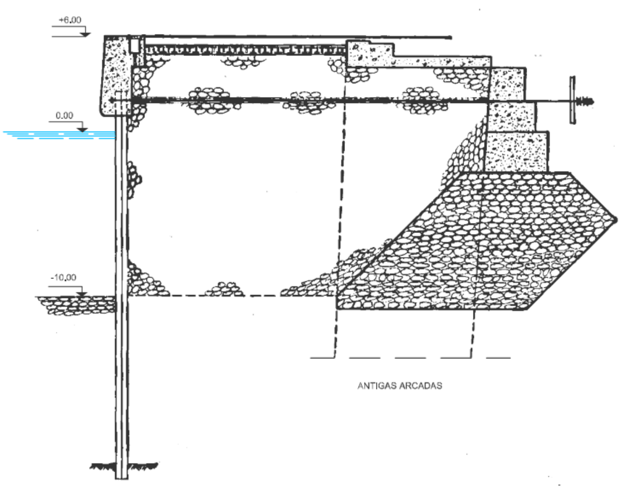


Figure 35 - Dock 1 North (Advanced Part) (Zone 6).

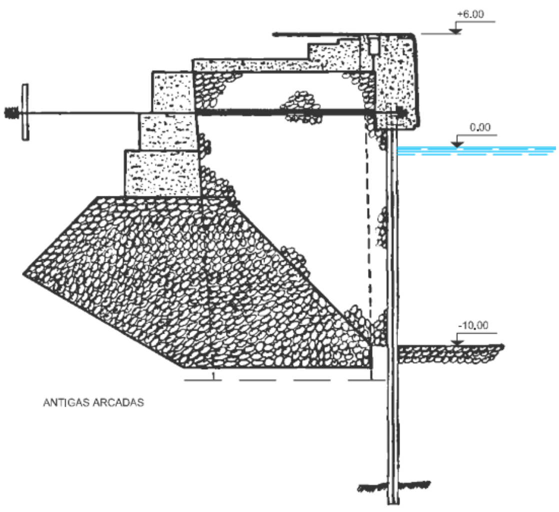


Figure 36 - Dock 1 South (Advanced Part) (Zone 23 and 24).

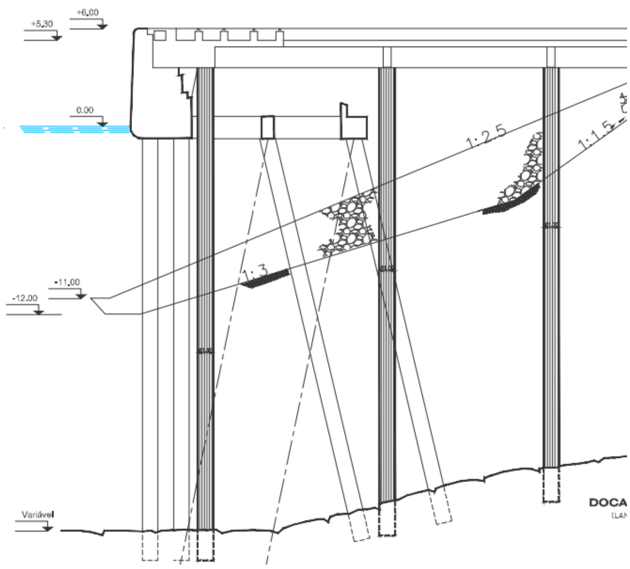


Figure 37 - Dock 2 North (Zone 8).

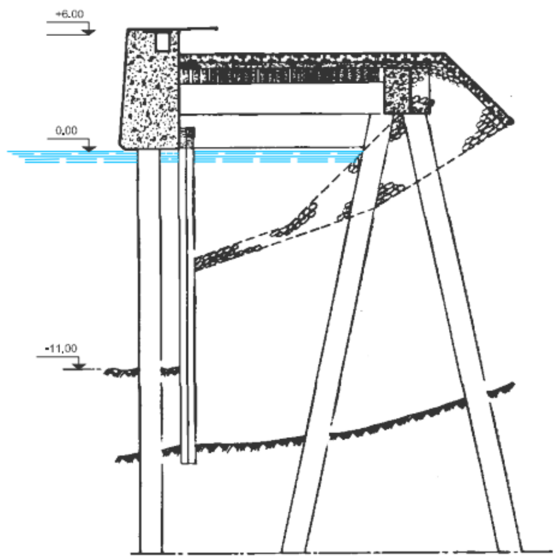


Figure 38 - Dock 2 North (Zone 9).

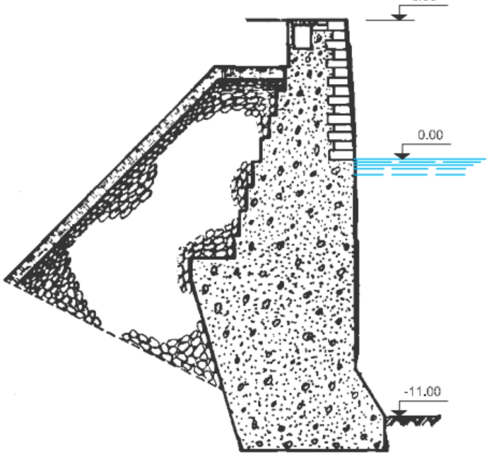


Figure 39 - Dock 2 South (Zones 17, 18 and 19).

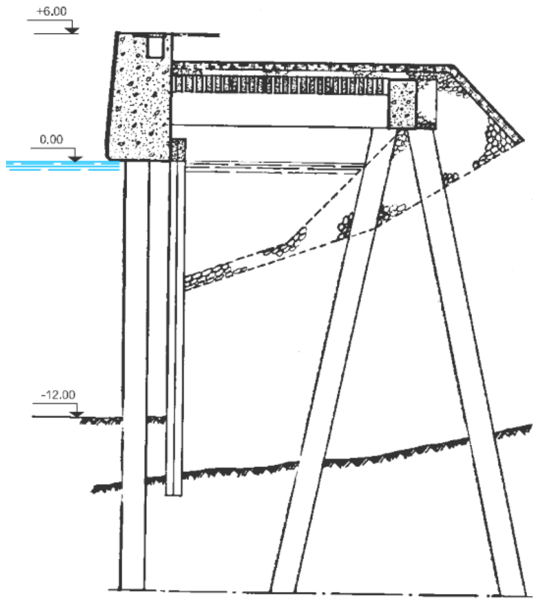


Figure 40 - Dock 4 North (Zone 10).

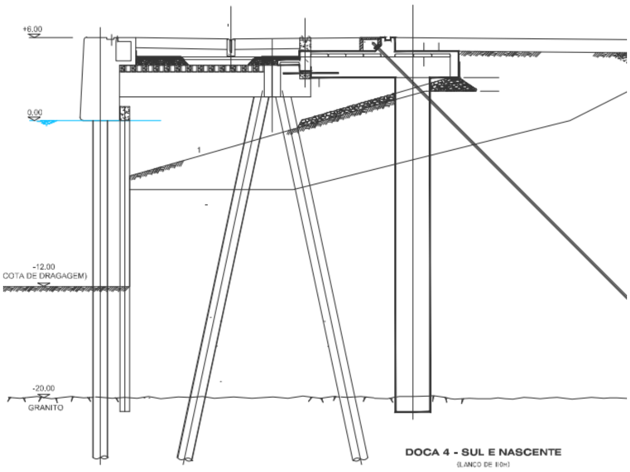


Figure 41 - Dock 4 South and East (Zones 11, 12 and 13).

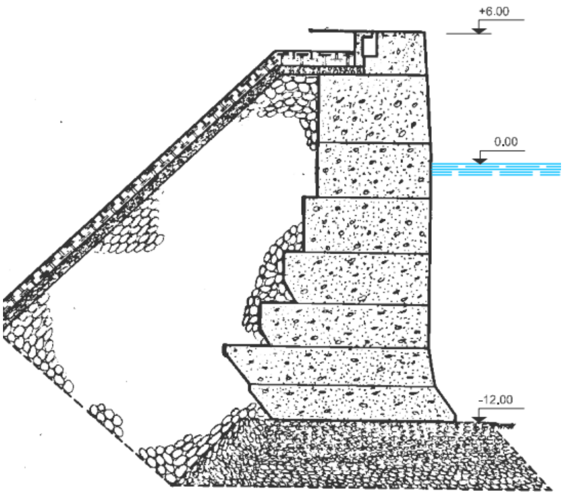


Figure 42 - Dock 4 South (Zone 14).

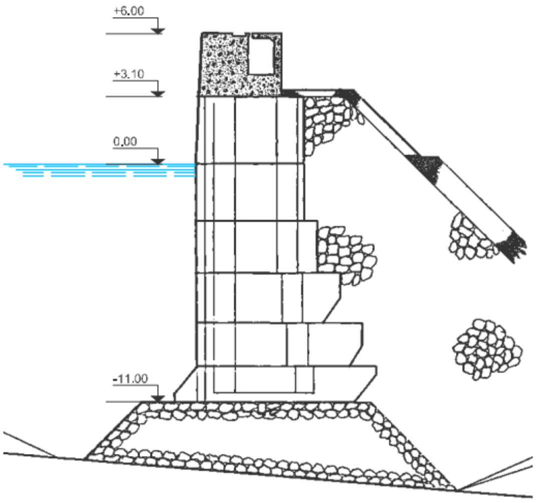


Figure 43 - Dock 4 West (Zone 15).

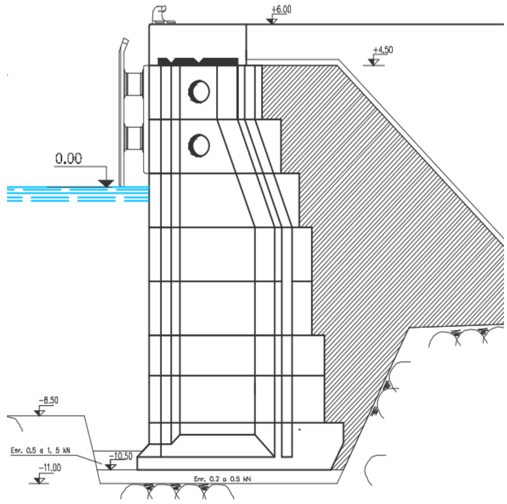


Figure 44 - Multipurpose Terminal (Zone 29 and 30).

There are some zones that are not defined by the profiles: zone 7 which is a ramp and zones 25 to 28 which don't exist right now and are part of the model used for future undergoing constructions but since it is already known that their structure will be a vertical wall, it was used a simple vertical wall model that can still characterize these zones.

Another aspect that needs special attention is the crest level. Currently every structure has an elevation of +6.0 m ZH, which is in accordance with the recommendation measures from the Spanish ROM 2.0-11 (Manuel González Herrero *et al.*, 2011). What is recommended regarding the berth draught (distance from the bottom sea level to the reference water level) is that the interruption of the ship's staying at berth from insufficient draught should not occur, so that other stoppage reasons would be the ones determining the operational level of the installation. Moreover, since the threshold values of climatic agents adopted as the limit for the performance of ship departure operations from the berth cannot be more restrictive than those for staying the ship at berth (otherwise, the ship could leave the berth but could not sail), it may be advisable to dimension the access and maneuvering areas using economical optimization criteria, to separate the stoppages of ship's staying at berth from the stoppages of maritime accessibility in relation to the adopted operational outer water level thresholds caused by tides (Manuel González Herrero *et al.*, 2011).

So, following the ROM document guidelines, the criteria to determine the minimum top levels of berthing structures for Leixões would require a freeboard between +1.5 and +2.5 m and in this case, the freeboard sits at +2 m which is inside the range values. The criteria applied here is available in Figure 45 and Table 13.

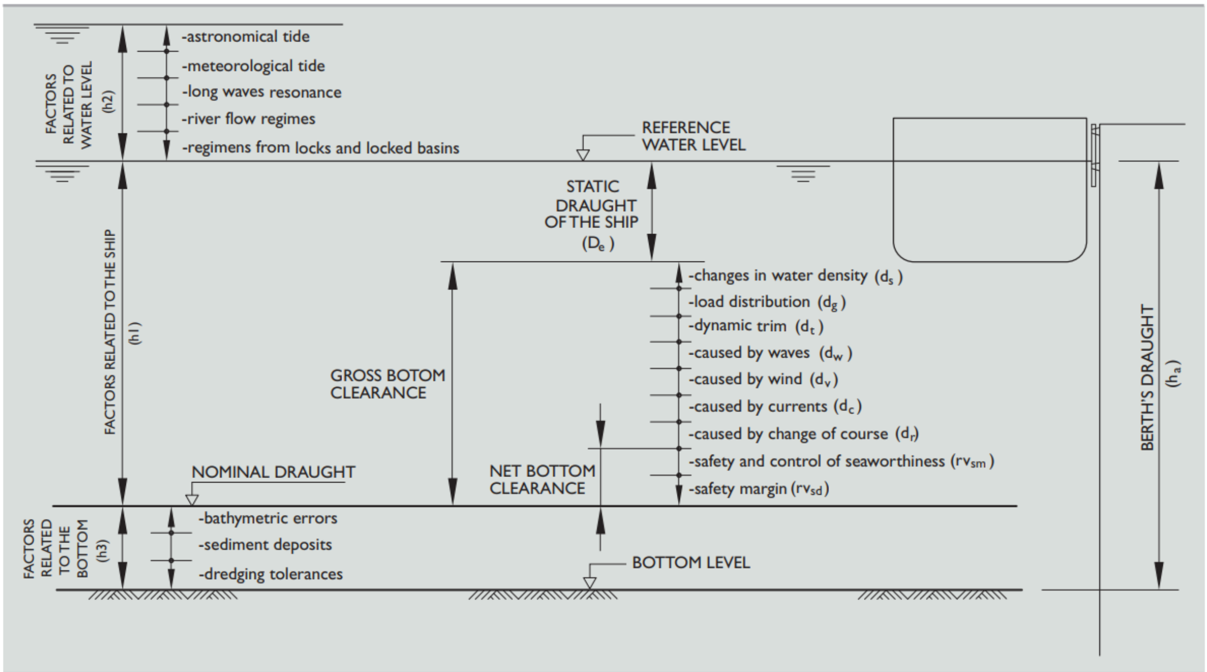


Figure 45 - Factors affecting the definition of the berthing alignment draught (ROM 2.0-11, 2019).

Table 13 - Criteria to determine the minimum top levels at fixed berthing structures (ROM 2.0-11, 2019).

CONDITIONS TYPE	REFERENCE LEVEL OF OUTER WATER	USE OF THE BERTHING STRUCTURE	FREEBOARD (IN M)
FROM OPERATION CONDITIONS	Upper level of operational tidal window 1)	Commercial, industrial and military uses	+ 1.50 ~ + 2.50 3)
		Fishing use	+ 0.50 ~ + 1.00 4)
		Nautical-sports uses	+ 0.15 ~ + 1.00 5)
FROM NO OVERTOPPING CONDITIONS OF OUTER WATER	Upper level of outer waters extreme window 2)	All uses	+ 0.50
FROM NO FLOODING CONDITIONS OF BACKFILL WATER TABLES	Upper level of backfill water tables extreme window	All uses	+ 0.50

Notes

(1) Operational window associated to tides (astronomical and meteorological) abd river flow regimes, when appropriate.
(2) Extreme window of outer waters, considering all agents affecting levels of outer waters on site (tides, waves, long waves, ...).
(3) A freeboard of 1.5 m will be considered when displacement of the largest vessel of the expected fleet at berth is lower or equal to 10000 t. For higher displacement of that vessel, the adopted freeboard may be up to 2.50 m.
(4) A freeboard of 0.5 m will be considered for short length vessels (< 12 m). Also, it is recommended in these cases that, from the lower level of the operational tidal window, the resulting freeboard up to top level will not br higher than 1.5 m. When not possible, a floating solution will be adopted.
(5) A freeboard of 0.15 m will be considered for short length vessels (< 12 m). Also, it is recommended in these cases that, from the lower level of the operational tidal window, the resulting freeboard up to top level will not br higher than 1.00 m. When not possible, a floating solution will be adopted.

4.3.3. Evaluation of the overtopping

At this point, the maritime inputs alongside the structure characteristics when applied to the tool will provide the awaited results. The scenarios that are going to be studied consist of the current situation at the port (null sea level rise), and future situations of possible sea level rises (SLR) in Portugal, 0.5, 1 and 1.5 m. Apart from the current situation, the hypotheses that are going to be studied are all represented by how the crest elevation varies according to the mean overtopping discharge, what this means is that an assessment was made, through an iterative process, that compares the current crest elevation with the required one according to a previously defined overtopping discharge limit. It is very important to define the limits, since this way, even more broader scenarios can be analyzed. The chosen limits for this study were based in an EurOtop Manual (2018) tables (Table 14), that defines different limits for different outcomes, for example, if the limit defined is 1.0 L/s/m, it is expected that people will be accessing these dikes or terminals, on the other hand if the purpose isn't to expect people, the limit can be slightly raised to 5.0 L/s/m. In this case, the chosen limits were, 1.0 L/s/m, 5.0 L/s/m and the present design mean discharge limit values. What this means is that some overtopping discharge limits were defined to design the terminals and docks that exist nowadays, and when applying these limits to different sea level rise scenarios, a more accurate and possible picture of how the quays and terminals will behave to these changes is obtained. The current design limits that were applied for each zone are showed in Table 15.

In order to complete this graphical analysis, a simple scheme of the Leixões port will be represented alongside the results in order to identify which areas and terminals may be more or less affected.

The results obtained applying the methodology are presented in the next chapter and it is important to note that in every parameter previously referred, the values that were chosen and applied in the methodology were always the ones associated to the worst-case scenario. So, even though it is unlikely that waves of this magnitude reach the inner quays of the Leixões port, for this analysis it was chosen to always apply the most conservative values in order to ensure "design for safety" results.

Table 14 - EurOtop limits for mean discharge (EurOtop, 2018).

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 2000 2000 2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Significant damage or sinking of larger yachts; H_{m0} > 5 m	>10	>5,000 – 30,000
Significant damage or sinking of larger yachts; H_{m0} = 3-5 m	>20	>5,000 – 30,000
Sinking small boats set 5-10 m from wall; H_{m0} = 3-5 m Damage to larger yachts	>5	>3,000-5,000
Safe for larger yachts; H_{m0} > 5 m	<5	<5,000
Safe for smaller boats set 5-10 m from wall; H_{m0} = 3-5 m	<1	<2,000
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000

Table 15 - Current mean overtopping discharge limits.

Zone	Current mean discharge limit (l/s/m)	Zone	Current mean discharge limit (l/s/m)
1	200	16	1
2	50	17	15
3	5	18	1
4	5	19	1
5	1	20	1
6	80	21	60
7	1	22	60
8	1	23	5
9	1	24	5
10	1	25	1
11	1	26	1
12	1	27	1
13	1	28	1
14	1	29	8
15	1	30	8

5
RESULTS ANALYSIS

The study results will be presented in this chapter and will consist of the evaluation of the overtopping both for the outer breakwaters and for the inner terminals and quay-walls.

5.1. OUTER BREAKWATER RESULTS

The results for the outer breakwater analysis consist of 3 graphics representing the mean overtopping discharge for the 4 different sections corresponding to profiles 1 to 4, respectively, presented in the methodology.

The results represent the values of $Q_{(95\%)}$ and $Q_{(5\%)}$ with 95% and 5% quantiles. For a return period of 2 years (Figure 46) the results were positive, with small mean overtopping discharges overall. The graphic presents values of less than 1.0 L/s/m for sections 1 and 2 and a slightly higher discharge for sections 3 and 4.

For the 10-year return period (Figure 47) the results show an increase in the value of q, with values around 10 L/s/m for section 1 and 2. For the remaining sections the values surpass the 10 L/s/m.

Finally, for a return period of 50 years (Figure 48), the results present a high increment for all of the sections. Sections 1 and 2 with values slightly less than 100 L/s/m and sections 3 and 4 present values larger than 100 L/s/m.

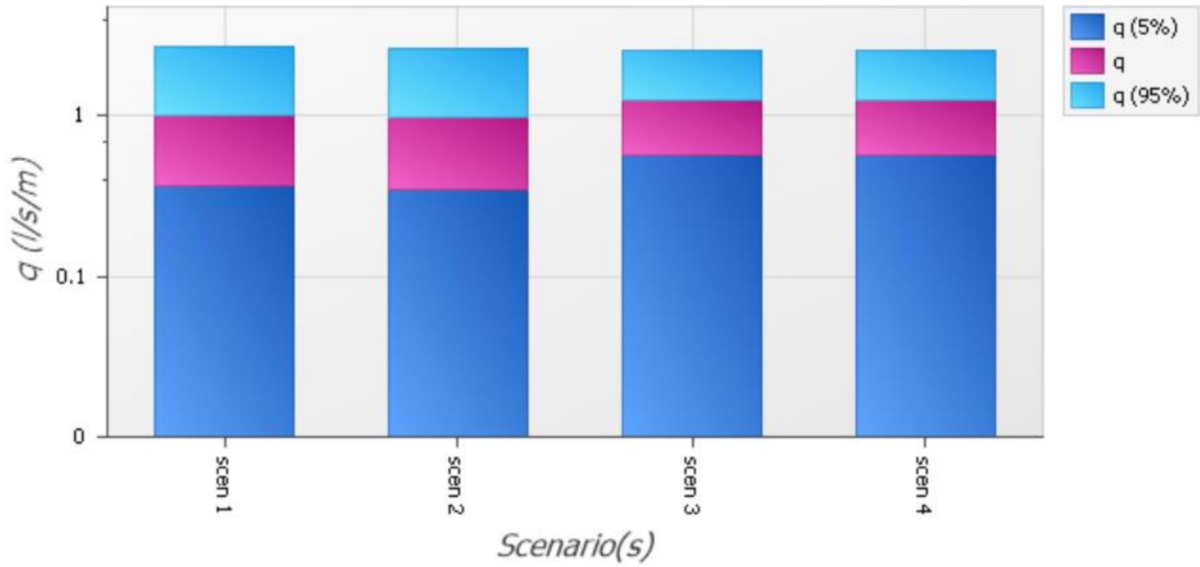


Figure 46 - Mean overtopping discharge for a return period of 2 years.

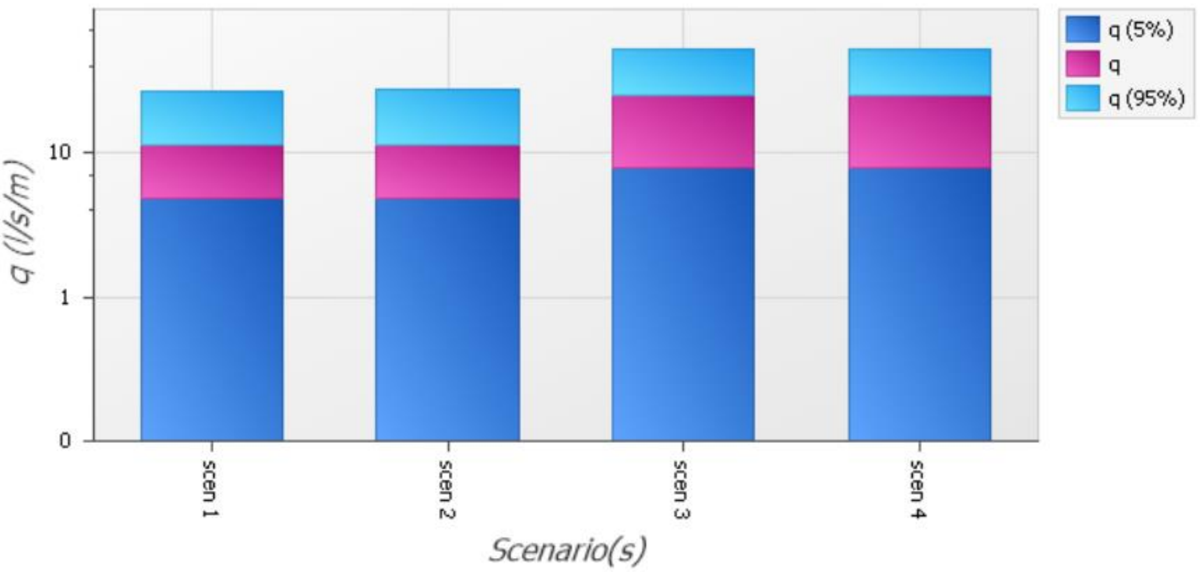


Figure 47 - Mean overtopping discharge for a return period of 10 years.

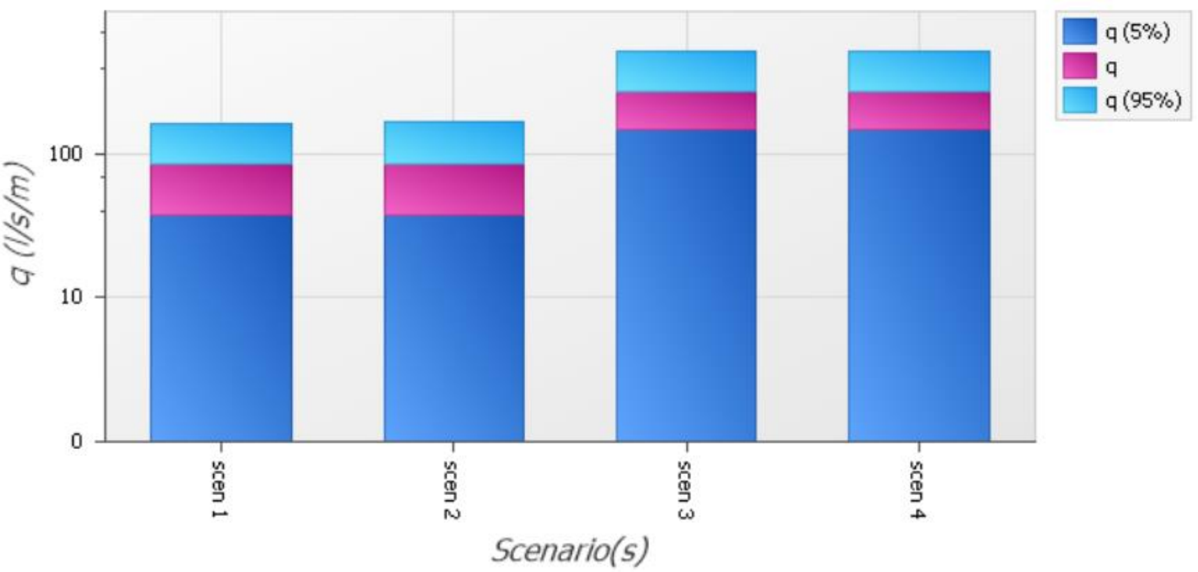


Figure 48 - Mean overtopping discharge for a return period of 50 years.

5.2. INNER QUAY-WALLS RESULTS

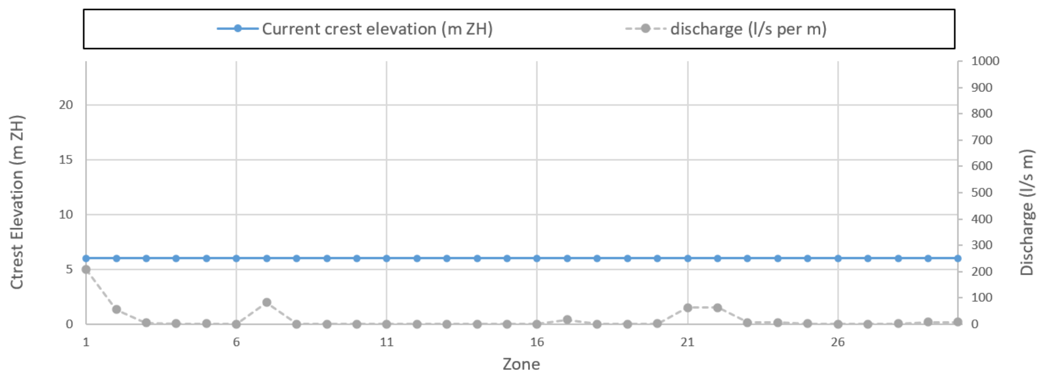
5.2.1. CURRENT CREST ELEVATION (NO CHANGES)

The results of the tool were obtained for the current situation and are presented in this subchapter, which translated into the mean overtopping discharge per meter, referring to the 30 zones and the 4 sea level rises scenarios analyzed. The tables are completed by a graphic analysis and a map with the zones with the worst mean discharge values. The graphs show a comparison between the values of q obtained for each zone and the value of the current elevation of each structure. Naturally the overtopping values will increase as the mean sea level rises. The results obtained made it possible to evaluate the presumed overtopping events that may exist in the structures and conclude which terminals are more exposed to maritime conditions and which are more protected considering their locations in the port and their physical characteristics. It is concluded that:

- The highest values of q correspond to the outer terminals at the northern areas;
- Comparing the graph and the map for SLR=0 (Table 16), the highest values 200 L/s/m correspond to zone 1 and 100 L/s/m correspond to zones 7 (Ro-Ro ramp), 21 and 22;
- At the SLR=0.5 m (Table 17) the values are slightly increased;
- As for the 1 m increament scenario (
- Table *18*) the values start to have high mean discharge values;
- The 1.5 m sea level rise (Table 19) presents extremely high values for zone 7 (almost 1000 L/s/m) and mean discharge of 600 L/s/m for zone 1.

Table 16 - Current Mean Overtopping Discharge.

SLR= 0



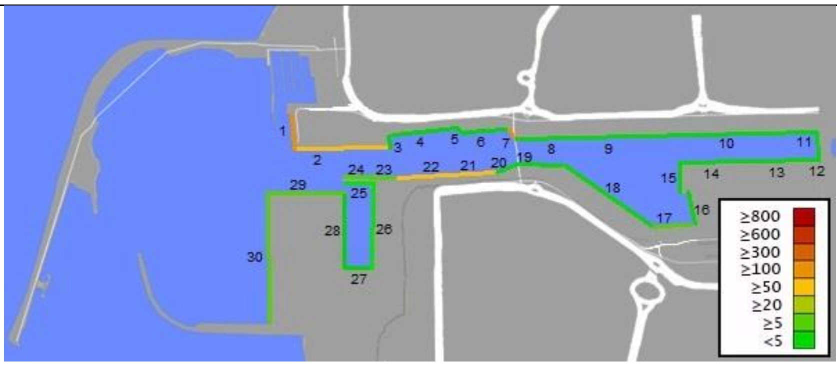
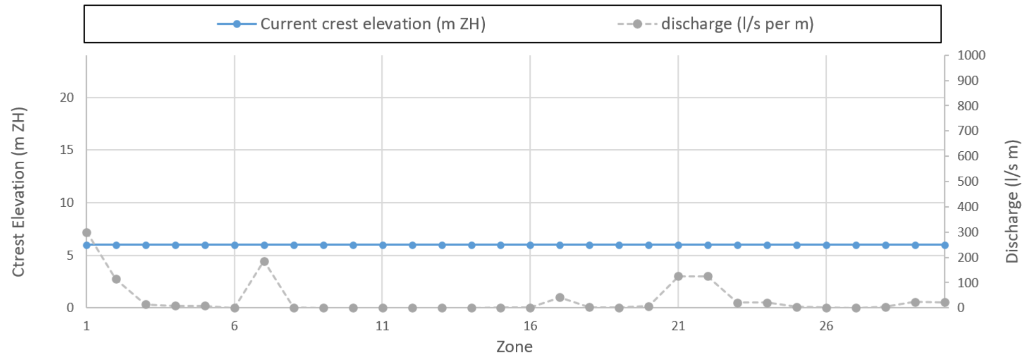


Table 17 - Mean Overtopping Discharge no crest elevation changes (SLR=0.5m).

SLR= 0.5m



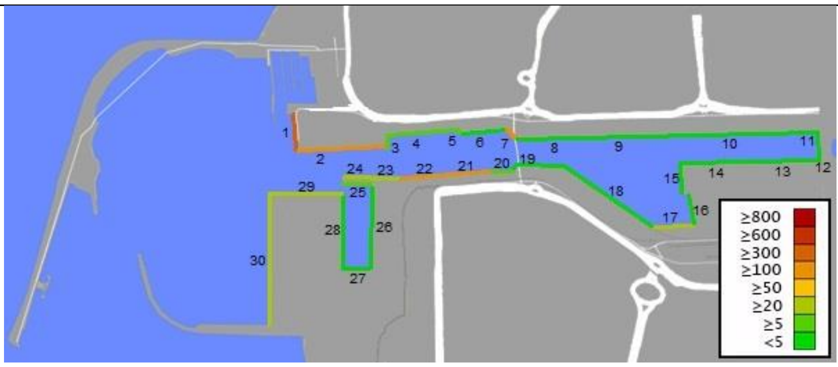


Table 18 - Mean Overtopping Discharge no crest elevation changes (SLR=1m).

SLR= 1m

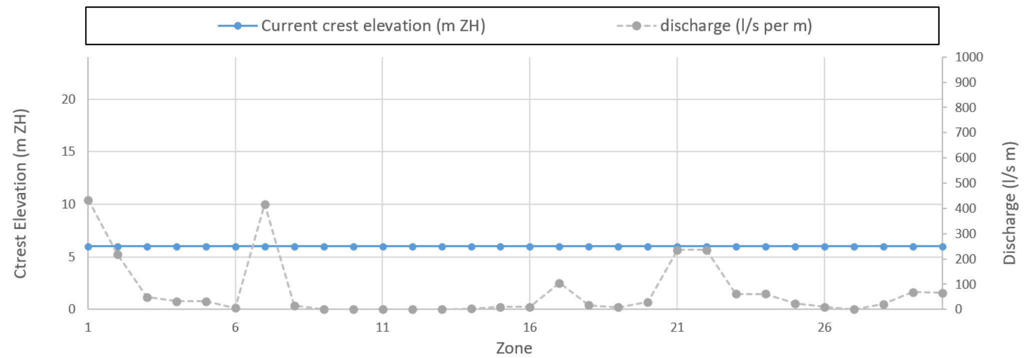
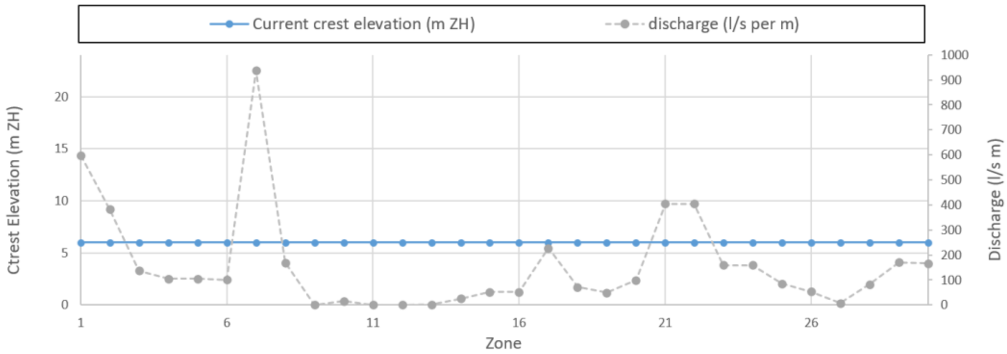


Table 19 - Mean Overtopping Discharge no crest elevation changes (SLR=1.5m).

SLR= 1.5m



5.2.2. WITH CREST ELEVATION CHANGES (1 L/S/M)

The results of the tool that were obtained for the limit of 1 L/s/m situation are presented in this subchapter. In this case, the crest was progressively elevated until the desired mean overtopping discharge limit was reached. The results are presented through tables referring to the required crest elevation of the 30 zones and the 4 sea level rises scenarios analyzed. The tables are constituted by a graphic analysis and a map with the zones with the highest required crest elevations, in this case it is more interesting to analyze that required crest variable instead of the mean discharge value since it will have a low value for every zone because of the increasing crest elevation. The graphs show a comparison between the values of q obtained for each zone and the value of the required elevation compared to the current elevation of each structure.

As it is expected, since the overtopping values increase as the mean sea level rises, the required crest elevation will also be higher. The results obtained made it possible to evaluate the presumed crest elevation that may exist in the structures and to conclude which terminals can in fact benefit from this solution.

For this mean discharge limit and SLR=0 (Table 20), it is concluded that:

- Zone 1 is the most aggravated, needing a 7 m elevation increment to induce a discharge value under 5 L/s/m, which is very unlikely (even impossible) to apply this mechanism as a solution;
- Zones 2, 7, 21 and 22 would require 3 m more to be able to stop the overtopping, which in some cases, depending on the structure characteristics could be applied.

When SLR=0.5 (Table 21):

- Zones 1, 2, 7, 21 and 22 still maintain the same conclusions as for SLR=0, but with the addition of zones 3, 4, 5, 20, 23, 24, 29 and 30 which are lightly increased by 1 m.

For SLR=1 (

Table *22*) the required crest starts to increase even more in more zones:

- Zone 1 requires an increment of 9 m;
- Other zones that are highly influenced by overtopping, zones 7, 21 and 22 require 4 m of crest elevation;
- Zone 2 will face a 3 m increase while zones 3, 17, 23, 24, 29 and 30 would need 2 more meters to counteract overtopping;
- For zones 4, 5, 15, 16, 18, 19, 20, 25, 26 and 28 a smaller elevation of 1 m is needed.

The highest SLR (Table 23) will induce the highest required crest elevation, and more specifically:

- For zone 1 the necessary crest elevation has the highest value of 9 m;
- The second highest elevation of the crest (4 m) occurs in zones 2, 7, 21 and 22. For zone 17 a 3 m elevation is necessary;
- The zones 3, 4, 5, 20, 23, 24, 25, 29 and 30 present more plausible structural changes to the terminals with a required crest of 2 m elevation and only 1 m for the zones 6, 8, 14, 15, 16, 18, 19, 26 and 28.

Table 20 - Mean Overtopping Discharge for 1 L/s/m limit (SLR=0m).

SLR= 0

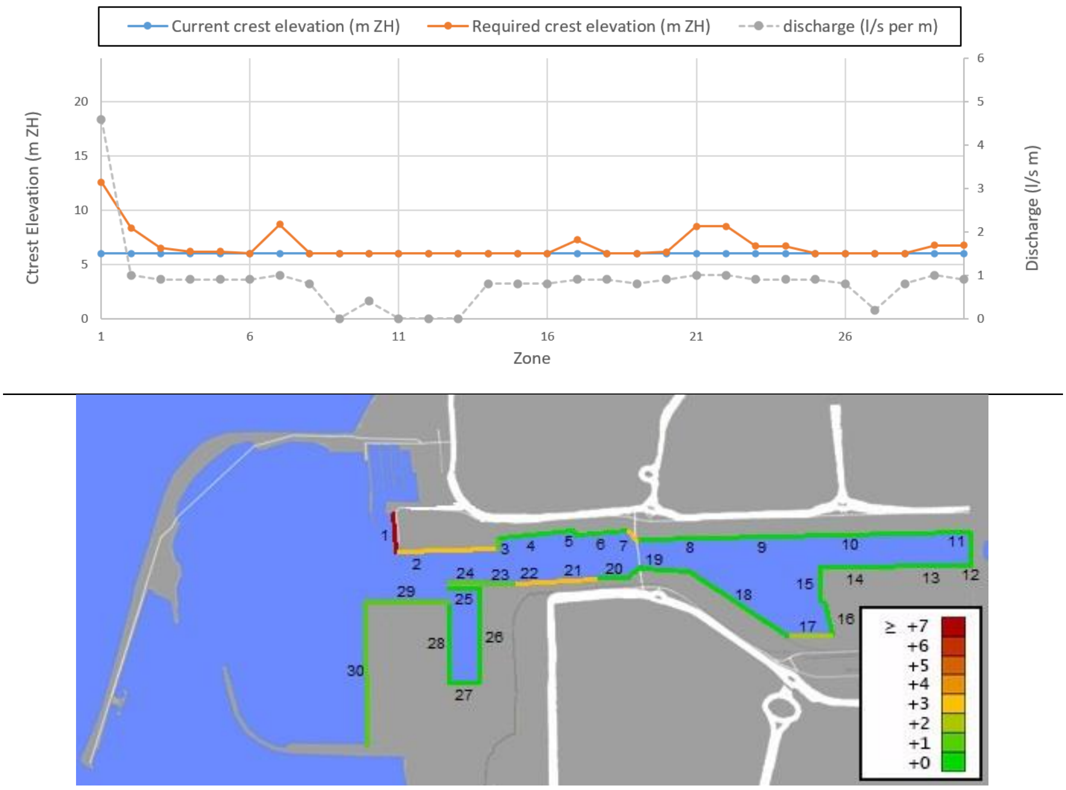


Table 21 - Mean Overtopping Discharge for 1 L/s/m limit (SLR=0.5m).

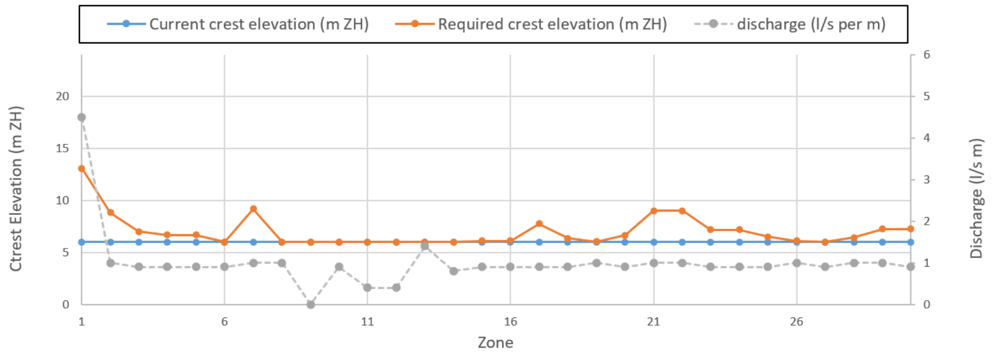
SLR= 0.5m



Table 22 - Mean Overtopping Discharge for 1 L/s/m limit (SLR=1m).

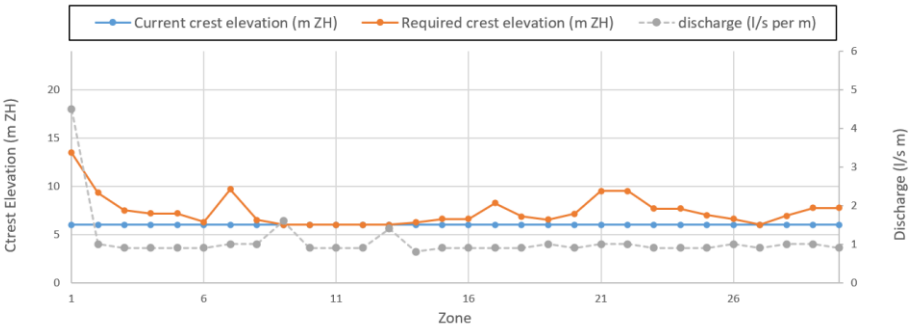
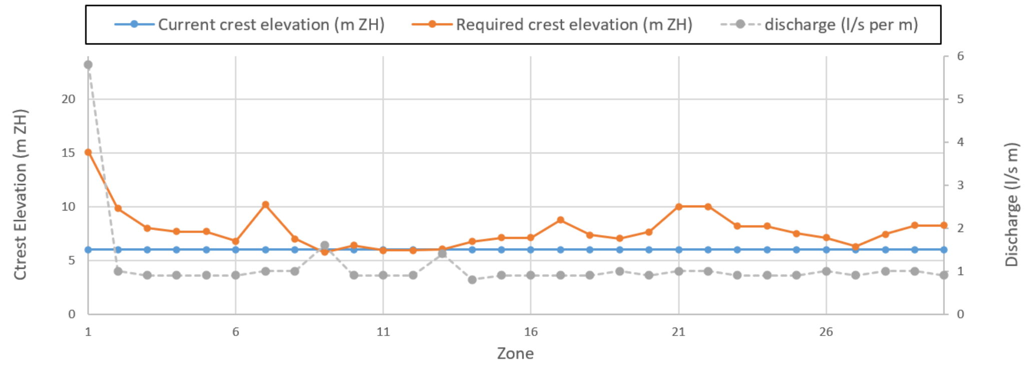
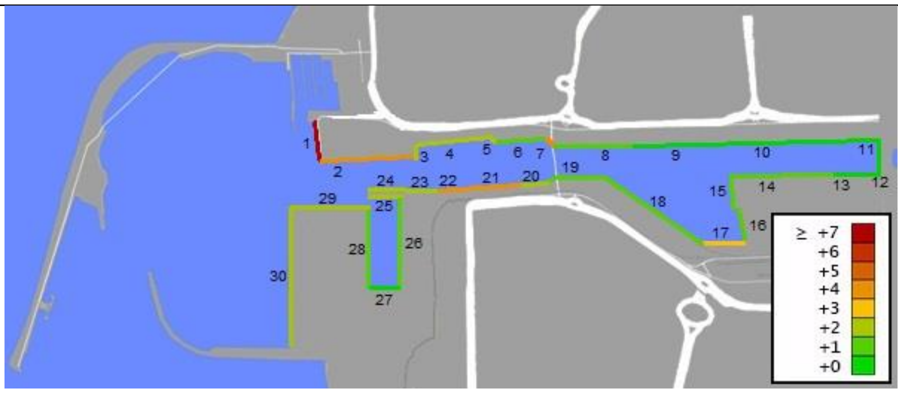
SLR= 1m



Table 23 - Mean Overtopping Discharge for 1 L/s/m limit (SLR=1.5m).

SLR= 1.5m





5.2.3. With Crest Elevation changes (5 L/s/m)

For the 5 L/s/m limit the same procedure as the 1 L/s/m was used, translating into the Table 24, Table 25,

Table *26* and Table 27 represented below. As it is expected, since the overtopping discharge limit is higher the values of required crest elevation will be lower than the ones calculated in the previously subchapter. The graphs on each table show a comparison between the values of q obtained for each zone and the value of the required elevation compared to the current elevation of each structure.

The results obtained made it possible to evaluate the presumed crest elevation that may be needed in the structures and to conclude which terminals are the most affected.

It is concluded that, for SLR=0 m (Table 24):

- Zone 1 is the most aggravated, needing a 5 m elevation increment to induce a discharge value under 5 L/s/m. This makes it very unlikely (even impossible) to apply this mechanism as a solution;
- Zones 2, 7, 21 and 22 would require 2 m more to be able to stop the overtopping, which in some cases, depending on the structure characteristics could be applied.

When SLR=0.5 (Table 25):

- Zones 1, 2, 7, 21 and 22 still maintain the same conclusions as for SLR=0 m, but with the addition of zones 3, 4, 5, 20, 23, 24, 29 and 30 which are slightly increased by 1 m.

From this point, with the value of SLR going up to 1 m (

Table *26*), the required crest starts to increase even more in more zones:

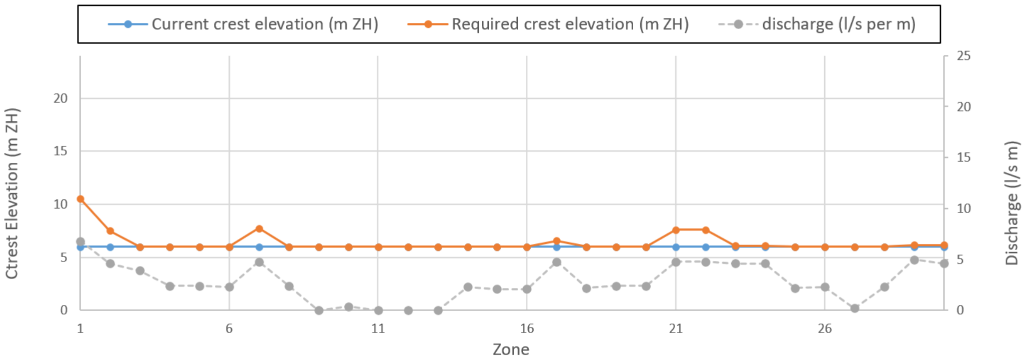
- Zone 1 requires an elevation increment of 8 m;
- Other zones that are highly influenced by overtopping, zones 7, 21 and 22 require 4 m of crest elevation;
- Zone 2 will face a 3 m increase while zones 3, 17, 23, 24, 29 and 30 would need 2 m more to counteract the overtopping;
- For zones 4, 5, 15, 16, 18, 19, 20, 25, 26 and 28 a smaller elevation of 1 m is needed.

The highest SLR (Table 27) will induce the highest required crest elevation, and more specifically:

- For zone 1 the necessary crest elevation has the highest value of 9 m;
- The second highest elevation of the crest (4 m) occurs in zones 2, 7, 21 and 22. For zone 17 a 3 m elevation is necessary;
- The zones 3, 4, 5, 20, 23, 24, 25, 29 and 30 present more achievable structural changes to the terminals with a required crest of 2 m elevation and only 1 m for the zones 6, 8, 14, 15, 16, 18, 19, 26 and 28.

Table 24 - Mean Overtopping Discharge for 5 L/s/m limit (SLR=0m).

SLR= 0m



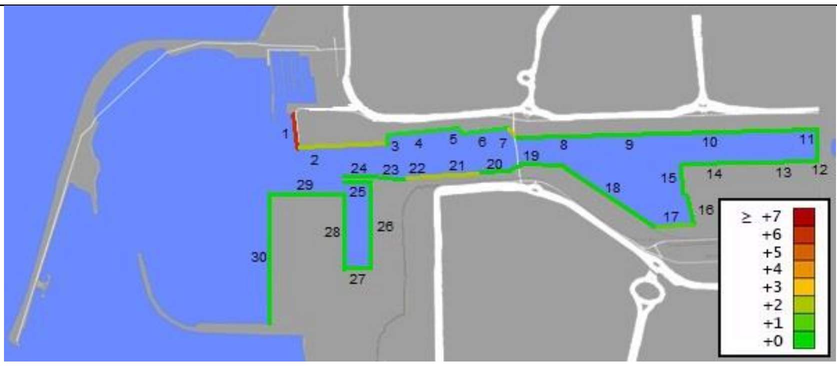


Table 25 - Mean Overtopping Discharge for 5 L/s/m limit (SLR=0.5m).

SLR= 0.5m

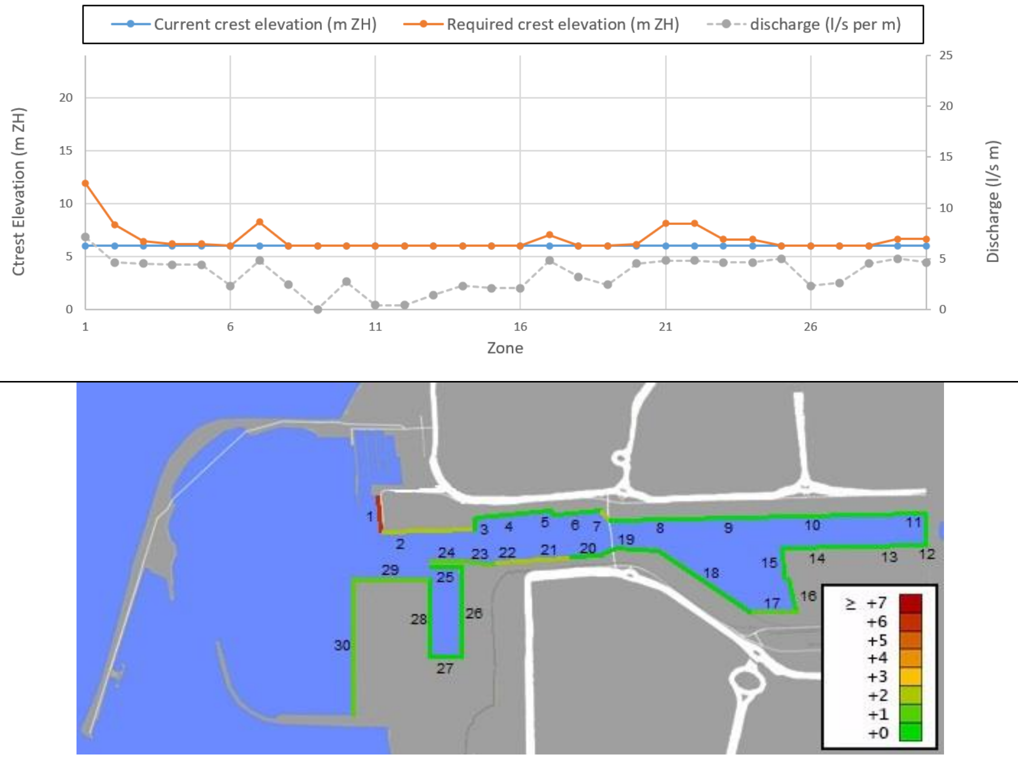


Table 26 - Mean Overtopping Discharge for 5 L/s/m limit (SLR=1m).

SLR= 1m

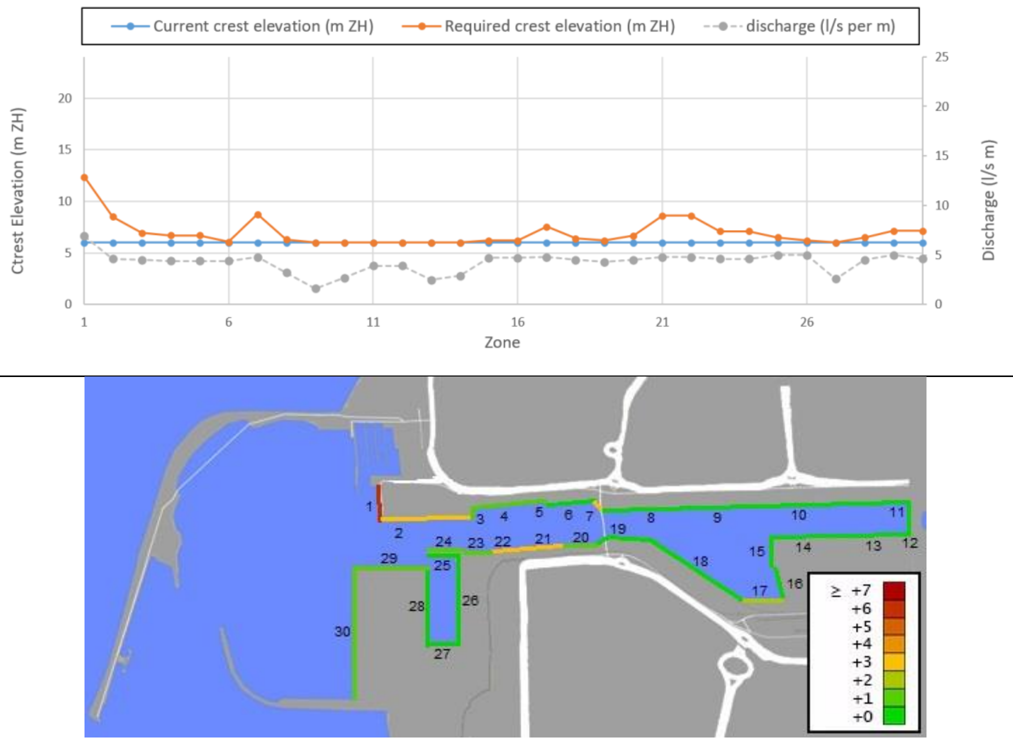
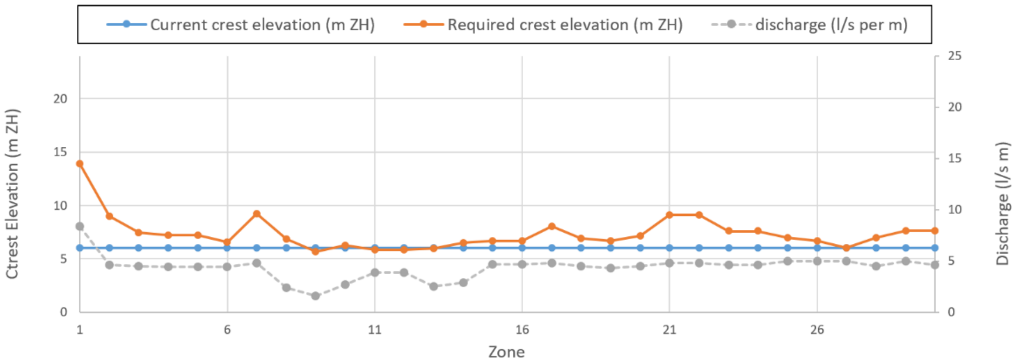


Table 27 - Mean Overtopping Discharge for 5 L/s/m limit (SLR=1.5m).

SLR= 1.5m



5.2.4. With Crest Elevation changes (current design limits)

On the matter of the current design limits, the same procedure as before was used, but in this case, since the limits were restricted to the design ones, the increments that will be obtained will consist of more realistic and approachable crest elevation. To analyze this scenario, naturally only 3 sea level rises were taken into account and for those, the following conclusions were obtained:

For SLR=0.5 m (Table 28), the only registered increase in the crest elevation was of 1 m in zones 1, 2, 7, 17, 20, 21, 21, 23 and 24.

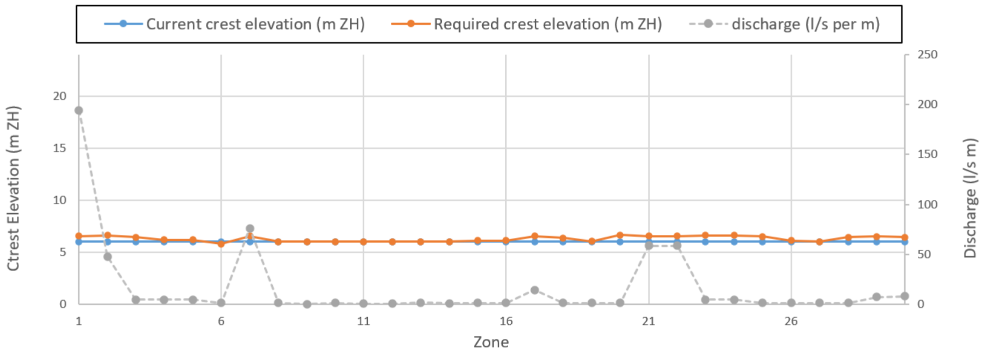
In the 1 m SLR scenario (

Table *29*), there is a 2 m elevation for zone 7, secondly, there are 1 m crest elevation increments in zones 1, 2, 3, 4, 5, 8, 15 to 26, 28, 29 and 30. For the rest of the zones no elevation is necessary.

For the last SLR (Table 30) there are still no crest elevations higher than 2 m, these are applied to zones 2, 7, 17, 20 to 25 and 29. For zones 1, 3, 4, 5, 6, 8, 14, 15, 16, 18, 19, 26, 28 and 30 the crest elevation would increase by 1 m. As for the rest of the zones no changes would be needed.

Table 28 - Mean Overtopping Discharge for current design limits (SLR=0.5m).

SLR= 0.5m



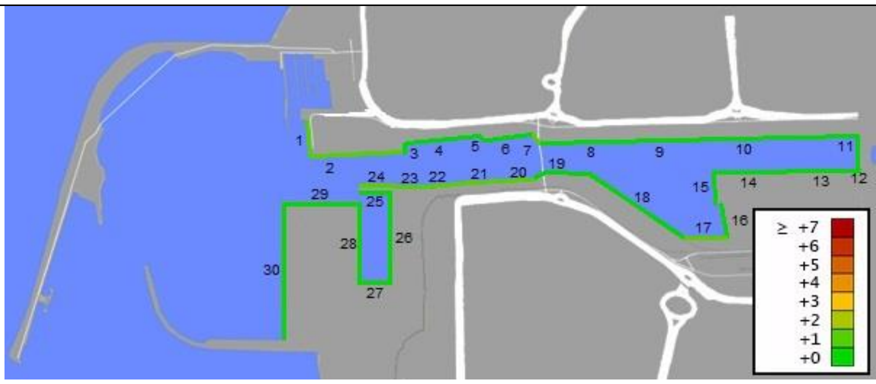
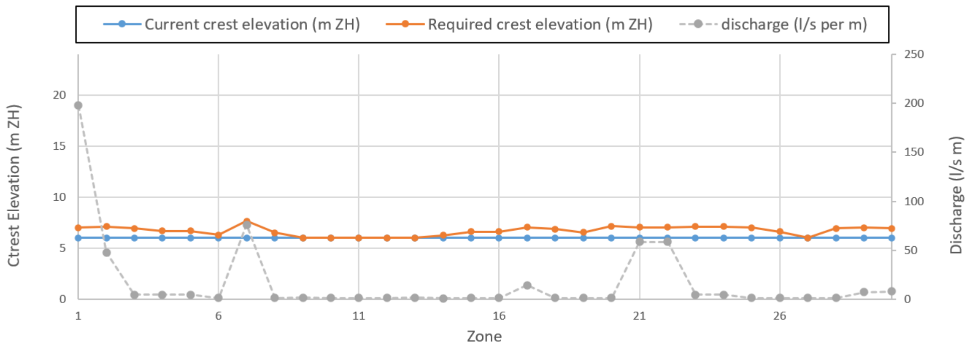


Table 29 - Mean Overtopping Discharge for current design limits (SLR=1m).

SLR= 1m



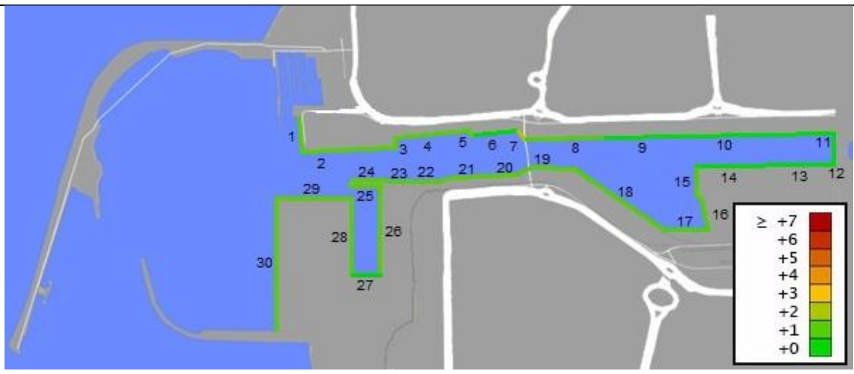
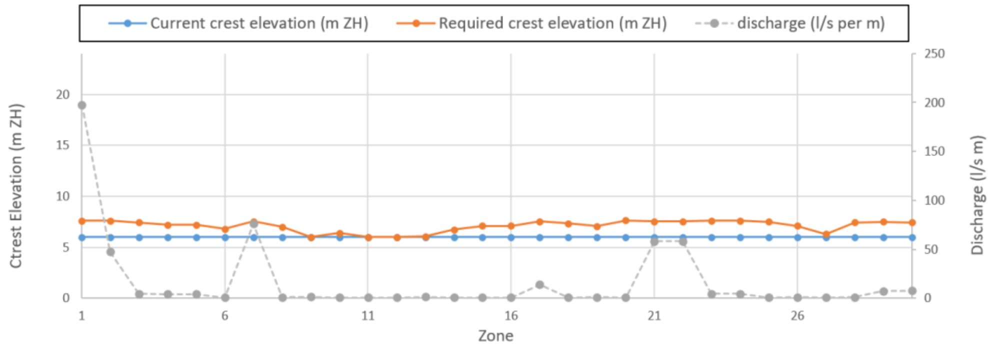
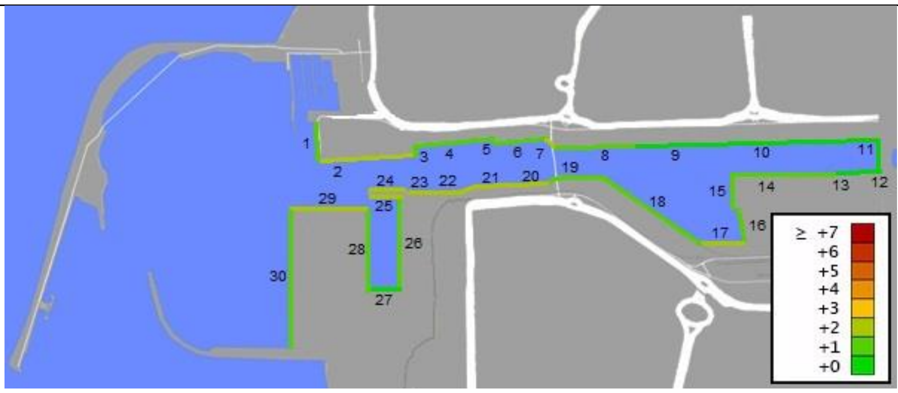


Table 30 - Mean Overtopping Discharge for current design limits (SLR=1.5m).

SLR= 1.5m





5.3. Measures

Unfortunately, this tool model cannot calculate the overtopping for some possible measures. But this fact does not depreciate that applying mitigation measures will have a positive impact on the overall operability and quality of the port, in fact, these results are used as a screening method to choose the best options to the port, and after that selection some calculations can be executed for further comprehension of possible mitigation measure.

Not all measures are compatible with the characteristics of the Port of Leixões but analyzing the results and based on previous examples and acquired knowledge it is possible to select the best options to be applied in this case.

The implementation of an Early Warning System will most likely occur just because of its price quality ratio and versatility, with the advantage of bringing good overall results and prepare the port for any inconvenience or disaster and consequent relocation of this goods.

As a complement to the EWS, the creation of a flood hazard map is also a possibility that can help to decide and discern which areas are more vulnerable, *i.e.*, which terminals need goods, vehicles or people moved from their hazardous location.

Another alternative would be to elevate. Although elevating the crest of the structures is a very effective measure, it can have complications since sometimes elevating the crest is not sufficient and it is necessary to raise all of the dock behind becoming an extremely expensive solution and, another important aspect that needs to be stated as a disadvantage is that elevating the terminals and docks may not be possible due to the heavy load in the foundations that such construction would entail. For example, for vertical walls structures it depends on the base, if the base is wide enough and the buttresses supporting the water impulse force are strong enough it would be possible to elevate the crest. This could be applied in the structures represented in Figure 33, Figure 39, Figure 42, Figure 43 and Figure 44.

However, there are other structures that are used in the port, diaphragm walls and open piled structures. These are more sensitive to this solution, most open piled structures (Figure 37, Figure 38, Figure 40 and Figure 41) wouldn't be able to withhold the weight of an elevation unless more foundation infrastructures were built or the quay was extended forward with new foundations and the required crest elevation which would lead to the loss of area in the navigation channel. The structures composed by diaphragm walls in this case are complemented with ground anchors or tie bars that strengthen the construction (Figure 34, Figure 35 and Figure 36). So, although there is a possibility that it could hold the weight, most likely (and as a safety measure) it would need more anchors or more steel sheet diaphragms to surely reinforce the terminal.

One elevation measure that can most certainly be used in this case would be to place an empty first container on every column of containers, therefore adding approximately 2 more meters of security in case of flooding or wave overtopping. The empty container would elevate the rest of the containers (which store goods) keeping them 2 m above the terminal crest. On the downside, this can only be applied in terminals that lodge containers, in the case of dry bulk terminals it would not be effective.

Creation of a wave return wall is also a possible structural measure that can be applied specially to the crest of the northern terminal (Zone 1 and 2). This structure is situated at the top of the vertical wall and returns the waves seawards, reducing overtopping (Van der Meer *et al*., 2018). In order to calculate how this structure can influence the overtopping discharge, there are formulas provided by EurOtop, that enable the design of these structures.

Following the design instructions displayed in Figure 49 and Figure 50, two possibilities were tested to compare how this measure can affect overtopping in every SLR scenario, the first was B_r and h_r take the values of 1.5 m each and the second 2 m each. The results are shown in Table 31 and represent the percentage of mean overtopping discharge (q) that is reduced from the original calculated one.

After the analysis of the results, it is possible to conclude that this measure would be indeed useful to counter overtopping in this terminal.

It is also important to understand the advantages and disadvantages of some measures that could be applied or not. For example, a storm surge barrier wouldn't be effective in this case, not only would be an extremely expensive measure but also it could be even more harmful for operability than the possible overtopping of some structures since it could close the port more times than when compared to tolerable overtopping. A storm surge barrier requires a lot of conditions to be met such as a compatible port layout and a certain water depth, with these many obstacles this measure would not be suitable in this case.

Table 31 - Percentage of mean overtopping discharge value reduction of a terminal without a wave return wall

Values of B_r and h_r	Present Situation	SLR=0.5m	SLR=1m	SLR=1.5m
1.5	-35%	-24%	-12%	-5%
2	-38%	-28%	-15%	-13%

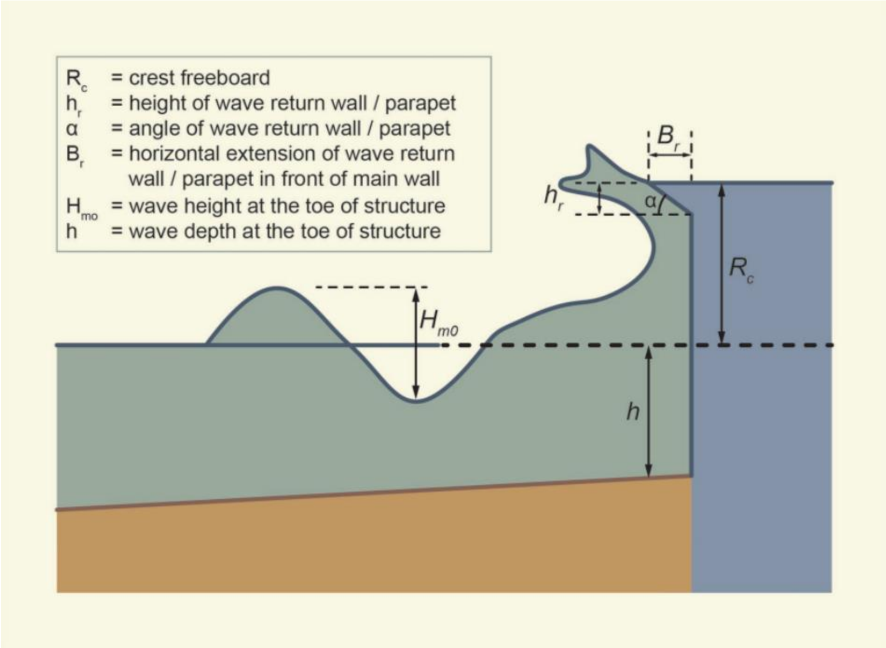


Figure 49 - Parameter definitions for structures with a wave return wall (EurOtop, 2018).

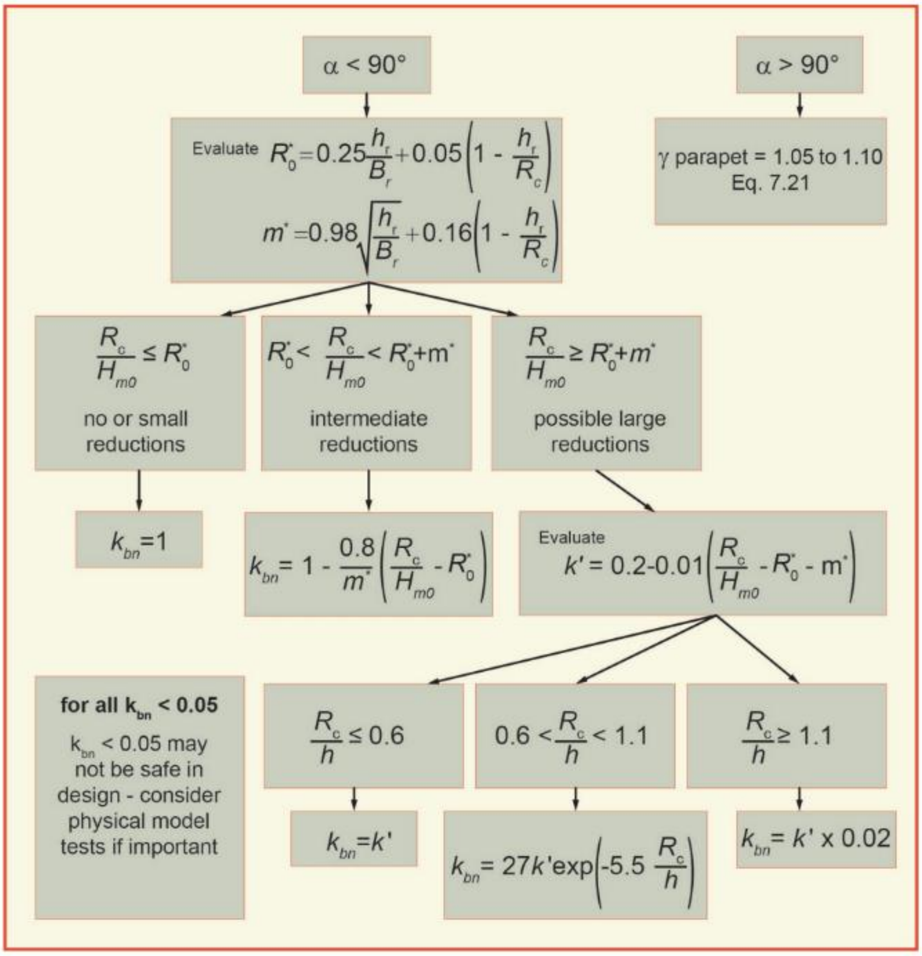


Figure 50 - Decision chart summarising the methodology for a wave return wall design guidance (EurOtop, 2018).

6
CONCLUSION AND FUTURE DEVELOPMENTS

6.1. CONCLUSIONS

In the port of Leixões, other studies have already been carried out concerning the assessment of the risk of overtopping, but always focused on the external structures and on the operability and safety of terminals closer to these structures. This study is the first to approach this theme in terms of assessment of the risk of overtopping in the internal structures of the port. For this, the methodology characterized in chapter 4 was used with the objective of evaluating the risk of overtopping and determining the possible measures to be applied. It is very important to note that despite all the care that was taken in the characterization of the physical processes, the analysis can always be incomplete, both due to the simplification of the processes in the calculations used and due to certain elements that were not taken into account in the methodology used, for example, the port resonance and seiches that can cause even more distortions in the overtopping results. Another point that also needs to be taken into account and that may have caused certain discrepancies in the results are the empirical EurOtop formulas used by the tool to calculate overtopping. These formulas have limits and different regimes associated with them, and when the data is applied on the boundaries of these limits the results may end up higher than expected. For example, in vertical quay walls it is possible that the conditions vary between impulsive and non-impulsive regimes, which in the end can represent some overestimation on the results.

Firstly, the maritime conditions recorded in the port of Leixões were characterized for the parameters of wave height, wave period and wave direction, characterizing the sea state. In addition, the tide level was also described and analyzed using the T-Tide tool that allowed the separation of the astronomical and meteorological components and then the extreme regimen analysis of the meteorological component.

Then, the sea agitation near the inland structures was characterized. For this it was necessary to define the different possible variables that would affect this component, namely the locally generated wind and vessel waves and agitation produced by the penetrating offshore waves. These results showed that the wave height inside the harbor is influenced essentially by the agitation caused by outside waves that impact the inner area. This fact is explained by the angle of incidence of the offshore waves, that affect this situation when their direction is from the south and southwest, which, despite their (usual) smaller size when compared to other directions, find almost no obstruction from the external defenses when entering the port.

Subsequently, overtopping flow was determined using a MATLAB® tool that compiles and applies empirical formulas to calculate overtopping. The structural characteristics of all terminals and inner docks (Zone 1 to 30) were studied, where it is also important to mention that due to lack of data for some zones (due to future changes in the port plan) simple models of vertical terminals were used.

Regarding the results obtained by the tool, it was found that zones 1, 2, 7, 21 and 22 were those that recorded the highest overtopping values for the situation of current crest elevation, especially for the scenarios of sea level rise 1 and 1.5 m where there were indeed large overtopping values. In the SLR=1.5 m scenario, values of 400 L/s/m and above were registered, particularly in zone 7 (ramp) where almost 1000 L/s/m were calculated. In the case where changes were already made to the crest elevation values and limits of 1 L/s/m, there were a lot of changes even on SLR=0 m, where the required crest elevation in zones 1 and 2 would reach 12 m and 8 m respectively. In zones 7, 17, 21 and 22 some significant changes are also made.

Naturally, the mean discharge and the required crest elevation in these zones will worsen as the sea level rises, registering levels of 14 m in zone 1 and 10 m in the other zones mentioned above. It is important to note that many areas (especially inward ones) suffer little or none with these sea level rises.

Next, the situation with a 5 L/s/m limit was analyzed, and similar, but less drastic, results were obtained. The critical zones would still remain the same but have lower required elevation values. As a comparison, for SLR=0 the required crest elevation for zone 1 was 10 m and 7 m for zones 2, 7, 21 and 22. These zones remain the most affected throughout the sea level rise scenarios even reaching crest heights of 14 m (zone 1) and in many zones 7 m and above.

Finally, using the current design limits, very optimistic and even more realistic results were obtained in relation to possible situations of an increment of the crest elevation. And what these results represent is that if it is in the interest of the port to maintain the current design limits for future changes in sea level, raising the crest level (in some cases) would certainly be a wise measure, especially in zones where the calculated required crest elevation wouldn't surpass the 2/3 m value.

Therefore, possible measures to be applied in these cases were evaluated. Since, for example, raising the level of the north container terminal (zone 1 and 2) would never be feasible due to the excessive calculated required elevation it would need, one possible solution would be to build a wave return wall that through the explanations showed in chapter 5 could lead to overtopping reductions around 15%~30%, which is an interesting reduction. Other measure would be the creation of an early warning system (EWS). And lastly the insertion of an empty container at the base of all columns of containers that contain goods, avoiding the damaging of these in a situation of overtopping or overflowing, this is an easy and cheap option, with feasible execution and whose only disadvantage is that it can only be used in container terminals.

In summary, the development of this work allowed the understanding of the importance of the assessment of the risk of overtopping on inland port structures and the possible measures to be taken to mitigate these risks. Similarly, the methodology used for this assessment allowed to effectively evaluate overtopping phenomena and the possible adaptation measures, essential for future decision-making and port management on this subject.

6.2. FUTURE DEVELOPMENTS

As future work that could be developed and executed, it is proposed the creation of an early warning system (EWS), allowing the prevention of future undesirable situations and future economic and human losses. In order to do that it would also be interesting to analyze how this measure would be applied in Leixões.

Relatively to other suggested accommodation measures, elevating the containers on the multipurpose terminal through the application of an empty container located at the base of every column of cargo storage containers could also be utilized for further protection in case of adverse environmental conditions.

Based on the quantitative results previously obtained in this dissertation, it is also important to consider the placement of a wave return wall in the north terminal in case new studies predict unexpected increments in sea level rise which raise the risk of overtopping.

Finally, a study is also proposed to analyze the quantitative impact of all the qualitative measures that could be used by the port of Leixões in the case of sea level rise.

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