Evaluation of the structural behaviour of the concrete slab of a dock

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Abstract

This dissertation has the main objective of performing the evaluation of a naval dock bottom slab structural behaviour. Considering the scope of the dissertation and the limitations relating to the development of the study to be carried out, the analysis will focus on one of the panels representing the dock´s bottom slab structure of the dock. For the type of structures concerned the soil-structure interaction presents an important influencing factor in the structure behaviour. In this way, the calibration of the foundation parameters is first performed through iterative methods, finalizing the analysis when the values of the deformed measurements in the "in situ" tests approximate those measured in the structural model. Calibrated the system, the analysis in study and the maximum load that the slab supports evaluation are carried out, considering the behaviour of bending and shear forces. In the evaluation for shear forces, a sensitivity analysis is performed on the soil and structure parameters, to verify its influence on the slab response and to define the reference rigidity for the structural analysis, using it as a basis for the evaluation of the maximum load carrying capacity for the different types of ships analysed. In the case of the evaluation for bending moment, the analysis is carried out only in the case of the larger ship, since it is verified that the conditioning force would be shear, exemplifying the influence of the different types of analysis when the structure is conditioned by bending behaviour.

Key Words: Dock, Bottom Slab, Structural Analysis, Soil-Structure Interaction, Sensitivity Analysis, Shear Force, Bending Moment

1. Introduction

The main objective of the dissertation is the evaluation of the structural behaviour of the dock´s bottom slab under the effect of the loads transmitted by ships. The study will focus on assessing the maximum capacity of the slab in relation to shear and bending moment forces. Considering that the structure's response to ship loads is strongly influenced by the soil-structure interaction, a sensitivity analysis to the parameters of rigidity of the soil and structure is carried out. The maximum capacity of the structure is evaluated based on different assumptions of the evolution of the characteristics of the soil over time, starting from the first campaigns of geotechnical prospecting carried out by Teixeira Duarte, Lda., in the 1970.

Structure in study

The study lies on the dock 21, specifically on Panel 11, because it´s where the ship´s motors are normally located when it´s docked on the structure, allowing a more thorough analysis of the problem.

Panel 11 consists of three longitudinal beams, the centre beam, located in the central strip of the bottom slab, 12 metres wide and 1.6 metres thick; The lateral beam is 8 metres wide and 1.4 metres thick; The regular zone, located between the lateral beam and the wall foundation zone, 13.5 metres wide and 1 metre thick, laid out in the figure 1.

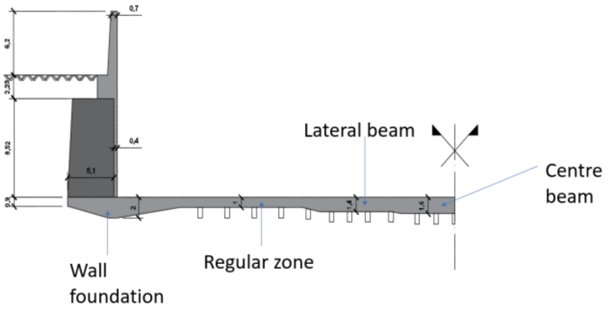


Figure 1. Dock section: type of panels 1 to 11.

Foundation

The foundation of the panels is divided into 2 zones: direct foundation in the peripheral walls and mixed foundation through reinforced concrete piles with a diameter of 0,52 metres and direct foundation in the soil, in the central area of the slab. The use of piles in the central zone has the main function to resist to tension forces, caused by the hydrostatic impulse created by the water existing in the soil when the dock is empty. If a ship is found inside, the loads are generally divided by the piles and the foundation of the slab directly on the ground.

2. Background

The importance of a good geotechnical study is high, often placed a side in relation to a concrete structural analysis. The dock in study is characterized by being a slab laid on the soil, indicating that the soil will have an important relevance in the structural behaviour of the structure. The efficiency of the structure is directly linked to the mechanical characteristics of the ground in which it is founded, being the factor that most influences the maximum load that the structure has capacity to withstand.

2.1. Geotechnical characteristics

2.1.1. Tests performed

In the 1970´s, during the structure´s construction, several campaigns of geotechnical prospecting were carried out, with the intent of determining the type of terrain where the dock would be constructed. During this campaign, two types of tests were executed: CPT test and loading test. We concluded from the CPT test that it´s a soil with good compactness, with peak resistance values (Rp) between 50 kg/cm^2 and 250 kg/cm^2 for depths between 1.0 and 12.0 meters around the pilot piles and between 120 kg/cm^2 and 250 kg/cm^2 on the bottom area of the dock (Teixeira Duarte, Lda, 1975). Based on the previous test, to calculate the rigidity of the piles the following force-deformation values are adopted, referring to the ultimate load supported by the piles:

Table 2. The result of the load tests for the 12 meters pile.

Tests – Piles 12 metres	
Compression	
Load	2500 kN
Deformation	3,35 mm
Tension	
Load	1300 kN
Deformation	7 mm

Table 1. Load values for the 8 meters pile.

Tests – Piles 8 metres (estimated values)	
Compression	
Load	2500 kN
Deformation	4,5 mm
Tension	
Load	1300 kN
Deformation	10 mm

Note: The values for the 8 metres pile were obtained by estimation from the results of the 12 metres pile, given the lack of information concerning tests on the 8 metres piles.

2.1.2. Deformability Modulus

The soil´s deformability modulus (E), or elasticity modulus, is a parameter that defines the rigidity of the soil. Obtaining this parameter can be done by correlation with the results of the CPT test. This correlation is characterized as $E = \alpha * R_p$, for sandy soils, where R_p is the peak resistance in (kg/m^2) and α represents a parameter that varies between 1.5 and 3, depending on the type of soil. (S. Coelho, 1996)

Given the good characteristics of the soil´s regularity and compactness, the parameter α was set to 3. Admitting as peak resistance the value of 200 kg/cm^2, thus concluding that the deformability module calculated through the correlation with the results from the CPT test are: $E = 3 * 200 = 600 \frac{kg}{cm^2} = 60000 \frac{kN}{m^2}$.

2.2. Type of foundations

Two types of foundation were defined in the design: direct and pile foundation.

The simulation of the interaction between the structure and the soil is performed through deformable supports with linear elastic behaviour (J. Santos, 2008) in order to find the best possible behaviour of each type of foundation. For the case of the foundation by piles these must present rigidity to tension and compression and for the case of the direct foundation only presents compression rigidity.

2.2.1. Pile Foundations

The deformable supports used to simulate pile foundation uses the concept of Reaction Modulus (k_s), defined by Poulos, 2018 as a convenient way to represent the behaviour of the soil through springs. The definition of the reaction modulus, in this case, is done based on the results from the CPT tests. Knowing the load capacity and the deformation of an isolated pile we can correlate them to obtain an estimate of the module with some level of confidence (N. Barounis, 2016). Using the concept of the reaction modulus, the rigidity of the spring (k_s) is defined as the ratio of applied pressure (q) and spring deformation (y), equation (1). (Vesic,1961)

$$kse = \frac{q}{y} \qquad (1)$$

In this way, it is possible to determine the rigidity associated with an isolated pile, remembering that the structure in question is based on several piles, concluding that the rigidity will be affected by the interaction of pressure bulb of the group, reducing the overall stiffness of the structure.

2.2.2. Direct Foundation

In the direct foundation zones, the structure is laid directly on the soil. To simulate this behaviour, it was considered the beam model proposed by Winkler (1867), suggesting a soil simulation through a series of independent springs, with elastic linear behaviour.

The rigidity of the springs is defined by the concept of the reaction modulus (k_s), discussed earlier, but in this case with application on the Winkler´s beam. Bowles, (1995) proposed a hypothesis based on the formulation of Vesic, where the reaction modulus is determined considering intrinsic parameters of the soil, through equation (2):

$$k_{sd} = \frac{Es}{(1-\nu^2)} \qquad (2)$$

Where (E_s) represents the soil´s deformability module and (ν) the Poisson coefficient. Fundamentally, the method of Winkler consists on the definition of the foundation as a beam supported by an elastic soil, where the supports are simulated with infinitely close independent springs (J. Santos, 2008).

2.3. Material characteristics

Bearing in mind that at the time of the project, the regulation in force was the *Regulamento de Estruturas de Betão Armado (REBA)– Decreto 47723 de 20/05/1967*, it is necessary to consider the characteristics of the materials presented in that regulation. The materials considered were the B225 concrete and the A40 SND steel, presenting the following characteristics:

Concrete: B225

- f_{cd} = 13 MPa
- E_c = 29 GPa;
- f_{ctm} = 2 MPa

Steel: A40 SND

- f_{syd} = 348 MPa
- Es = 210 GPa

The analysis will be carried out using the current standard, Eurocode 2 (EN 1992). The materials used have characteristics equivalent to the C20/25 concrete and the A400 NR steel.

Moment-Curvature diagram

The behaviour of a concrete member can be represented by the moment-curvature diagram. This diagram is divided into 3 states: elastic, cracking and steel yielding. In the initial loading moments, the section is in the elastic state (state I), a phase where there is no cracking, ensuring the resistance by the concrete. After reaching the cracking moment (A), the resistance of the concrete is not enough to withstand the actions and the first cracks begin to be formed, reaching the cracking state (state II). This phase is characterized for non-linear behaviour, with a high loss of rigidity soon after the opening of the first crack, depending its behaviour on the type of reinforcement inserted in the section. State III stands out from state II by the increase in curvature, a consequence of the loss of stiffness felt after reaching the yielding point (B). After reaching the yielding point, the moment-curvature diagram reaches a plateau where large increases of curvature occur at practically constant moment.

Illustrated in figure 2.

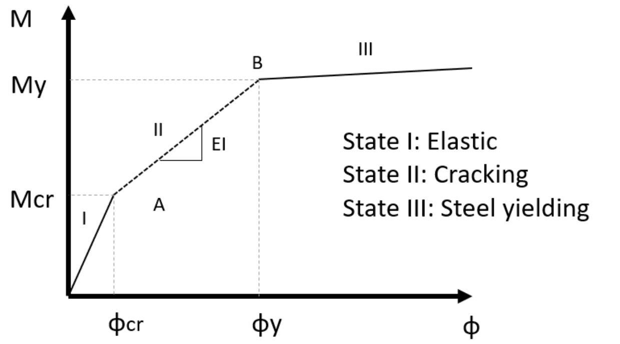


Figure 2. Moment-curvature diagram of reinforced concrete section.

Unaware of the slab´s cracking state, it should be considered, conservatively, that due to the numerous variations of load to which the slab was subject, it may present cracks, implying a possible reduction in the rigidity of the bottom slab. For that, a reduction of 50% of the slab´s rigidity is defined as the first approximation for the sensitivity analysis of the soil parameters.

2.4. Project actions
2.4.1. Ground action

The action of the terrain on the structure is represented by the form of soil´s impulse. This impulse is the designation given to the occurrence of lateral pressures induced by the terrain on the structure, in this case, in the back of the lateral wall and in the upper face of the foundation.

2.4.2. Hydrostatic action

The hydrostatic action corresponds to the pressure exerted on the dock by the water.

The consideration of groundwater level at low tide and high tide wouldn´t be advantageous, so it will be considered a medium level, between the two.

2.4.3. Ship Action

The action of the ship represents the ship´s load on the bottom slab when it is inside the dock. The transfer of weight is made through prefabricated concrete blocks.

Figure 3. Docked ship on dock 20.

3. Structure modelling
3.1 Terrain modelling and definition of the order of actions

The soil´s parameters are simulated in two ways: in the direct foundation zone it is considered a series of springs with elastic behaviour with rigidity to compression and in the foundation through piles the modelling is done through link elements, with non-linear elastic behaviour, where there are two sections referring to compression and tension rigidity. Rigidity is introduced in the form of a force-deformation diagram, according to the pile results obtained by the CPT test, present in tables 1 and 2.

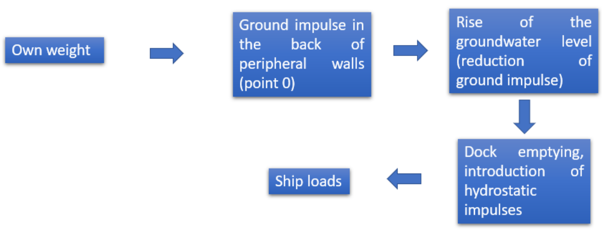
The definition of the number of piles in each alignment is clarified by analysing the plan of implantation of the piles, coming from the initial project, indicated in the following table the type of pile for a 4 metres section:

Table 3. Number and type of piles in each alignment

Alignment	Number of piles on a 4 (m) section	Type (s)	
1	1	12 m	
2	1	12 m	
3	1	12 m	
4	1	12 m	
5	2	12 m	
6	2	12 m	
7	2	12 m	
8	2	12 m	
9	2	12 m	
10	2	12 m	8 m
11	2	12 m	8 m
12	2	12 m	8 m

Sequence of actions

The structure is loaded by various actions from different sources and temporal occurrences. The consideration of the actions with simultaneous acting would cause the behaviour resulting from the model to not represent the actual behaviour of the structure, therefore, the consideration of the action order of the loads is extremely important. To do this, one must understand the way in which they operate to complete a valid order based on occurrences to which the structure was subject. Briefly it is represented the order of the actions in a schematic form:



Note: The reporting of loading cases and the placement of the deformation measuring apparatus is done by J. Fernandes & R. Correia, in the article "*Structural Behaviour of a dry dock with Pile-Anchored bottom slab*", dated 1980.

The measurements of the deformations presented in the following section, refer to predetermined points, shown in figure 4.

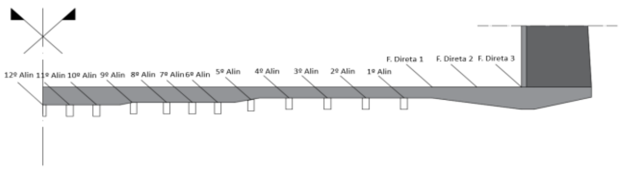


Figure 4. Characterization of the measuring points of the deformation on panel 11.

3.2. Modulation frame vs shell

The importance of comparing the two models lies in the differences inherent to each type of modulation: frame or shell. Concluding that modelling through frame may be more advantageous, because it is a simpler formulation allowing the obtention of results in a direct way, instead of integrating the tension on the section´s width, like it´s done on the shell modulation. Figure 5.

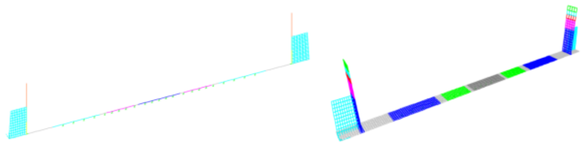


Figure 5. Slab modulation with shell and frame elements.

By analysing the deformed models of the two modulations: frame and shell, it turned out that the structure's response in the two cases are similar, allowing us to conclude that, from the point of view of the deformation, both models are valid and offer similar results. Evaluating the positive and negative aspects of both formulations the choice relies on the use of the frame section with 4 metres wide.

3.3. Calibration of soil parameters

To allow the best possible definition of the soil´s rigidity, a retro analysis is carried out based on the deformations measured by LNEC, where in an iterative way, the rigidity input parameters of the springs are varied to approximate the deformed measurements in the model with the measurements in the tests. First analysing the rigidity of the direct foundation and then calibrating the reduction coefficient of the rigidity of the piles, promoted by the interaction of the pressure bulb, starting from the rigidity of the isolated pile.

3.3.1. Direct Foundation

The following table shows the input parameters in a succinct manner to obtain the reaction module, as well as the values obtained by J. Fernandes & R. Correia in their analysis.

Table 4. Comparative analysis of the parameters for obtaining the value of K_{sd}.

	Author	J. Fernandes & R. Correia
E(kN/m^2)	60000	90000
Coef. Poisson	0.3	0.3
If	2.5	2.5
b (m)	10	6
K_{sd} (kN/m/m^2)	3000	8000

The definition of intrinsic values relating to soil parameters are complicated, being of the discernment of each author to check whether these conform to reality or not, in this case it was considered a more conservative analysis where it is accepted as first analysis, a rigidity for the soil of 3000 kN/m/m^2. Concluding that the rigidity of the first iteration is considerably lower than the actual, since the impulse felt in the back of the peripheral walls, assisted by the lack of rigidity of the soil, promotes an excessive rotation of the foundation in to the interior of the dock. The next step on the iterative analysis is the increase of the k_{sd} value, defining it on table 6.

Table 5. Rigidity parameters used in different iterations.

Iterations	Value of k_{sd} (kN/m/m^2)
1	3000
2	20000
3	30000
4	64000

The following figure shows the results of the above iterations.

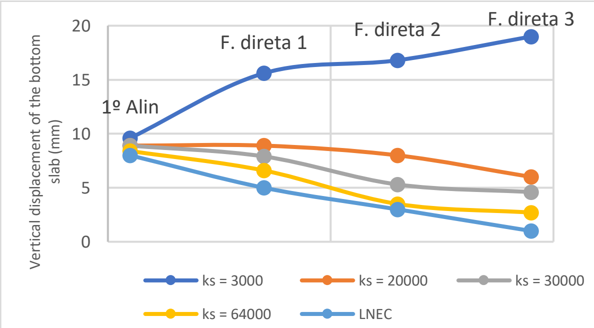


Figure 6. Deformation of the bottom slab referring to the different iterations.

As expected, the increase in soil´s rigidity approximates the deformation measured in the model with the LNEC measurements, concluding that the final soil´s rigidity is 64000 kN/m/m^2. Although the deformation is not exactly equal to the measurements in the tests, it is considered that the error associated with the difference between the two deformations, referring to the k_{sd} = 64000 kN/m/m^2 and the values measured by LNEC is insignificant and will not cause future problems.

3.3.2. Foundation by Piles

Defined the location and number of piles in each alignment, an iterative analysis for the rigidity reduction parameter of the piles is carried out due to the interaction of the pressure bulb. In a similar way to the direct foundation, values referring to the first iteration were arbitrated and the changes were made in order to match the deformations determined in the model and those obtained in the tests, limiting the reduction factor between 1 and 7, i.e. the reduction will be made by dividing the rigidity associated with the isolated pile by factor reductions until the expected results are reached.

In the following phases of the iterative analysis, the individual increase in the reduction factor is affected by each pile, i.e. the reduction parameter varies based on countless factors, such as proximity between the pile and the number of piles in a group. This way, and observing the deformations showed in the test, it is concluded that the factor reduction will be greater in the central and smaller on the side alignments, representing in the following table and figure, the results of the iterative process.

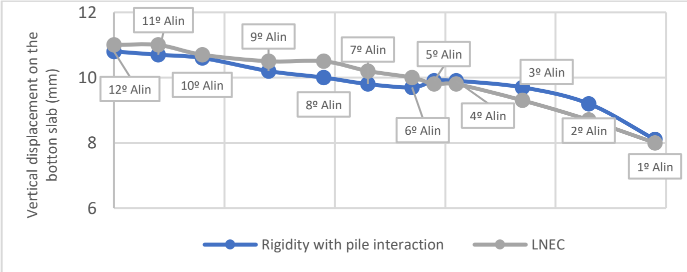


Figure 7. Deformation referring to the last iteration compared to LNEC.

Table 6. Comparison of the reduction coefficient in the first iteration and the last one.

Alignment	First iteration	Last Iteration
	Reduction coefficient	**Reduction coefficient**
1	1	1
2	1	1
2	1	1
4	1	2
5	1	3
6	1	4
7	1	4
8	1	4
9	1	4
10	1	4
11	1	5
12	1	5

The results presented on table 7 show that the rigidity of the piles located in the central alignments (12th and 11th) suffer a greater reduction due to the lower spacing between the piles than in the lateral alignments. Noting that the main aspect that leads to the reduction of rigidity is the proximity between piles.

3.4. Setting and load location

3.4.1. Case 1 – Large ship

In this case, it is intended to analyse the stress created by a large ship, for example a cargo ship. Given their large dimensions, we defined the location of the concrete blocks along the entire width of the bottom slab, as follows:

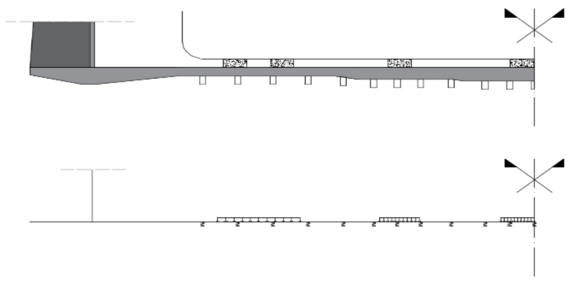


Figure 8. Concrete blocks and load disposal of for case 1.

The evaluation of the ultimate load in case 1 has enormous relevance for the study in question, due to the consideration of the largest ship to which the slab may be subjected.

3.4.2. Case 2 – Small/Medium dimensions ship

Case 2 simulates the load of a small/medium sized ship. Defining the load disposition for the two cases as followed:

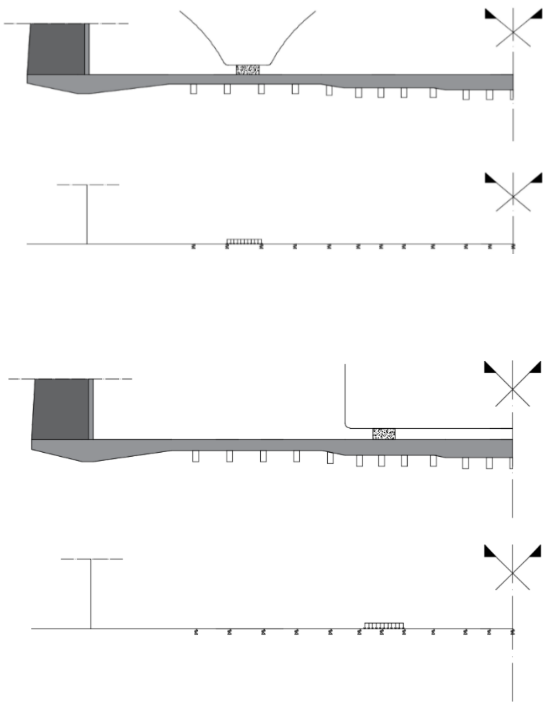
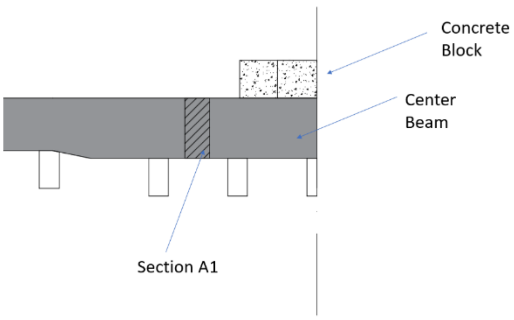


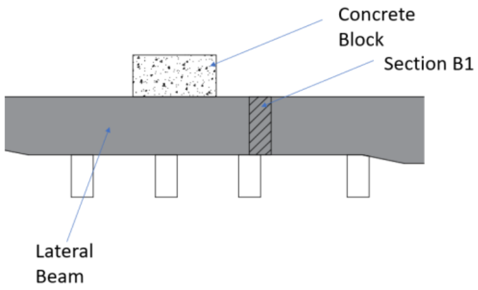
Figure 9. Concrete blocks and load disposal of for case 2.

To carry out the ultimate load analysis, it is considered the cracking state adopted for Case 1.

4. Structural capacity for shear force

The purpose of this section is the assessment of the ultimate load, resisted by the bottom slab, for shear force. In that case critical sections were defined to verify the safety of the shear force, demonstrated in figure 10.





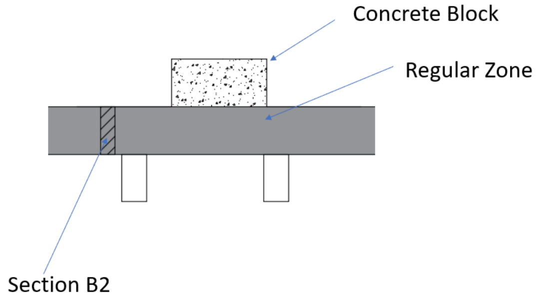


Figure 10. Definition of the analysis zones for the vatious load cases

The section analysis called A1 in the figure 10, is related to Case 1, or in the case for a large ship, aiming at determining the maximum load on the centre beam, sections B1 and B2 are related to case 2, or to the case of small and medium sized ships, to determine the maximum load on the lateral beam and on the regular area. Critical zones are defined at a distance $d/2$ of the concrete block, this way considers the load portion which is transmitted directly to the support, in this case to the piles.

4.2. Case 1 – Large ship

To determine the maximum load, we considered various types of loads, shown in table 8, coincident with the placement of different sized ships. The gradual increase of the load allows a representativeness of the effects in the slab.

Table 7. Types of load used in the assessment of the maximum load.

Case 1			
Type	A (kN/m)	B (kN/m)	C (kN/m)
1	1250	312,5	104
2	1500	375	125
3	2000	500	167
4	2500	625	208
5	3000	750	250
6	4000	1000	333
7	5000	1250	417
8	6000	1500	500

Before starting the analysis, considering the modification of parameters such as soil or slab stiffness, we analyse the influence that the two types of foundation: direct and by piles, face to a degrade of the calculated conditions determined earlier. For this, 3 test cases are considered: the reference case (I), the case where the rigidity of the direct foundation is reduced (II) and the case where there is loss of rigidity in the piles (III). Comparing the three cases, we conclude that the critical foundation is the direct one, in this case a deterioration of the initial conditions will have enormous influence in the result. On the other hand, a variation in the rigidity conditions of the foundation by piles do not influence the result. To understand this conclusion, we remember the purpose of the implementation of piles in the structure: resistance to tension forces induced by the hydrostatic impulses on the bottom slab. The implementation of piles is not very influential in the case of compression forces, these are fully resisted by the direct foundation.

Sensitivity analysis to the rigidity of the soil

In the sensitivity analysis to the rigidity of the soil, the change of rigidity of the direct foundation will be affected to interpret its influence on the final behaviour of the structure, varying between 20% and 130% of the rigidity calibrated in section 3. The slabs rigidity used in this initial phase is equal to that considered in the influence between the direct foundation and the piles, 50% of the initial rigidity. The rigidities considered in the sensitivity analysis are those defined in table 8.

Table 8. Summary of the rigidity values used in the sensitivity analysis of the terrain parameters.

Percentage in relation to the value defined in section 3	Rigidity (Value of k_{sd})
20%	12800 kN/m/m^2
50%	32000 kN/m/m^2
100%	64000 kN/m/m^2
130%	83200 kN/m/m^2

Presenting the results in the figure 10.

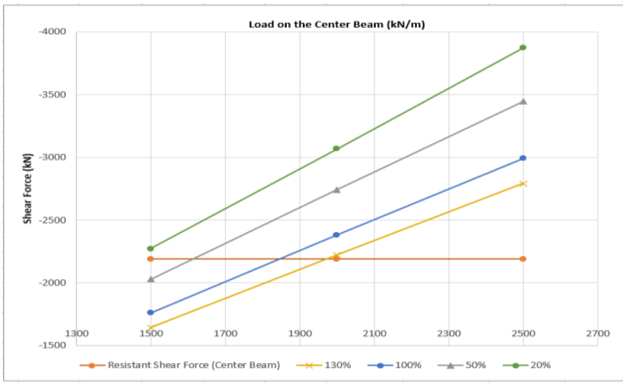


Figure 11. Results of the sensitivity analysis to the parameters of the soil in the section of Analysis A1.

Looking at the figure 11, a decrease in the maximum load resisted by section A1 is observed as the foundation's resistance decreases, fluctuating between the values 1400 and 2000 kN/m. Reflecting about the values analysed, it is considered as the maximum load to which the centre beam may be subject will be an average value between the two: 1700 kN/m, or 240 Ton/m,. The values presented as extremes are excluded, because they are unrealistic or against safety. It is then defined as reference a reduction of the rigidity of 50%.

With the rigidity of the direct foundation defined, the next step is to measure the influence of the slab´s rigidity. Considering in this analysis the following cases: consideration of concrete in excellent condition and the consideration of concrete in a poor state of preservation. The slab´s rigidity is represented in the table below.

Table 9. Arbitrated values used on the sensitivity analysis of the slab´s rigidity.

Percentage compared to total value
30 %
50 %
75 %
100 %

Presenting the results in the following figure:

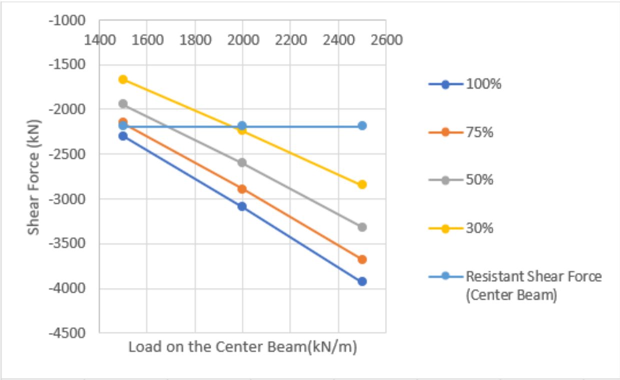


Figure 12. Results of the sensitivity analysis of the slab´s rigidity in the analysis section A1.

Looking at the figure 12, there is an increase in the maximum load resisted by the bottom slab, as the slab´s rigidity decreases, setting the maximum load between the values 1400 and 1900 kN/m.

Similarly, the approach used to determine the direct foundation rigidity, a 50 % reduction is considered in the slab´s rigidity. Defining as reference parameters and representative of the cracking state of the structure a soil´s rigidity of 32000 kN/m/m^2 and a 50 % reduction on the slab´s rigidity, defining the maximum load in the centre beam section of about 1700 kN/m, or 240 Ton/m.

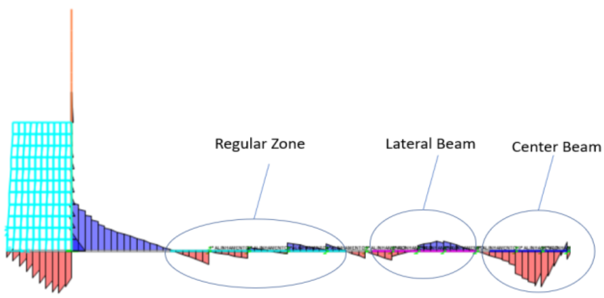


Figure 13. Shear force diagram, for the bottom slab with the ultimate load.

4.3. Case 2 – small/medium sized ship

The second case is divided into two distinct analyses: assessment of the capacity of the regular zone and the lateral beam, considering the ship displacements shown in the figure 9 and changing the ship load on the slab between 300 and 900 kN/m.

To do this, the rigidity definitions determined in case 1 are considered. A study like Case 1 will be carried out, where the load is gradually increased until the limit resistance is reached. Firstly, we proceed on the evaluation of the maximum load resisted by the lateral beam by evaluating the resistant capacity for section B1. The case of the slab and direct foundation rigidity reduction of 50%, and by comparison between the measured shear force in the finite element model and the resistant shear in section B1, it is determined that the maximum load resisted by the lateral beam is about 550 kN/m, or 50 Ton/m. It is then evaluated the resistant capacity of section B2 concerning the maximum load that can be applied in the regular zone. It is determined that the maximum load to be applied in the regular zone, to ensure safety and avoid the rupture of the structure, is about 400 kN/m or 35 Ton/m.

4.4. Final load

Completed the analyses and determined the maximum loads in each section, the different results are summarized, indicating a possible load distribution to be applied to Panel 11 that complies with the safety and avoid its rupture by lack of resistance to the shear force. It is shown in the figure 14 a possible distribution of the loads.

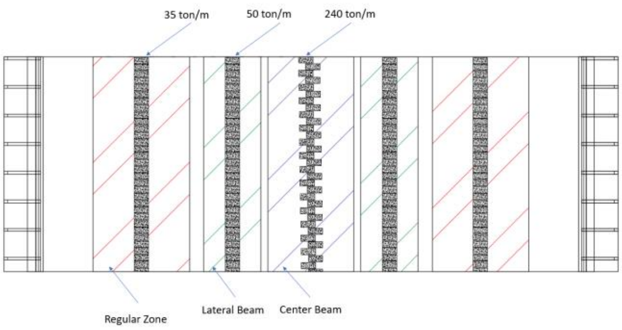


Figure 14. Load distribution on panel 11.

5. Structural capacity related to the bending moment

This section focuses the ideas on 2 types of analysis, linear and nonlinear analysis exemplifying the main differences and the approaches adopted in each case, to determine the maximum resistance capacity of the structure to bending moment in the central section of the slab. The determination of bending resistance is carried out only in the case of the large ship, determining only the maximum load on the centre beam, since this analysis is not a priority like the assessment of the resistant capacity through shear force.

5.1. Linear analysis

Defined the analysis characteristics, the maximum load resisted by the section is evaluated, considering the reference case of rigidity: reducing the slab and soil´s rigidity by 50%, due to cracking phenomena and due to lack of information relating to the soil. The analysis is carried out in two sections considered critical, due to the substantial increase of the bending moment, represented in the following figure:

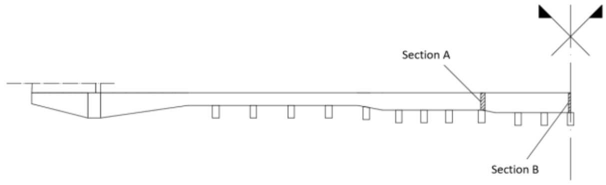


Figure 15. Definition of the location of the analysis sections.

After analysing the various types of load discussed previously, it is concluded that the relevant load types are 5, 6 and 7 (The different types of loads are illustrated in table 7) . Cases 1 to 4 are safe to the structure since they do not violate the safety condition and type 8 is an extreme case where the structure would be in rupture. Determining that the critical section is A, the maximum load that could handle is around 3000 kN/m, much lower than in section B, right about 4000 kN/m, concluding that the maximum load resisted by panel 11 on the centre beam, performing a linear analysis is 420 Ton/m. Figure 15.

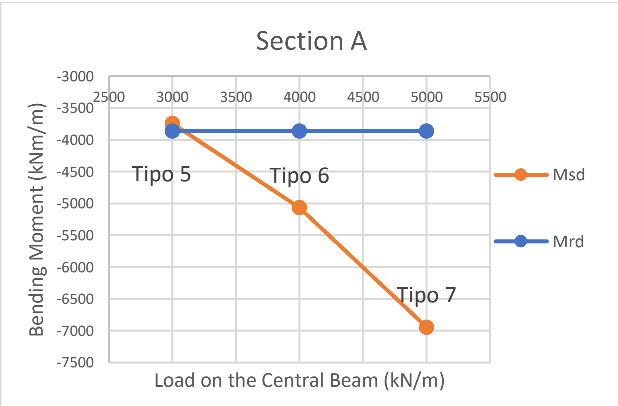


Figure 16. Analysis of load types 5, 6 and 7 in test sections A.

5.2. Non-linear analysis

The non-linear analysis considers the amount of reinforcement in determining the slab´s capacity, as the rupture is reached when the steel is in the plastic branch and not when it reaches his yielding point. Another characteristic for non-linear design to be possible is that the structure should present ductility, that is, it must be able to withstand large deformations without loss of resistant capacity.

This type of analysis presents enormous advantages, allowing to optimize the structures by taking the maximum advantage of the materials, as well as a better understanding of the behaviour of the structure in limits or close to collapse situations. (M. L. Gambhir, 2013) This method represents an added value in relation to the previous analysis, in the sense that it uses the reserve of resistance not considered in the linear analysis, exploring the material´s maximum resistant capacities.

5.2.1. Modelling

The modelling of the non-linear analysis in this case was carried out using the plastic hinge concept. The definition of plastic hinge begins with the moment-curvature diagram obtained in each section, noting that the plastic bearings placement excludes the peripheral walls area because it's not very relevant, given the enormous amount of reinforcement. Defined the hinge formulations in the finite element model, the following question comes: What is the maximum load that the slab supports? What is the collapse mechanism that should be observed? In the non-linear analysis occurs the formation of plastic hinges until the structural redundancy is exhausted, culminating in the overall collapse of the structure. Given the numerous structural redundancy points that the slab presents the occurrence of general collapse is extremely complicated, concluding that the solution falls into the consideration of partial collapse, that is, the occurrence of collapse from a certain area of the structure not affecting the structural integrity of the other zones, evaluating the various effects of the load types.

Watching the results, it turns out that section A will be the first to form a plastic hinge and consequent rupture, presenting in type 5 a plastic hinge formation, and in type 6 resistance loss, symbolizing the rupture of the section. In section B, yielding point is obtain in type 6 but in the following phases it remains at a constant level, never occurring section collapse, illustrative in figure 18. The exact loading point at which the rupture in section A is between the load types 5 and 6, observing a jump from the elastic branch in Type 5 and section rupture in type 6, proved impossible to identify the maximum load, therefore, two intermediate load types of those presented in type 5 and 6 were considered. The intermediate cases covered were those presented in table 10.

Table 10. Intermediate load types analysed.

Intermediate case			
Type	A (kN/m)	B(kN/m)	C(kN/m)
5 A	3250	815	270
5 B	3500	875	290

Concluding, from the results, the existence of a level of baseline in type(5 A and consequent rupture in type 5 B. Determining that the maximum load to which the structure resists, in the case of the large ship and considering a non-linear modelling, is about 3500 kN/m, based directly on the centre beam, or about 500 Ton/m. Considering that any load above the value presented would lead to partial collapse of the structure through the formation of plastic hinges in section A and consequent collapse of the central zone of the slab as follows (figure 17):

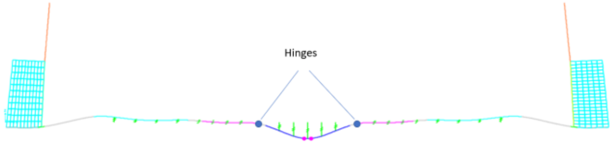


Figure 17. Illustration of partial rupture of the slab after reaching the maximum load.

5.3. Flexural-Resistant capacity

In the case of non-linear modelling it's considered the plastic resistance of the section, creating a level of baseline and keeping constant at the resisting bending moment until the ductility of the section is exhausted. Figure 18 compares the results obtained between linear and non-linear modelling as the incident load increases on the bottom slab

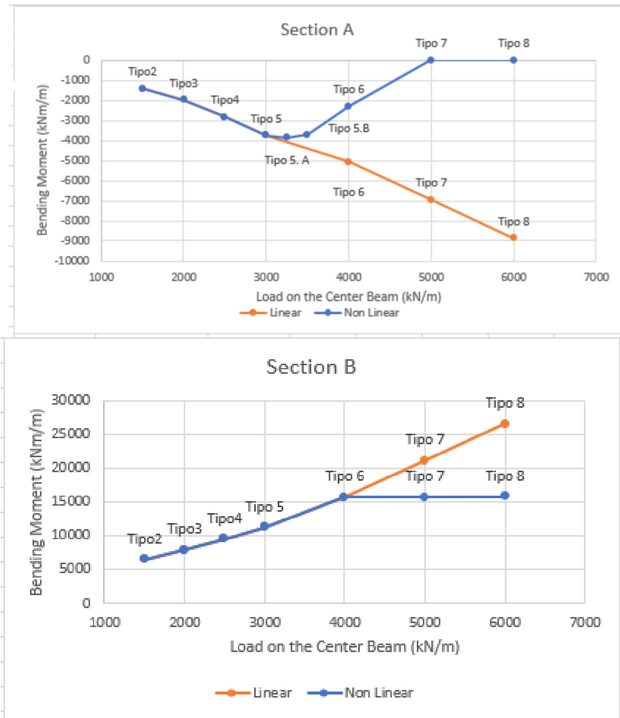


Figure 18. Comparison of the bending moment in sections A and B for linear and non-linear modelling

The consideration of non-linear analysis benefits the assessment of the resistant capacity in the case of existing structures, exploring the ductility inherent to the structure and defining the point at which the structure will suffer irreversible damage, determining a maximum load, exerted by the concrete blocks in the centre beam zone, of 3500 kN/m corresponding to a ship of about 500 ton/m

6. Conclusion

In this dissertation, the study aimed at the determining the maximum load capacity that a dock can withstand, focusing specifically on the analyses on the dock's panel 11. Firstly, the soil and piles stiffness were calibrated in the finite element model, using as a starting point the classical theories and evolving iteratively in order to approximate the deformed measurements in the model with those obtained through experimental tests. It was verified that the pile rigidity reduction was due to the interaction of the pressure bulb phenomena, checking reductions up to 80% of the initial stiffness in the central zone, coincident with the zone where the spacing between piles is smaller. This reduction in rigidity is being attenuated as it approaches the peripheral walls, being noted that the spacing between piles increases from the centre of the slab to the foundation zone of the peripheral walls, concluding that the reduction of stiffness through the interaction of the pressure bulb phenomena is mostly controlled by spacing between piles. The previous analysis was based on the geotechnical prospecting campaign carried out in the 1970s, performing a later sensitivity analysis for the soil and slab´s rigidity parameters. Subsequently, the influence of the soil and slab´s rigidity variation was evaluated, defining as a reference case a 50% reduction in slab and soil rigidity. This was the reference case considered out on the analyses in search of the maximum load to which the slab could be subjected.

Once the soil´s calibration phase has been completed, the maximum load supported by the bottom slab is considered for the work, considering two forces: shear force and bending moment. The determination of the ultimate load for bending moment was divided into two analyses: linear and non-linear. Concluding that in order to perform a non-linear analysis, a priori knowledge of the amount of reinforcement in the structure is necessary and, therefore, the ideal solution for analysis of existing structures.

Two cases were defined regarding the type of ships to which the dock could be subjected by, dividing in to large ship in case 1, where the analysis is made for shear and bending forces and small and medium-sized ship in case 2, where analysis is performed for shear force.

Comparing the analyses for case 1, the linear and non-linear situation, it was verified that the maximum weight of ship that the centre beam supports in the linear situation is about 420 tons / m, a value well below that obtained by the nonlinear analysis of 500 ton / m, revealing a great discrepancy between the results. In the evaluation of the behaviour of the slab against the shear force it was concluded that the maximum weight is 240 ton / m. In this way, the maximum load supported by the dock is conditioned by the resistant shear force of the slab.

7. **References**

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