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Seismic Assessment of a 'Placa' Building

M. Monteiro; R. Bento

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1. CASE STUDY – DESCRIPTION OF A ‘PLACA’ BUILDING

1.1 Introduction

The existing building stock in Portugal results from more than eight centuries of history expansion. Excluding monumental buildings, such as churches, palaces and convents, the building area is primarily composed by residential buildings, built making use of the available materials and traditional techniques of construction that changed very little through time. The assessment of the constructive materials, techniques and structural behaviour of the main typologies of buildings in Lisbon, gives a general panorama on the evolution of constructive systems in Portugal mainland.

It is estimated that half of the existing building stock in Lisbon County is composed by old masonry buildings (Ravarra *et al*., 2001). The survey ‘Censos 2001’ (INE, 2002) confirmed that approximately 67% of these buildings are in need of structural intervention works and that 10% are in an advanced state of degradation. In fact, the functions that old buildings still maintain nowadays, justify the concern about their structural safety and seismic vulnerability.

In Lisbon is possible to identify specific periods of the old masonry buildings, built long earlier than the massive reinforced concrete framing construction. In this context, four typologies of masonry buildings are typically recognized in the Lisbon Metropolitan Area: buildings built before 1755, ‘Pombalino’ buildings built after the 1755 Earthquake, ‘Gaioleiro’ buildings built between 1870 and 1930 and ‘Placa’ buildings between 1940 and 1960, a short-term structural solution which precede the reinforced concrete buildings.

The increasing concern about the built heritage led to the development of several studies regarding the assessment of structural performance and seismic vulnerability of old masonry buildings in Lisbon, especially the ‘Pombalino’ Buildings and ‘Gaioleiro’ Buildings. However, there are still very few studies regarding ‘Placa’ buildings, because, although there are many examples of these buildings in Lisbon, this transition period was very short, it lasts less than 20 years.

So, the main purpose of this work is to study the seismic vulnerability of ‘Placa’ buildings in Lisbon, which corresponds to the last type of buildings made by masonry.

For the seismic assessment of this type of building is need to choose a landmark building. For this work the ‘Rabo de Bacalhau’ buildings type was chosen, which is the most representative in this period (1940-1960). These buildings have a particular plant design, which is easy to identify because the buildings have the salient body in the back.

Figure 1 shows the location of such buildings in Lisbon.

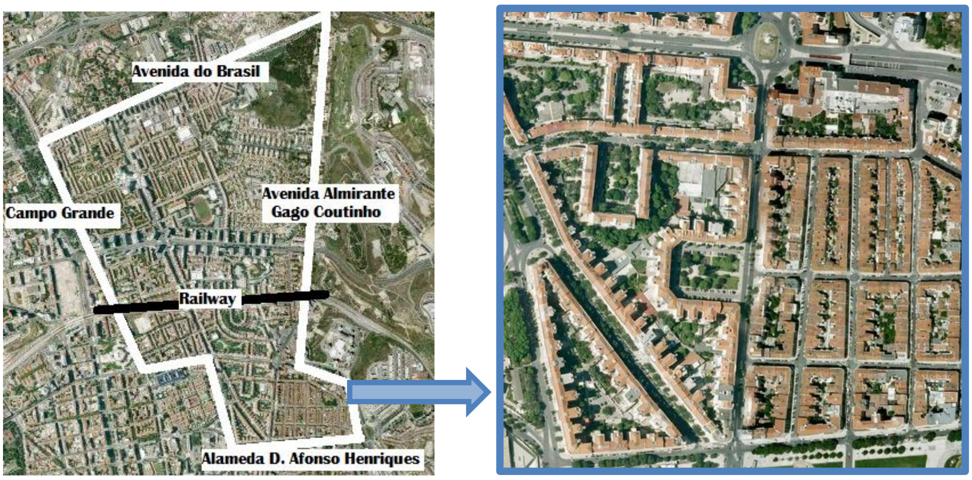


Figure 1 – Location of 'Placa buildings' and the 'Rabo de Bacalhau' buildings (Google Maps)

This type of construction has the mixed structural elements of masonry and concrete, but the main characteristic corresponds to the slab in concrete instead of the wooden floor on the previous masonry buildings.

The building studied was chosen to fulfil the following requirements:

- To belong to the beginning of the period of this type of construction, because these buildings have less structural elements in reinforced concrete;
- To have access to drawings and documents supporting the initial project;
- To be possible to visit the building, aiming to identify more accurately the materials and the dimensions of the structural elements, to verify if some structural modifications had been introduced over the years and to perform the in-situ ambient vibration tests.

1.2 General Characterization of the Building

The 'Placa' building used for this study was completed in 1940 and is located at Rua Actor Isidoro N. 13, belonging to the Bairro dos Actores, near the Alameda D. Afonso Henriques in Lisbon (Figure 2). This building is inserted into a block, where all buildings have the same characteristics and almost the same date of construction.

Figure 2 – Location of the 'Placa' Building on study Building located in the Bairro dos Actores (Google Maps)

According to the "Descriptive Memory ", the ground was considered in "favourable conditions" (Descriptive Memory, 1939). However, as shown in Figure 3, the building under consideration is located in a ground type C, according to the classification criterion of Eurocode 8 (EC8.1, 2009).

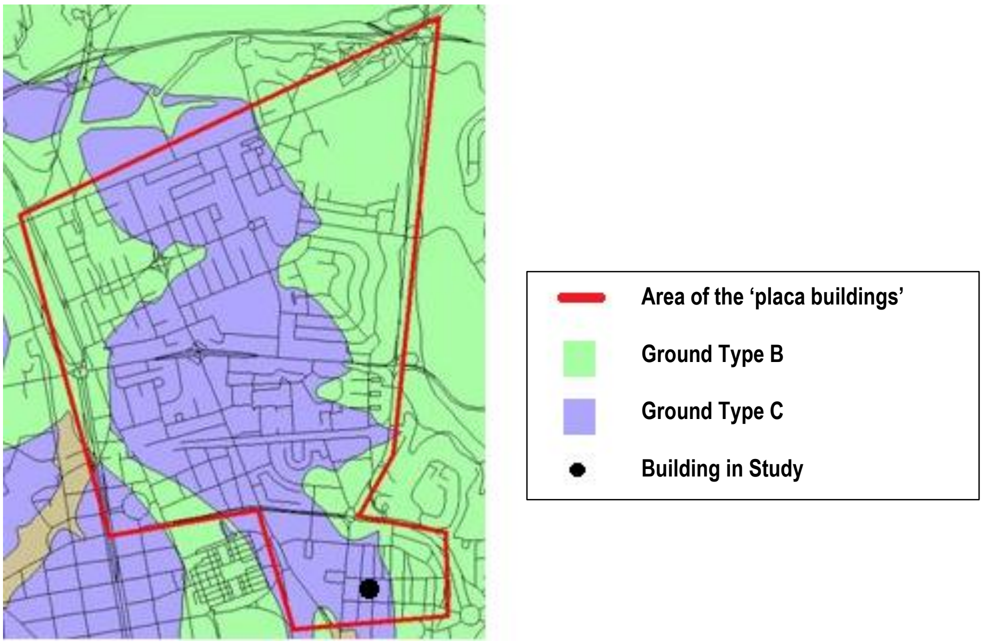


Figure 3 - Location of the building under study according the Geotechnical Plant of Lisbon. [adapted from (Almeida, 1997), (Almeida et al., 1986) and (CML, 2008)]

The building chosen has four floors (ground floor and three floors high), as shown in Figure 4. The story height is constant with 3.0 m, while the ground floor has a story height of 3.25m.

The total height of the building is around 17m, from the street level until the top of the roof. The façade of the building has a length of 14.5m and has 20.5m in depth.

Figure 4 –Building façade.

Figure 5 - Back of the building.

For the study of this building, it was possible to access all the information available from the original design, which include plants, façade designs, some calculations for the design of reinforced concrete structure and other elements (Figure 6 to Figure 8). All of these elements are available in Annex 1.

Moreover, to complement this information, some visits to the building were made, where some photos were taken as well as the dimensions of several structural elements were measured. These values were then compared with the original ones depicted on the drawings.

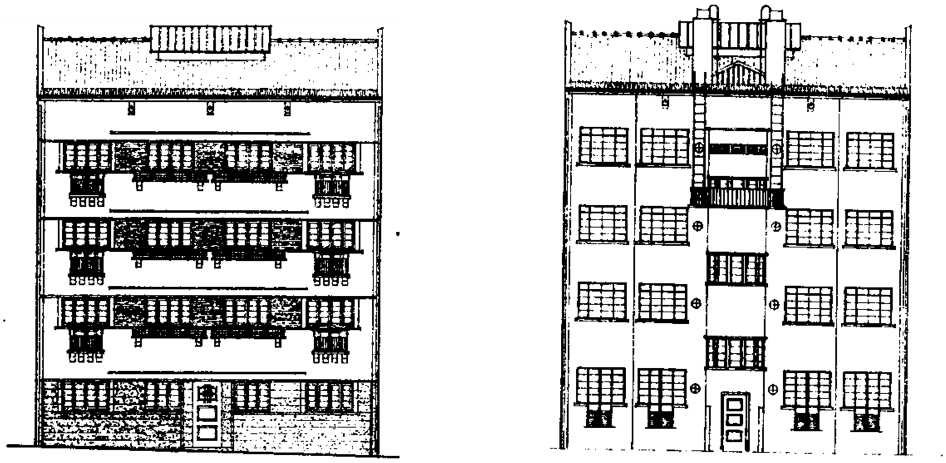


Figure 6 – Front Façade Design and Back Façade Design (Descriptive Memory, 1939)

Figure 7 – Section A-B (Descriptive Memory, 1939)

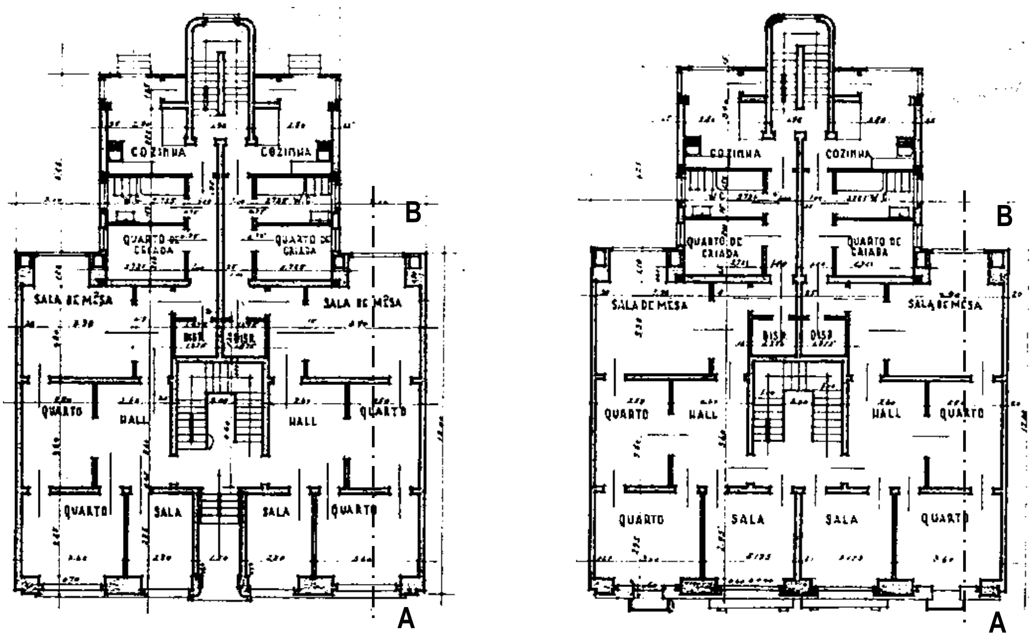


Figure 8 – Plant of the Ground Floor and of the High Floors (Descriptive Memory, 1939)

According to the information available in the Municipal Archives of Lisbon (1939) this building was targeted of rehabilitation works in 1988. However these works were not structural works aiming the seismic strength of the building; they were only to repair the exterior of the façades, roofs and staircase (Figure 9).

Currently this building is habitable and with a good appearance, like almost all the buildings of this period.

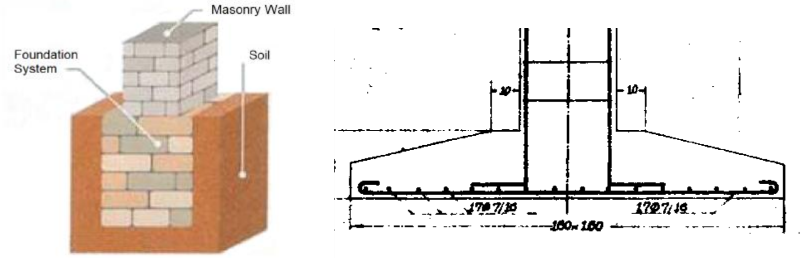
Figure 9 – Photograph taken during the repair works of the façade on the building in front.

1.3 Structural Elements

Although it was not done any intrusive testing, the original project of the building (Annex 1) as well as in-situ analyse of the building were enough to define adequately of the structural elements.

1.3.1 Foundations

In this building it was used two different types for the execution of foundations (Figure 10). In general, the foundations were executed continuously over all masonry walls, with about twice the thickness of the walls so as the maximum stresses do not to exceed 2 MPa according to the original design (Descriptive Memory, 1939). As this building has a reinforced concrete frame structure on the back of the building, the column foundations were also made of reinforced concrete (Figure 10 b). The foundation layout is defined in Figure 11.



a) b)

Figure 10 – Foundation system: a) Continuous walls in limestone masonry (Appleton, 2003) b) Reinforced concrete foundations (Descriptive Memory, 1939).

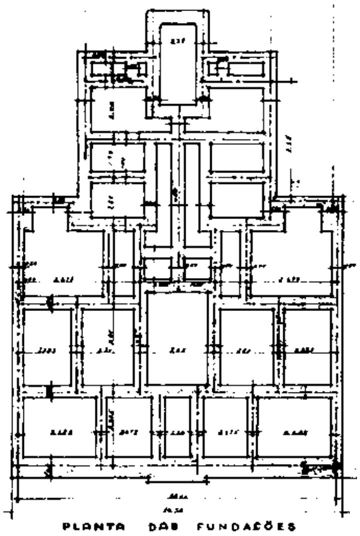
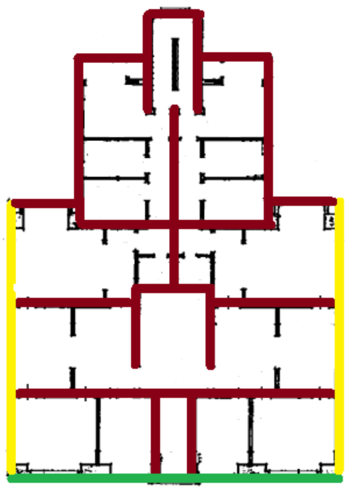


Figure 11 - Foundation plant (Descriptive Memory, 1939).

1.3.2 Walls

This building has different types of walls as shown in Figure 12. The walls on the front façade are made by rubble stone with a 0.7m of thickness and do not have a reduction in height (Figure 13). However, there is a zone between the windows in each floor where the thickness of the front wall is only about 0.35cm, as can be seen in Figure 7.

The exterior walls on the back of the building are made by solid brickwork with 0.2m of thickness, which are supported on a reinforced concrete structure.



Colour	Designation	Material	Thickness
	Partition Walls	Hollow brickwork	0.1 m
	Interior and Exterior Walls	Solid brickwork	0.2 m
	Gable Walls	Concrete blocks	0.2 m
	Front façade wall	Rubble masonry	0.7 m

Figure 12 – Location of the different types of wall.

In this period, there are references to the first use of the concrete blocks in the walls of buildings (Figure 14). These blocks were used in the gable walls. It should be refer that the gable walls on each building of the quarter are not shared.

The interior walls are made by brick masonry (solid and hollow) and the wall thickness varies from 0.10m to 0.20m. All the walls that supporting loads (including the stairs walls) are made by solid brick with 0.2m (Figure 14). The other partitions walls are made by hollow brickwork with 0.1m of thick, thus they do not really contribute to the seismic resistance of the building.

Figure 13 – Rubble masonry walls - Photograph taken during the repair works of the façade of the building in front.

Figure 14 - Gable wall (on the left) and the stair wall (on the right) in another building at the same period.

1.3.3 Reinforced Concrete Elements

In this type of buildings the reinforced concrete frame structure started to appear at the back of the buildings that corresponds to the humid zones (kitchens and bathrooms). However these elements were designed with minimum dimensions and only for the vertical loads.

Through the Figure 15 one can identify the reinforced concrete frame at the back of the building, constituted by beams and columns. Furthermore, it is also possible to observe the existence of reinforced concrete lintel in the façade wall at each floor at level of the superior length of the window. The dimensions of each reinforced concrete elements are shown in Table 1 and identified in Figure 15.

To confirm the existence of reinforced concrete elements in the structure as well as its position according with the Descriptive Memory (1939), some experimental tests were conducted using the cover meter. This equipment is a sophisticated device for the non-destructive location of the steel reinforcement and for the measurement of concrete cover and bar diameters, using the “Eddy current Theory” with pulse induction as the measuring method. In the Annex 5 the results of this test performed are shown.

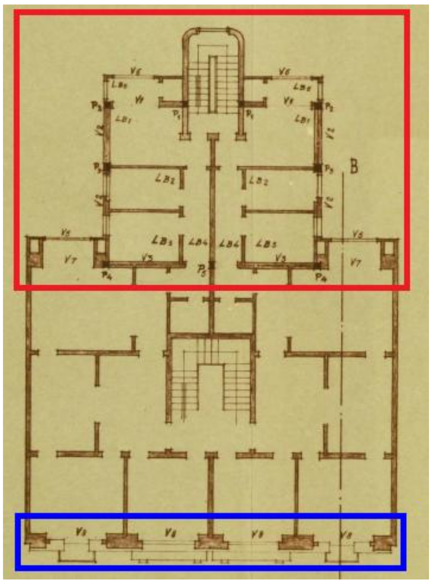


Figure 15 - Location of the reinforced concrete elements in the structure (Descriptive Memory, 1939)

Table 1 – Dimensions of the reinforced concrete elements (Descriptive Memory, 1939)

Columns	Dimensions (m)
P1	0.2x0.3
P2	0.3x0.3
P3	0.3x0.35
P4	0.3x0.3
P5	0.35x0.3

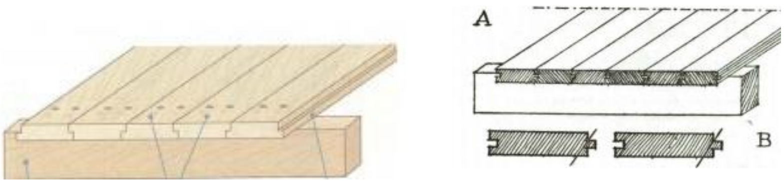
Beams	Dimensions (m)
V1	0.3x0.23
V2	0.3x0.23
V3	0.35x0.23
V5	0.3x0.13
V6	0.3x0.13
V7	0.3x0.23
V8	0.3x0.66

1.3.4 Floor

The building floor consists by different materials. At the front, the building has a wooden floor, so it has a flexible behaviour. On the back of the building, supported by the reinforced concrete frame, there is a concrete slab, thus providing rigid plan behaviour, although the thickness of the slab is reduced. According with the Descriptive Memory (1939), the concrete slabs have 0.1m thickness and has reinforcement in both directions.

The wooden floor is made up of wooden beams (0.08m x 0.18m) spaced from 0.4m to 0.4m and they are tight by billets. The wooden floors are supported directly on the exterior and interior walls and the overall thickness is about 0.30m.

According to Costa (1955) the Portuguese floor (“soalho à portuguesa”) was the most common used in economic houses construction. However, in 'houses of limited income’ the predominant type flooring was the English (“soalho à inglesa”) (Descriptive Memory, 1939). The great advantage of this floor is that the nails are not visible on the flooring, as they are at the Portuguese floor (Figure 16).



a) b)

Figure 16 – Two types of flooring: a) Portuguese flooring (Appleton, 2003) and b) English flooring (Costa, 1971)

1.3.5 Stairs and Roof

The roof of the building consists of ceramic tiles, which are supported on a wooden structure that is composed by a set of parallel trusses connected by purlins (main beams) and common rafters and slats (Figure 17).

The main staircase is built by a wooden structure to be compatible with the wooden floor (Figure 18), while the secondary stair was built by iron structure (Figure 19).

Figure 17 – Wooden structure on the coverage in another building at the same period.

Figure 18 – The main stairs of the building in study

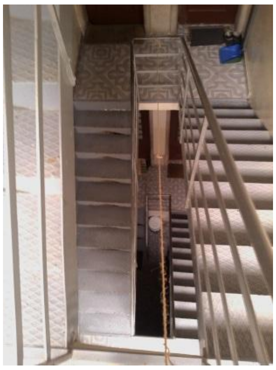


Figure 19 - The service stairs of the building in study

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2. NUMERICAL MODEL

2.1 Introduction

The suitability of the building analysis model is essential for the adequate seismic assessment of the building. Thus, despite the local visits and different experimental tests performed it is necessary to consider several hypotheses that should reflect, as much as possible, the current behaviour of the structure.

Therefore, in this section was defined the mechanical characteristics of materials used, the values admitted for the mass in the building elements, and the elements used in the modelling of the building. The final model was calibrated based on dynamic test characterization, as previously referred.

The model was performed using SAP 2000 computer program (SAP2000, 1995), which is based on the finite element method.

2.2 Mechanical Properties of Materials

The modelling of the building studied began with the definition for the mechanical properties of materials. Preferably, these values should be based on tests performed in the elements of own building. However the building is still habitable, so intrusive tests were not possible to do, which is why the values entered in the model were determined based on data available on some literature, related to studies developed in buildings with similar properties (Annex 3 and Annex 4) (Cardoso, 2002; Costa and Oliveira, 1989).

Table 2 shows some of the properties of materials used in the building: weight density, modulus of elasticity, Poisson's ratio and also the strength values of materials for compression σ_c, tension σ_t and shear τ.

Table 2 – The properties of the materials used in the model.

Materials	Local	Weight density γ (kN/m^3)	Modulus of Elasticity E (GPa)	Poisson's ratio ν	Strength of Materials		
					σ_c [MPa]	σ_t [MPa]	τ [MPa]
Rubble Masonry	Front façade	22	3	0.2	4	0.4	0.2
Concrete Block	Gable Wall	14	2	0.2	4	0.5	0.3
Brick Masonry	Interior walls and exterior walls on the back	14.6	1	0.2	5	0.1	0.2
Wood	Floor and Coverage	6	6	0.2	50	0 (F_{max} = 10kN)[1]	5
Reinforced Concrete	Frame and Floor	24	29	0.2	Concrete f_{cd} = 10.7	Steel f_{yd} = 204	

[1] Assumed value of the pull-out strength for the connections between the timber elements and the façade - timber beams attached to masonry walls (Lopes et. al, 2004)

2.2.1 Masonry

The value of the modulus of elasticity E assumed in this work is an average value of the overall wall behaviour instead than a single masonry element. This assumption is mainly due to lack of homogeneity in the masonry behaviour. This parameter is model adequately the behaviour of the structure. So, it was used as initial estimate 3.0 GPa for rubble masonry, 2.0 GPa for the concrete blocks and 1.0 GPa for the brick masonry. Afterwards, some adjustments were made based on the experimental values obtained for the dynamic characteristics of structure, by means of the in-situ ambient vibration tests.

In this study the damping coefficient considered was 5%, but some other studies suggested higher values. The adoption of this value was considered more conservative because increases the forces due to seismic actions (Costa and Arede, 2004).

The masonry comprises two distinct elements: stone and mortar. The joint strength is higher than the strength of mortar, which is the weakest material. The values of the maximum tensile stress are very low, including only a residual strength due to the cohesion of the mortar-block connection. As far as the shear strength concerned, it is worth to be mentioned that the disruption is fragile and occurs through displacement of the blocks in the matrix of the mortar (Branco, 2005; Cardoso, 2002).

According to the Descriptive Memory (1939), the masonry used on the façade is the *lioz* (a kind of limestone), typical of the Lisbon region and it is extensively used in the façade walls at this period. Besides that, the mortar in the exterior walls is made of cement and sand, while in the interior walls the mortar is made by lime and sand (Descriptive Memory, 1939).

Figure 20 shows that there is little mortar in the connection of the masonry blocks. So it is possible that the strength values can be even smaller.

Figure 20 – Photograph taken during the repair works of the façade, on the building in front, where it can observe little mortar on the middle of the rubble masonry.

2.2.2 Wood

The national pine is the type of timber most often used in these buildings, and according with the description of the building design was the wood used, for which reason their characteristics were adopted in this model.

The damping coefficient considered was 5%, although some authors consider the damping of 10% for wood. As, in these type of buildings the masonry elements are the ones that significantly influence the seismic response, it was assumed a damping equal for all elements of the structure (Branco, 2005), (Cardoso, 2002) and (Costa and Arede, 2004).

2.2.3 Concrete

According to the description of the building design (Descriptive Memory, 1939), the reinforced concrete was designated by "reinforced cement" (in Portuguese "cimento armado") and it should follow the concepts mentioned in Reinforced Concrete Regulation of the season, 1935 (in Portuguese 'Regulamento do Betão Armado' which is in annex in the "Regulamento Geral da Construção Urbana", RGCU).

The cement used was the national Normal Portland manufacturing and its dosage was composed by 300 kg of cement, 400l of sand and 800l of gravel (Descriptive Memory, 1939).

The description of the building design only shows the density of concrete used, so for the other mechanical proprieties the values which are in the Reinforced Concrete Regulations of the time were considered.

The resistance was defined by the average of test results for the compression simple, to 28 days, on cubes with 20cm of edge, and should be more than 120 Kg/cm^2 (Beton Armado, 1918) and 180 Kg/cm^2 (RBA, 1935). According with these codes the design value of concrete compressive strength is around 50 Kg/cm^2. So in this study it was considered that the concrete has the properties of the previous concrete of class B180 (REBA, 1967) and now C15/20 (EC2.1, 2010).

2.2.4 Steel

The code of 1935 refers that the steel available has with an ultimate stress of rupture superior than 3700 kg/cm^2. So, for the steel used in this building it was the previous class A24 (REBA, 1967) corresponding to the current A235 (EC2.1, 2010). At that time, the steel bars were smooth and were designated by inches.

2.3 Definition of Mass

The total mass of the building is crucial to define the dynamic response of the building, and thus the frequencies and the modes of vibration. When higher is the mass, lower is the fundamental frequency. Therefore, the masses should be carefully identified, the values as well as their distribution (Azevedo and Proença, 1991).

The building in study is still habitable, so not only was considered the mass of the building elements but also the possible life loads. The values of the masses were taken from the Descriptive Memory (1939), Techniques Tables (Ferreira, 1974) and the remainder bibliography.

All components of the building structure consider the mass in definition of the own material. The exception is in the definition of the concrete slab at the back of the building where it was decided to model a rigid diaphragm behaviour and apply the equivalent mass at its center of mass.

The masses of the elements mentioned above are summarized in the following Table 3. These values are used to define the mass density and weight of the elements corresponding to model the floors, as described later on.

Table 3 – Mass and weight distribution.

Zone	**Dead loads** [kN/m^2]	**Remaining Permanent Load** [kN/m^2]	**Overload** [kN/m^2]
Wood floor	0.7 a)	0.6 a)	2.0 a)
Concrete slab	2.4 a)	0.6 a)	2.0 a)
Roof wood	1.3 b)	0.6 a)	0.4 a)

a) (Descriptive Memory, 1939)

b) (Branco, 2007) (Farinha and Reis, 1988)

2.4 Structural Elements for the Building Model

The building model was defining by means of different finite elements available in SAP2000. Each structural component of the building was modelled with the element, which is capable to produce the most appropriate behaviour. So, it was used elements type shell and frame (SAP 2000).

2.4.1 Masonry Walls

For modelling of the masonry walls were adopted two-dimensional finite elements (shell) of four nodes. The main purpose of using these elements, instead of three-dimensional, was based on the interest of simplifying the model. The two-dimensional elements are sufficient to measure the maximum response values, identifying its location in the structure, which is required in this work.

The size of the shell elements was defined by adaptation of the mesh nodes according with the different alignments that there are in the model.

To model the building it was considered all masonry walls, with the exception of interior partition walls, and were represented all the openings (doors and windows) of the façades and of the interior walls with the real dimensions and location (Figure 21).

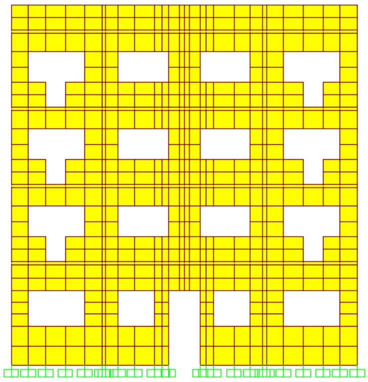


Figure 21 – Façade wall with the openings.

The walls on the back of the building, although are included on the concrete frame structure, were taken into account in the model because these walls have an important role in the resistance of the structure.

There are two kinds of interior walls, as previously referred, with solid bricks and bricks with holes. For the model only it was considered the walls with solid bricks because they are the ones that significantly affect the stiffness and strength of the structure.

So, as shown in Figure 12, all the walls were represented in the model (Figure 22) for the dynamic analysis, with the exception of the partition walls (identified by the black colour in Figure 12), since they have a reduced thickness and thus a small influence on the seismic response of the building.

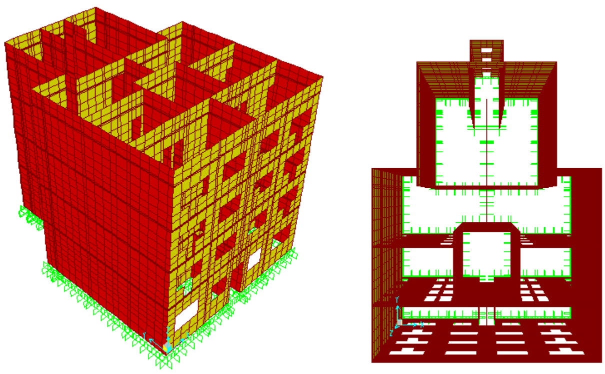
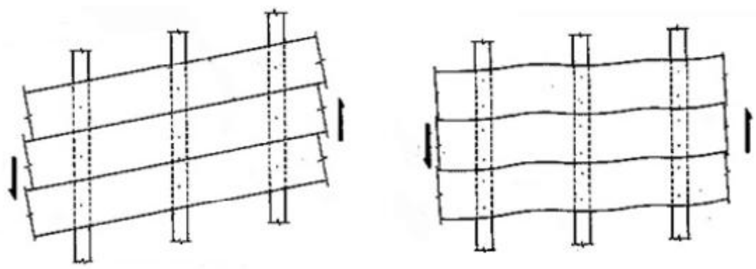


Figure 22 – Masonry walls in the building model

2.4.2 Floors

Wooden Floor

For the wooden floors, the hypothesis of rigid floor is not acceptable because the planks of wood, which link the support beams of the floor, do not ensure sufficient rigidity. As an example, it is assumed that the deformation shown in Figure 23 can occur.



a) b)

Figure 23 – Possible deformation in wooden floor: a) distortion: one nail per plank of wood; b) flexion: two nails per plank of wood (Carvalho and Oliveira, 1997)

To model the wooden floor a mesh of bar elements was used, where were introduced bars to simulate the wooden floor beams, assuming that these bars are arranged according shorter span, which in this case corresponds to the direction perpendicular to the façade wall. This orientation is also mentioned in the literature (Segurado, nd), since the timber beams were placed over the masonry walls resistant, that are parallels to the façade.

In order to adjust the mesh modelling of the timber beams (Figure 24), according the spacing between elements in real distribution, it was created four classes of spaces, avoiding defining different properties for each element which had a different influence area.

Each class of elements considers that the beams have a section of 0.18×0.08 m^2 and a spacing of 0.40m to the axis, corresponding to 2.5 beams per meter. These dimensions are common in this type of building and the section of the wooden floor is shown in Figure 25. For each class of spacing it was calculated the area and the moments of inertia per meter.

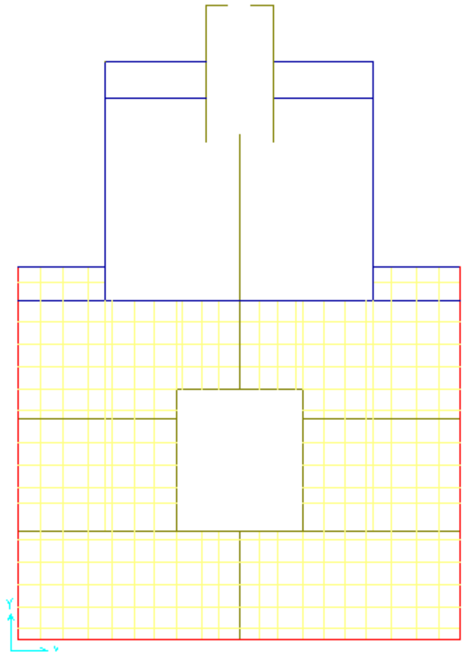


Figure 24 – Mesh of wooden beams in plant.

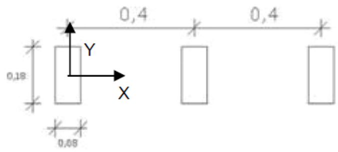


Figure 25 – Section of wooden floor per meter (Branco, 2005)

To determine the correct moments of inertia for each class of spacing was enough multiply the values calculated per meter (I_x) by their spacing and divide them by the moment of inertia of one beam (I'_x), according to the following equation (Eq. 1).

$$Correction\ of\ Inertia = \frac{I_x \times Spacing}{I'_x} \quad \text{(Eq. 1)}$$

The definition of the masses and weights of each element took into account the mass defined in the previous section and it was considered an equivalent density per element. The elements are designed with a section of 0.18 × 0.08 m^2, with densities equivalent according to the class spacing and were calculated according to equation 2.

$$Density\ Equivalent = \frac{Spacing \times Mass}{Area} \quad \textbf{(Eq. 2)}$$

In the direction perpendicular to the main bar elements were used bars with the dimensions 0.08 x 0.08 m^2, in order to represent the billets sections usual in this type of flooring. The weight of these elements is distributed by the main elements (Branco, 2005). These bars have a small area only, because the objective is to prevent relative displacement between the alignments that unite them.

Since it is unknown the conditions of connections between the wooden beams to the masonry wall, another study was performed, as these connections to the masonry could be weak points of the structure.

So, was chosen to label the ends of the bars that connect to the masonry (Figure 26). However, this modification did not cause significant changes to the response of the structure.

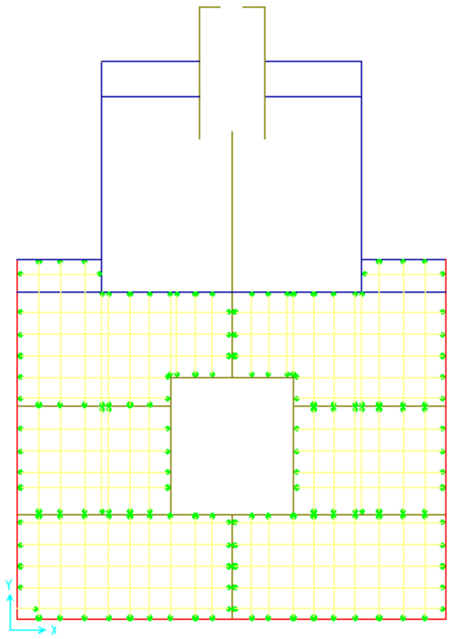


Figure 26 - Placement of the hinge bars at the ends on the links between the wooden beams to masonry walls.

<u>Reinforced Concrete Floor</u>

The slab on the back of the building has a reduced thickness (only 10 cm). Nevertheless, it was chosen a rigid diaphragm to model the slab, although some authors supporter that, in these cases, the behaviour of the floor in its plan cannot be considered as rigid.

In this way, it was considered that the floors don not have axial deformations, but may have displacements perpendicular to the plane (it can occur deformations due to bending). After modelling the rigid diaphragm, it was applied at the center of mass to each floor all of the mass representing the weight of the concrete slab, as well as the load it supports.

2.4.3 Reinforced Concrete Frames

Frame elements were also used to model the concrete frame, i.e. the beams and columns, as shown in Figure 27 and Figure 28.

Some concrete beams, according to the description memory of the building, are simply supported on masonry walls. Thus, in the model, these elements were labelled at their ends.

A good example is on the façade wall where there are some reinforced concrete lintels over the openings (Figure 28).

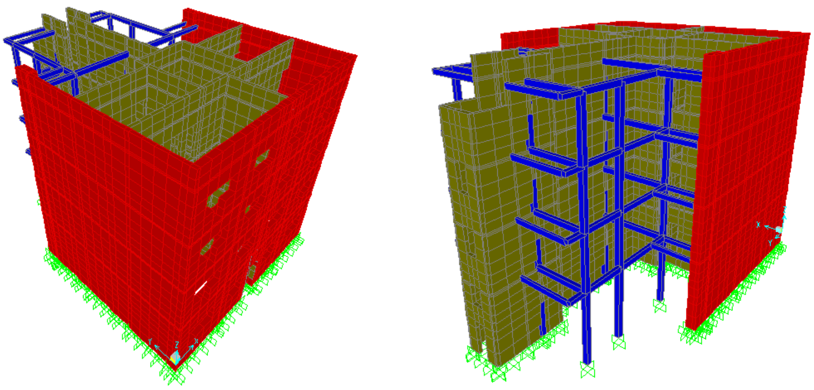


Figure 27 - Model without the exterior walls on the back to show the reinforced concrete frame.

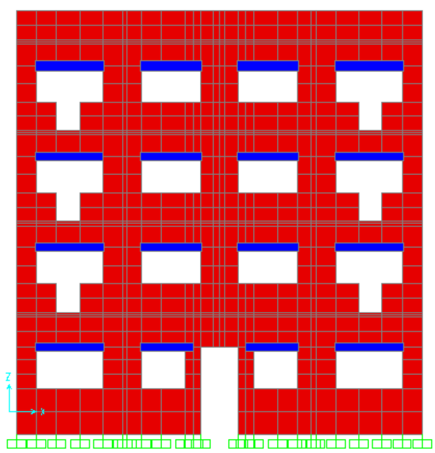


Figure 28 – Façade Model with the reinforced concrete lintels over the openings.

2.5 3D Model

After defining materials, the structural elements of the building and the values of the mass, the model has been analysed, in a general way. Figure 29 and Figure 30 show various views of the model, to provide a general insight of the model defined.

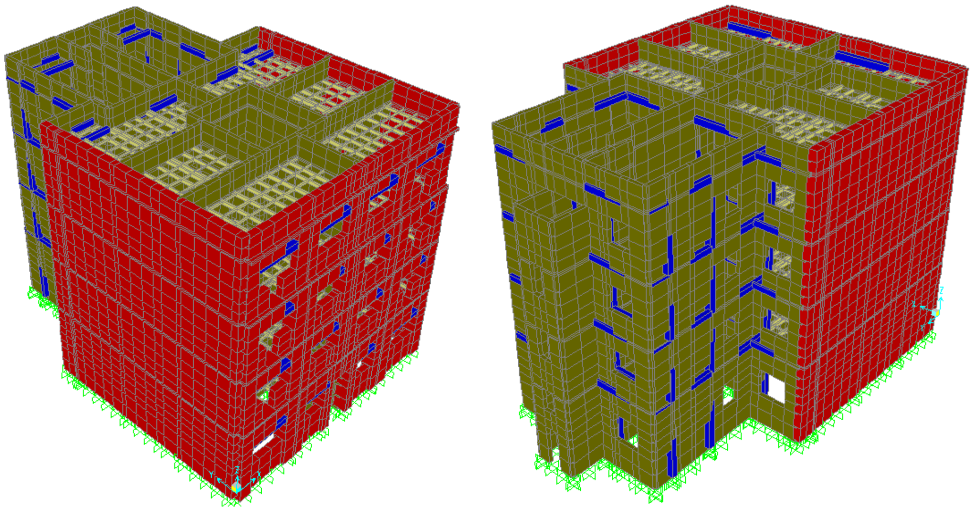


Figure 29 – 3D Model perspective.

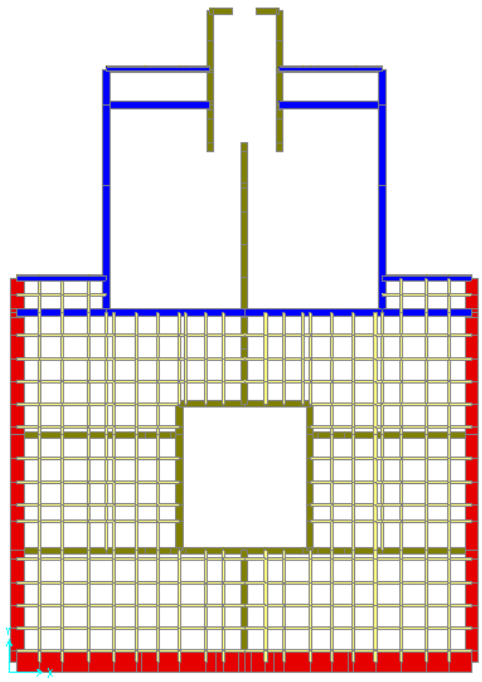


Figure 30 - Plant model of the building.

The foundations were considered fully restrained, because most of the settlements have already occurred and the land has good features according to the geological map of the area (e-Geo).

The structure of the roof was not considered on the model. However it was considered the influence of its mass in the dynamic analysis.

Furthermore, it is important to refer that both stairs (main and the secondary) were not considered in the model once the first is constructed of wood and the second is made iron and it is in the outer contour of the building.

Moreover, the decorative elements of the façade and the balconies also were not considered in the model.

3. DYNAMIC CHARACTERISATION STUDIES – IN-SITU AMBIENT VIBRATION TESTS

3.1 Introduction

The model described in the previous chapter aims to provide a good approximation of the real behaviour of the building. However some calibration is still need to define adequately some of the parameters assuming in the model, specially the current characteristics of the different materials used.

Thus, the focus in this chapter is given on the definition of the dynamic characteristics of the building, leading to the calibration of the numerical model, defined in the previous chapter. Based on the frequencies and mode shapes identified, in the experimental campaign, it was possible to improve the characterization of the dynamic structural system behaviour of the 'Placa' building, and therefore to calibrate the analytical model developed.

3.2 In-Situ Ambient Vibration Tests

As already mentioned, it is crucial identify the dynamic characteristics of the structure in study for a better calibration of the model created, with direct influence on the characterization of the seismic response. In this sense were carried out several of tests with the purpose to obtain the fundamental frequencies and the most relevant vibration modes, in order to perform the dynamic characterization.

The dynamic characterization tests consist on the analysis of data on the structure's response imposed by dynamic vibration of the environment or forced which not affect its integrity. The response is registered in terms of acceleration and it is possible to identify the frequencies through the Fourier transform of these signals, which correspond to the peaks of the Fourier spectrum obtained.

The ambient vibration records were obtained with triaxial episensor force balance accelerometer existing in the ICIST of Instituto Superior Técnico, configurable from a PC laptop (Figure 32) via the Quick Talk software (Kinemetrics).

The data processing allows the reading of the signals simultaneously on all channels (all the directions). Each channel has an accelerometer that records accelerations over time (Cardoso, 2002; Guerreiro and Azevedo, 2001; Guerreiro and Azevedo, 2004).

These devices are brand Kinemetrics model Etna, with internal sensors Episensor, whose specifications are listed in the following (Figure 31 and Figure 3.2):

- Dynamic range greater than 135dB;
- Bandwidth of the DC sensor to 200 Hz;

• Full-scale is selectable through hardware-between 0.25 g and 4.0 g (corresponding to the gravitational acceleration g) and are particularly adjusted for low frequency vibrations (0-10 Hz).

Figure 32 – Etna device (Kinemetrics)

Figure 31 - Equipment connected to computer, which read the signals of acceleration in each direction.

The tests were performed on 23 of February 2012, starting at 3 pm. Figure 33 shows the location of the tests on the plant of the top floor. The equipment was placed in some places with some eccentricity relative to the center of the building for registering the most important modes of vibration including the torsion. The device had the X-axis direction parallel to the façade, the Y-axis was perpendicular to the façade and the Z-axis corresponded to the vertical.

During the testes were made eighteen vibration records, each with duration of 120 seconds, induced by environmental noise, caused by the vehicles on the street. The designations of each record are shown in the Table 4 and in Figure 33.

Table 4 - Characteristics of the tests carried out.

Record	Local	Record	Local	Record	Local	Record	Local	Record	Local
GD001	A	GE001	B	GF001	C	GG001	D	GH001	F
GD002	A	GE002	B	GF002	C	GG002	D	GH002	F
GD003	A	GE003	B	GF003	C	GG003	E		
GD004	A	GE004	B	GF004	C	GG004	E		

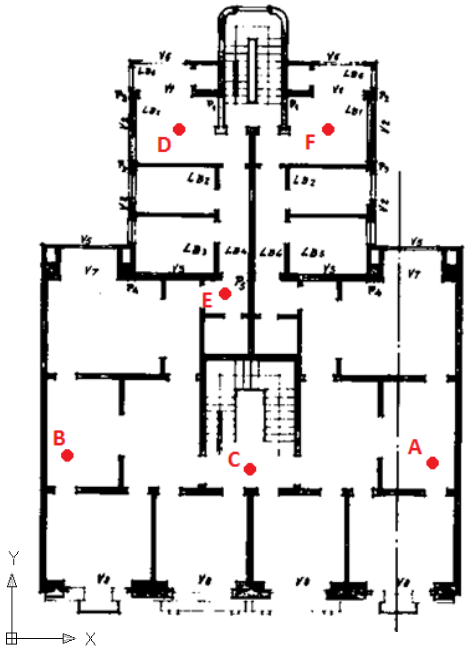
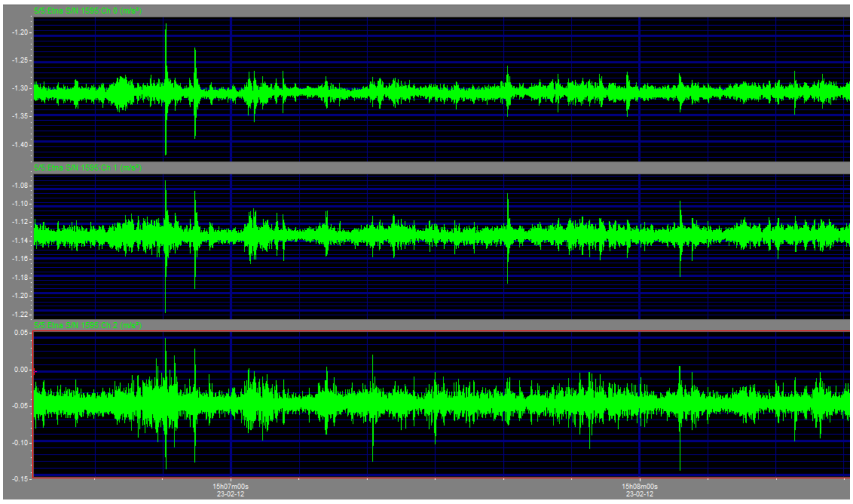


Figure 33 – Places on the last floor where it obtained the records

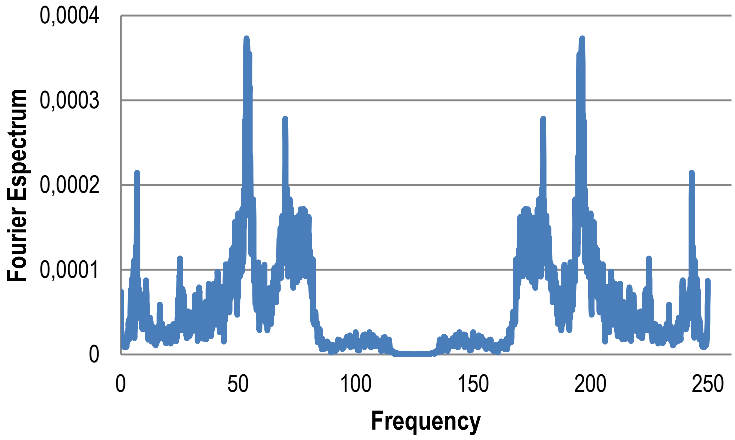
The following graph (Graph 1) shows an example of the GE002 recorded accelerogram for each channel. The signals from the other tests are similar. The program used for reading the accelerograms called TSoft (TSoft).



Graph 1 - Accelerogram recorded for the test GE002: the signals correspond to channel of the X-Y-Z direction by this order; the maximum amplitude is about 0.1 m/s²

3.3 Experimental Results

Based on all registered signals the results obtaining the Fourier spectrum that identified the natural frequencies for each direction were processed. The concepts and calculations made are shown in annex (Annex 2) (Branco, 2007). The example of the Fourier spectrum for X-direction based in all the records is shown in the graph below (Graph 2).

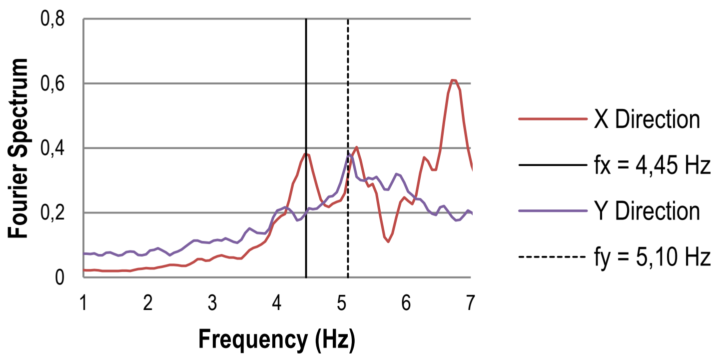


Graph 2 – The Fourier spectrum for the record in the X channel.

The graphs show symmetry in relation to 125Hz because it is related to the use of the algorithm FFT (Fast Fourier Transform).

In the Z-direction only were recorded very high excitation frequencies, as expected, which correspond to vibration modes with small contribution of the mass, caused by vertical translations of the floor. As this direction has little practical interest to dynamic characterization of the building, it will only be analysed in greater detail the results from the X and Y channels.

The Graph 3 represents the overlap of both Fourier spectra (X and Y directions) for the range between 1 to 7 Hz, which corresponds the main frequencies of the building under study.



Graph 3 – Fourier Spectrum in each direction between the 1 and 7 Hz.

Based on the graph 3 is possible to conclude that the fundamental frequency is 4.45Hz in X-direction and it is 5.1Hz in the Y-direction. Furthermore, and although this may not be clearly noticed in the graph, there is a mode corresponding to a vibration torsion phenomenon of 5.85 Hz.

At first sight, these values seem very high, even if they are compared with other studies for masonry buildings of Lisbon, which are on the middle of aggregate buildings, such as 'Pombalinos' buildings or 'Gaioleiros' buildings. Usually, according to some studies (Branco, 2007; Oliveira, 2009; Jesus, 2007) this type of buildings (Pombalino and Gaioleiros) has the fundamental frequency around 2 and 3 Hz.

However, one may take into account the fact that the building in study has a different structure when compared with the previous buildings of masonry in Lisbon.

The fact that this building is on the middle of aggregate buildings is not the only reason that justifies the high frequency values. In the 'Placa' buildings, the appearance of the reinforced concrete in the structure (essentially at the level of floors) and the substitution of the wooden walls by walls of brick masonry, have a great influence on the dynamic response of the building.

Although there is no study about specific 'Placa' buildings, according to Oliveira (2004 and 2010) there are some documents present several studies about the fundamental periods of vibration of some buildings in Portugal from in-situ experimental tests (Table 5).

Table 5 - Average fundamental frequency of some buildings which were studied. (Oliveira, 2004)

Types of buildings according the time of construction	Number of Floors	Average fundamental frequency [Hz]
Before 1900	5-7	2.3 – 3
1900-1940	4-7	3.7 – 3.8
1940-1950	3-6	5 – 5.5

This study shows that this type of masonry buildings was becoming stiffer than the previous one, mainly due to the introduction of reinforced concrete elements on the buildings.

Furthermore, there are some studies that show some characteristics of buildings around the same time of construction and with the same type of structure, which supports the frequency values, obtained in the dynamic characterization tests (Annex 3).

3.4 Calibration of the Numerical Model - Sensitivity Analysis

The model described in the previous chapter is intended to provide a good approximation of the current behaviour of the building. However, there is always the possibility of some parameter could be not adequately defined, essentially as far as the characteristics of the materials concerned.

Thus, the calibration of the model was performed, aiming to reach the dynamic characteristics of the building, i.e. the frequencies and mode shapes identified experimentally.

The calibration had special focus on the definition on the modulus of elasticity (Young Modulus) for masonry. The masonry is the material with the most variable proprieties because is a heterogeneous material, anisotropic and discontinuous, and besides that, its properties are greatly conditioned by the construction technique and the conservation status. After to consider several options and performed a lot of studies it was chosen to increase modulus of elasticity of brick masonry from 1 to 1.3 GPa.

Since the frequencies obtained experimentally were high, it was considered relevant to the study the building take into account in the model the influence of adjacent buildings, which somehow increase the stiffness due to the quarter effect. So in the model, the building has been replicated for each of the sides with the same properties (Figure 34).

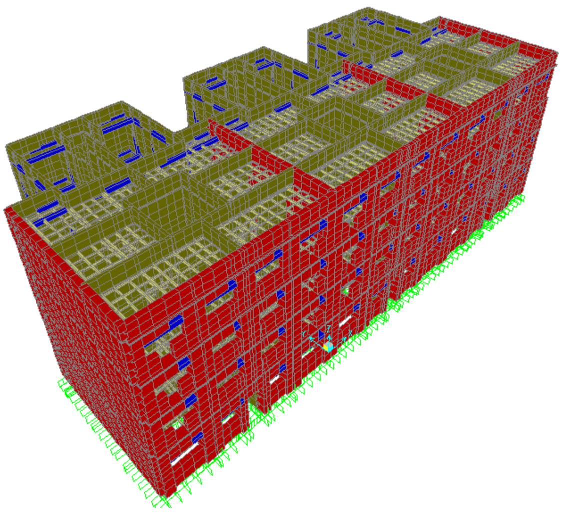


Figure 34 – The model with the adjacent buildings in each side.

Table 6 presents the comparative results obtained with the two models previously referred (isolated model and the aggregate buildings). The latter leads to values closer to the experimental values.

Table 6 - Comparison between the fundamental frequencies of the models and experimental tests

Description	**Ux [Hz]**	**Uy [Hz]**	**Rz [Hz]**	**Error f/f_{exp} [%]**	**Error f/f_{exp} [%]**	**Error f/f_{exp} [%]**
Experimental	4.45	5.10	5.85	1.00	1.00	1.00
Isolated Model	3.88	5.14	5.76	0.87	1.01	0.98
Aggregate Buildings	4.12	5.03	5.24	0.93	0.99	0.90

The model considering the adjacent buildings was chosen as the final model, and is this model that the seismic study was developed. As expected, the buildings adjacent introduce rigidity on the model in the longitudinal direction approaching the fundamental frequency to the frequency obtained in the experimental tests.

4. SEISMIC ASSESSMENT OF THE 'PLACA' BUILDING - LINEAR DYNAMIC ANALYSIS

4.1 Introduction

This chapter describes the results obtained by means of linear dynamic response spectrum analysis.

It begins by characterizing the dynamic behaviour of the building in terms of frequency and vibration modes. Then it is verify the seismic safety assessment of the building and the results are presented for the seismic load combination and analysing and discussed in terms of displacements and stresses, in several structural elements of the building.

4.2 Seismic Action Definition

The response spectra are obtained in accordance with the provisions on Eurocode 8 and the National Annex (2009). Reflecting the two types of seismic sources (interplate and intraplate) there are two different zonings in Portugal continental that are included in the Portuguese annex of EC8 (2009). So, in this way, were considered two types of earthquakes, because the building responds differently to each one. The design spectrum is defined by the following expressions, as presented in EC8 - 3.2.2.2:

$$0 < T < T_b:\ S_d(T) = a_g.S.\left[\frac{2}{3} + \frac{T}{T_b}.\left(\frac{2,5}{q} - \frac{2}{3}\right)\right] \quad \textbf{(Eq. 3)}$$

$$T_b < T < T_c:\ S_d(T) = a_g.S.\frac{2,5}{q} \quad \textbf{(Eq. 4)}$$

$$T_c < T < T_d:\ S_d(T) = a_g.S.\frac{2,5}{q}.\left[\frac{T_c}{T}\right] > \beta.a_g \quad \textbf{(Eq. 5)}$$

$$T_d < T:\ S_d(T) = a_g.S.\frac{2,5}{q}.\left[\frac{T_c.T_d}{T^2}\right] > \beta.a_g \quad \textbf{(Eq. 6)}$$

Where:

$S_d(T)$ is the design spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I.a_{gR}$), function of the importance of the structure (γ_I) and of the value of the ground acceleration reference (a_{gR});

T_b is the lower limit of the period of the constant spectral acceleration branch;

T_c is the upper limit of the period of the constant spectral acceleration branch;

T_d is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

q is the behaviour factor;

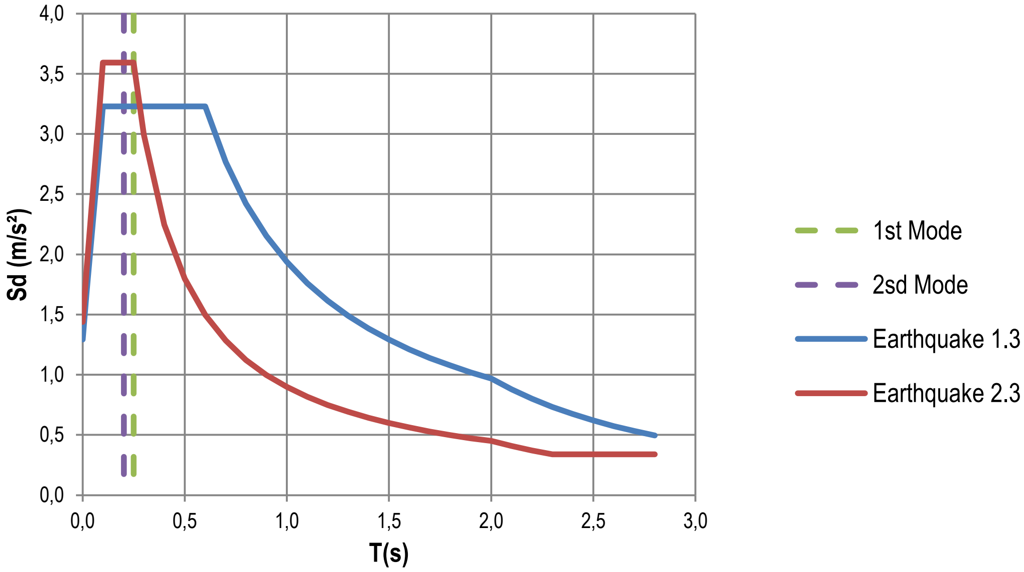
β is the lower bound factor for the horizontal design spectrum.

The aforementioned parameters depend on several factors including the type of soil of foundation, its geographical location and the material predominantly used.

As already mentioned, the building is located in Lisbon, and therefore in accordance with Portuguese annex of EC8 (2009) it is in the seismic zone 1.3 and 2.3, for the earthquake type 1 and 2, respectively. The area in question has properties related to the ground type C and the building is considered to belonging to the common importance class of buildings (class II: γ_I = 1.0).

In relation to existent structures, Part 3 of Eurocode 8 (2005), this document does not refer behaviour factor (q factor), which must to be used. However, if there is not any kind of recommendations, it may be adopts the value of 1.5 for this coefficient to the side of safety (EC8.3, 2005). So the building is limited to a minimum of energy dissipation, which is consistent because many of these structural elements in masonry are neither reinforced or have not any concern in terms of its design seismic (Lopes et. al, 2004).

Thus, the required parameters for the both response spectra were calculated, which are illustrated in Graph 4.



Graph 4 - The design spectrum for the both earthquakes

By analysing the graph, it is observed that the earthquake 2.3 is the determinant earthquake according to the fundamental periods of the structure (the periods correspondent to the first and second modes are depicted in the graph).

In this study the CQC combination for the modal combination and SRSS combination for the directional combination were adopted.

The combination of actions used to assess the level of stress and displacement of the structural elements to Ultimate Limit State is defined in the following equation:

$$S_d = 1 \times S_{CP} + \Psi_2 \times S_Q + 1 \times S_S \quad \textbf{(Eq. 7)}$$

Where:

S_{CP} Effects due to the action of permanent loads;

S_Q Effects due to the action of overloads distributed;

S_S Effects due to seismic action;

Ψ_2 Factor for quasi-permanent value of a variable action ($\Psi_2 = 0.3$).

4.3 Dynamic characteristics of the building

As mentioned in the previous chapter the model chosen was the one that considers the influence of adjacent buildings. For better understanding the model the most relevant modes of vibration in the building are presented, as shown in the Table 7 and from Figure 35 to Figure 38.

Table 7 - Frequencies and mass participation of the main modes of vibration of the building model.

Modes	Period	Frequency	Translation in X		Translation in Y		Rotation in X	
	[s]	[Hz]	%	Σ %	%	Σ %	%	Σ %
1	0.24	4.12	0.64	0.64	5.1E-09	5.1E-09	0.24	0.24
2	0.20	5.03	1.8E-08	0.64	0.70	0.70	0.12	0.36
3	0.19	5.24	3.6E-04	0.64	4.9E-08	0.70	0.32	0.68
4	0.18	5.67	2.0E-07	0.64	4.9E-03	0.70	8.2E-04	0.68
5	0.16	6.18	0.09	0.74	5.7E-09	0.70	0.02	0.69
6	0.14	7.00	0.00	0.74	8.1E-09	0.70	0.01	0.71

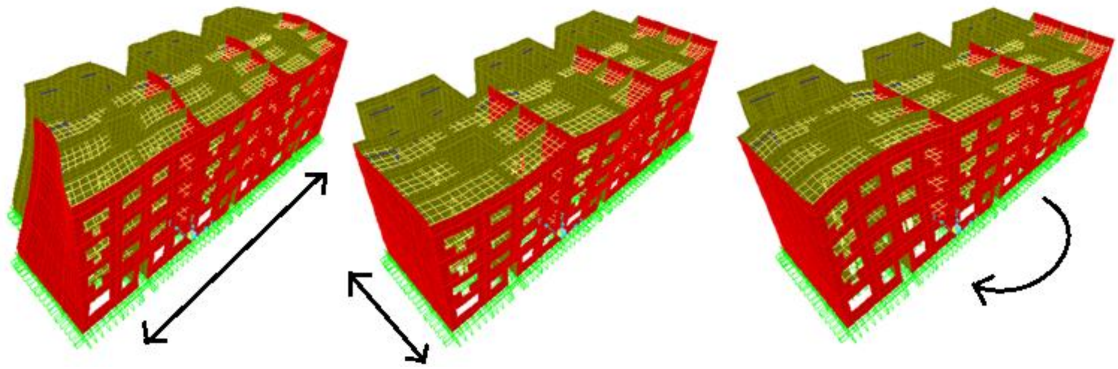


Figure 35 - The first three vibration modes of the model.

The first mode has a natural frequency of 4.12 Hz and has a participation of mass around 65%. This vibration mode is an overall translation of the building according to X-direction, because is the direction which has a smaller rigidity due to having only the façade wall (with openings) as a resistant wall.

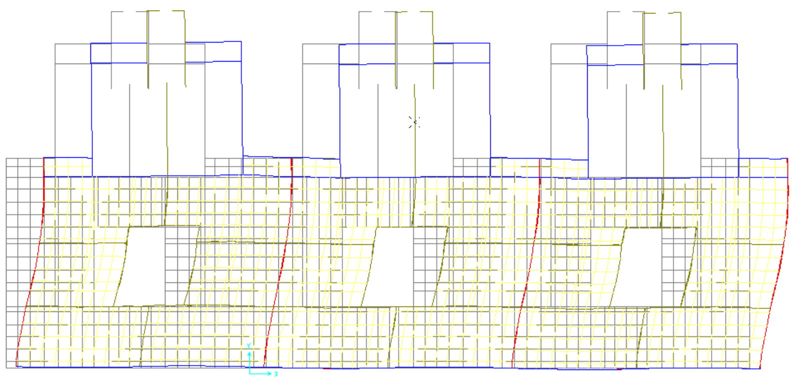


Figure 36 - The first vibration mode of the model.

The second mode of vibration occurs for a natural frequency of 5.03 Hz and also has a high participation of the mass, around 70%. This vibration mode is an overall translation of the building according to the Y-direction, which corresponds to the direction more rigid, due to the presence of the gable walls.

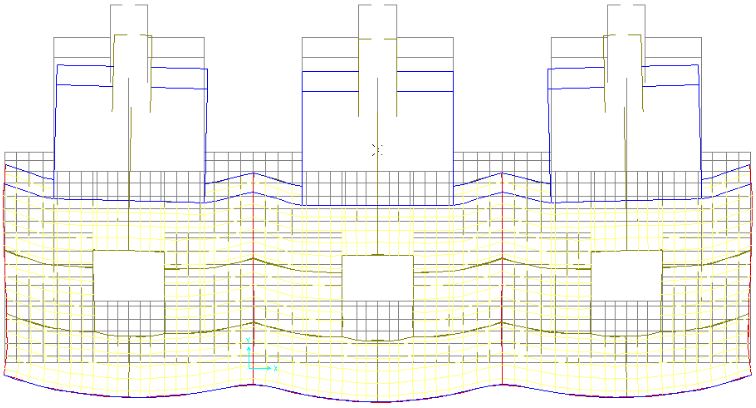


Figure 37 - The second vibration mode of the model

Finally, the third mode vibration is a global torsional mode with a frequency of 5.24Hz but only with 30% participation of mass, with the effect of aggregate buildings restricts the rotation capacity.

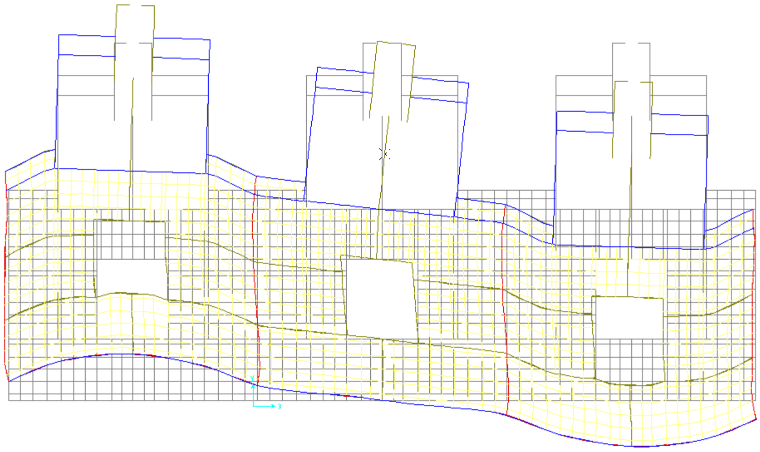


Figure 38 – The third vibration mode of the model

Another important aspect to consider is it is worth to take, or not, the effects of accidental torsion in the model of the building.

It is not possible to treat the aggregate building as if it is a single large building in plant because part of the floors is not rigid. However the sharing of gable walls on the adjacent buildings strongly reduces the overall torsional effects at each building, as the buildings cannot rotate separately.

Thus, as the building will be analysed considering the quarter effect, there is no need to consider the additional torsional eccentricities, as defined in the Eurocode 8 (2009) for analysis of structures.

4.4 Influence of the Quarter in the Building Response

Firstly, the effect of the quarter due to the adjacent buildings was assessed comparing the displacements of both, the building inserted in the quarter or isolated. The objective is to determine the influence of the adjacent buildings (which represent the quarter) on the response of the building under study.

To better interpret the results, in the Figure 39 is identified of the alignments where were recorded the displacements.

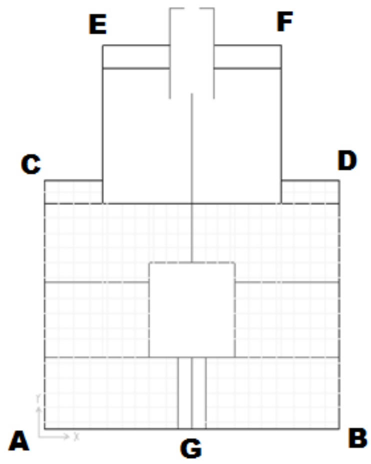
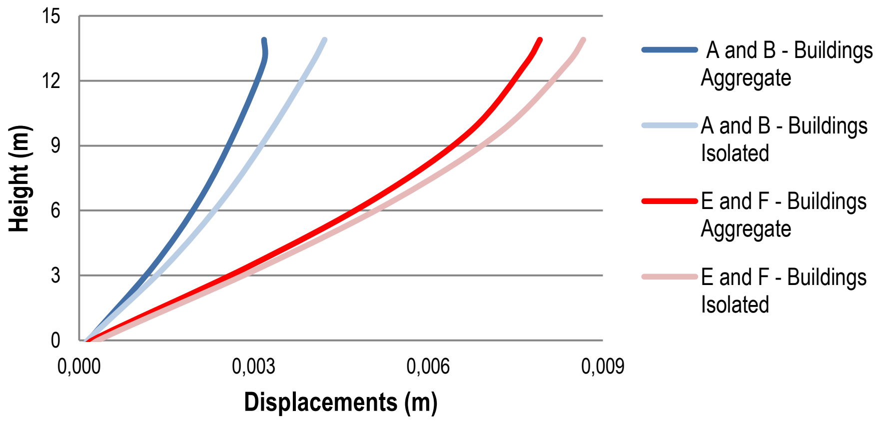
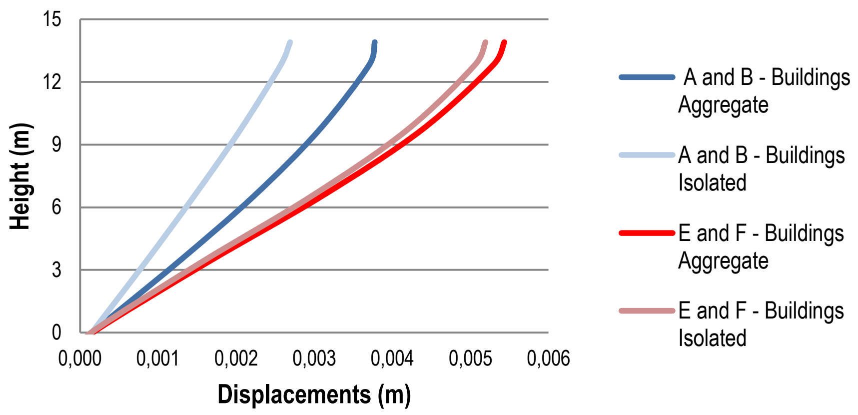


Figure 39 – Places in plant where they were registered values of the displacements in height.



Graph 5 – Displacements in X direction for each model considering the building isolated and the building aggregate.



Graph 6 - Displacements in Y direction for each model considering the building isolated and the aggregate buildings.

Taking into account only the difference on the behaviour of the structure isolated and the one inserted on the quarter, from Graph 5 can be observed that the horizontal displacement in the longitudinal direction (alignment of front façade) is larger when the structure is considered isolated, as expected.

The reduction of deformation in the x direction considering the effect of quarter is about 25% in the alignment of the front façade, and 9% for the alignment on the back façade. Therefore, is natural that the reduction of displacements is greater on the alignment of the masonry façade walls because are longer and more rigid, while on the back, the building continues to works almost without any restriction, even considering the effect of the quarter.

The increase of the displacements on the transverse direction of the building (Graph 6), considering the effect of the quarter, can be explained by the fact that the increase of the inertia in this direction is no greater in the same proportion than the rigidity increase of the gable walls.

Besides this, the model does not correspond exactly to the reality because it does not consider all the buildings that complete the quarter, mainly those located in the opposite direction, which will increase the stiffness in this direction.

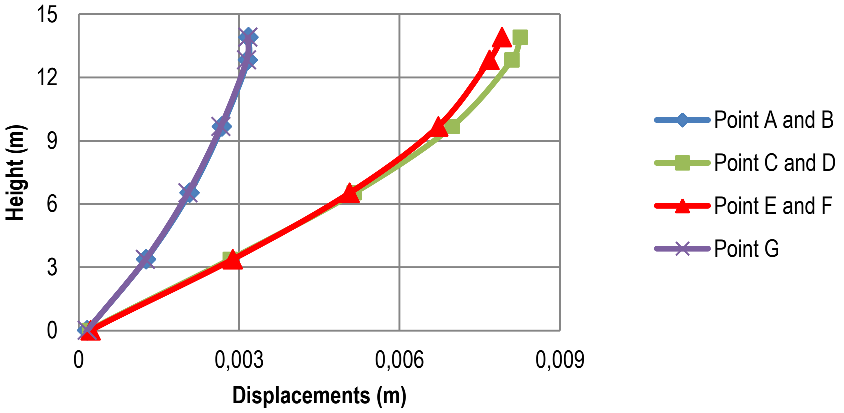
4.5 Seismic Assessment

According to EC8-3 (2005), the seismic safety assessment is a quantitative process that checks if an existing structure (damaged or not) is capable to resist to the design seismic combination and ensures, at the same time, an adequate level of damage. This means that the performance requirements are met.

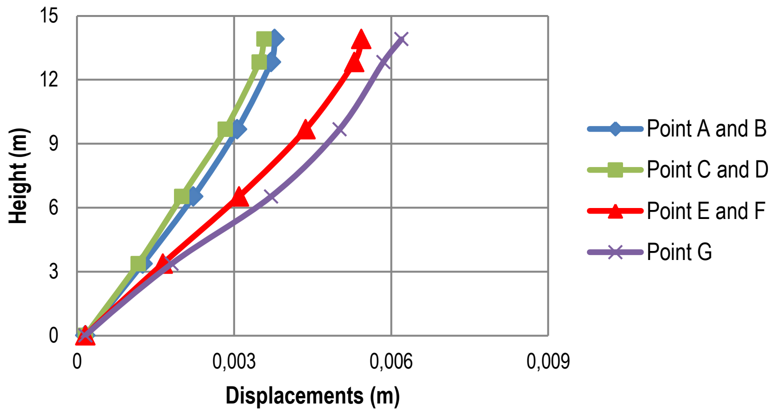
The performance assessment includes a study of the displacements in reference points, a survey of the strengths and stresses in several structural elements and then, the results are compared with the strength values of the materials, previously defined in the Table 2.

4.5.1 Displacements

In the following graphs (Graph 7 and Graph 8) are indicated the displacements in different points at the building plant, according the Figure 39, along the height and in both directions of the model adopted for the study.



Graph 7 – Displacements in X direction for the model adopted.



Graph 8 - Displacements in Y direction for the model adopted.

The analysis of previous graphics (Graph 7 and Graph 8) allows outline some conclusions:

First, only looking for the displacements on aligning of the façade wall (point A, B and G) the graphs show that the values are inferior in the longitudinal direction, since that is more rigid direction, with façades longer and stronger, which restrict the global deformation.

However, it is in the point G theret is truly remarkable this difference, where the displacement in the direction perpendicular to the front façade is about twice the displacement in the direction parallel to the façade wall. In addition to the reason mentioned abovementioned, this is mainly due to the fact that the wooden floor is not rigid in plan and therefore the point in the middle of the front wall will have a greater displacement for out-of-plane of the façade, because is quite distant from the resistant walls (gable walls).

In relation to the values of displacements at the back of the building (point E and F), it can be observed that these are higher than the displacements in alignment of the front façade (except for point G), since this part of the building works almost like an isolated body, that has only one constraint which connects the central body, and it has more mass on the floors due to the concrete slab (i.e. higher inertia forces). So this body on the back is more flexible and therefore has greater displacement, where the displacements in the x-direction are greater than the y-direction.

The displacements assessment includes determining the horizontal displacement in vertical alignments close to the center of the façade, since are the areas more distant of the bracing walls and resistant walls.

In accordance with the recommendations in EC8-1 (2009) for the general requirement of "limited damage " a set of limits to the relative displacement between floors must be met according the following limits:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$$d_r v \leq 0.005h;$$

b) for buildings having ductile non-structural elements:

$$d_r v \leq 0.0075h;$$

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$d_r v \leq 0.01h$$

Where:

d_r is the design interstorey drift;

h is the storey height;

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement, and in this case $v = 0.55$ (EC8.1 NA; 2009).

After determined the relative displacements between floors and taking account the limits recommended in EC8-1 (2009), the requisite of "damage limitation" has been satisfied as established by the limit defined in a) in the code.

Then, it was decided to calculate the "interstory drift" from the ratio of the relative displacement between two consecutive floors and the floor height. This parameter, expressed as a percentage, will subsequently be compared with the limits imposed by the American Code FEMA 356/357 (2000) that is more demanding and it establishes a classification on the state of degradation for a unreinforced masonry structure, as shown in Table 8.

Table 8 - Structural Performance Levels and Damage for Unreinforced Masonry Infill Walls (FEMA 356, 2000) and (FEMA 357, 2000)

Structural Performance Levels	Description	Interstory Drift
Immediate Occupancy	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.	0.1%
Life Safety	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	0.5%
Collapse Prevention	Extensive cracking and crushing; portions of face course shed. Some walls dislodge.	0.6%

Thus, in Table 9 is shown the interstory drift in the two kinds of floor that there is the building.

Table 9 – Interstory drift in each type of floor and in each direction

	Wooden Floor		Concrete Floor	
	Ux	Uy	Ux	Uy
R/C	0.07%	0.05%	0.09%	0.05%
1st floor	0.06%	0.05%	0.08%	0.05%
2nd floor	0.05%	0.04%	0.06%	0.04%
3rd floor	0.02%	0.02%	0.04%	0.03%

By analysing the table above, it is observed that the limits imposed by the regulation are not exceeded. This is mainly due to the high stiffness of the building, with displacements values very low.

Anyway, it can be observed that the lower floors are the floors, which have more displacements, mainly in the longitudinal direction and in the area of the concrete floor, as expected.

4.5.2 Strengths and Stresses in Structural Elements

In this section are presented the values of the stresses obtained for the walls and for other structural elements (wooden floor and the reinforced concrete frame) and then some comparison with strengths values is done. For each wall, first is presented the stress distribution corresponding to the vertical loads and then the results obtained for the seismic load combination.

4.5.2.1 Façade Walls

Figure 40 to Figure 45 depict the distribution of vertical and shear stresses in the façade wall. The stresses values shown in all figures are in kPa.

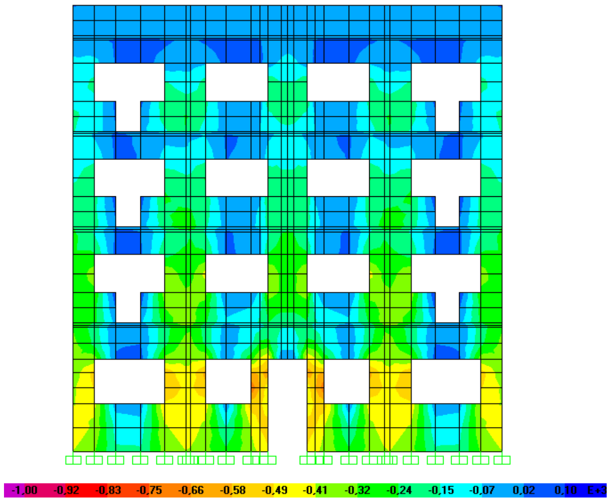


Figure 40 - Vertical stress diagram for the masonry of the façade due to the quasi-permanent loads.

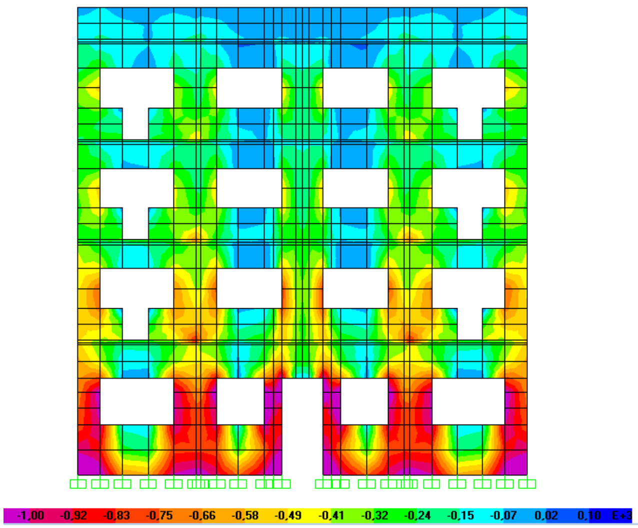


Figure 41 - Vertical stress diagram for the masonry of the façade due to the seismic load combination (maximum compression stress)

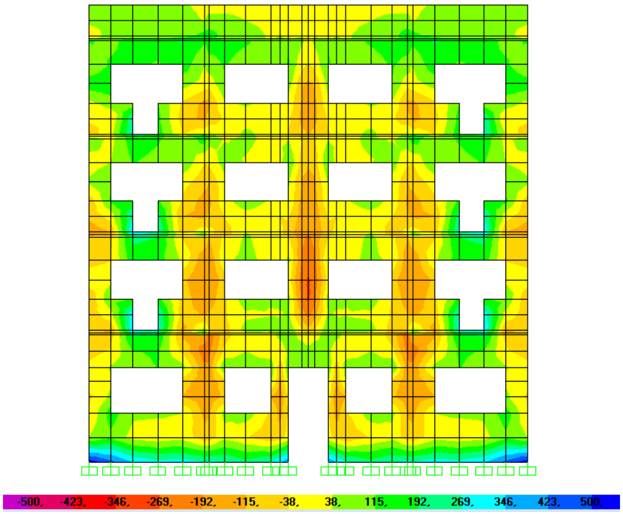


Figure 42 - Vertical stress diagram for the masonry of the façade due to the seismic load combination (maximum tensile stress)

The stress analysis in the masonry elements of the façade shows that for the vertical loads (Figure 40), the stress values are not relevant, so they can be ignored.

Analysing the distribution of stresses due to the seismic load combination, there are considerable compression efforts, particularly at level of the vertical elements on the ground floor façade (Figure 41). However, in this case the maximum stress corresponds to approximately 35% of the compressive stress capacity defined for the limestone and therefore is still far of the resistant values considered (4 MPa, Table 2).

The use of masonry as a structural element is essentially based on its compression functioning. But it is also necessary to consider the tensile and shear stresses, which can be installed in the structural element due to the eccentricity of vertical load with the actuation of the seismic load.

In general, according with the distribution of stresses in the masonry presented in the previous figures, there is a stress concentration close to the openings. In fact, the corners of the doors and windows are areas of stress concentration because of the discontinuities that existing in the façade (Figure 42). This zones corresponding to the areas where the cracking is expected.

In the vertical elements of the façades at the level of the ground floor there are considerable axial tensile stresses (Figure 42) but which do not exceed the resistance values (just 60% of the total). Although it is considered that the joints between the stone elements of masonry does not resist to tensile forces, the occurrence of tensile stresses do not mean occurring directly the collapse of the structure but only some tensile cracks in the joints.

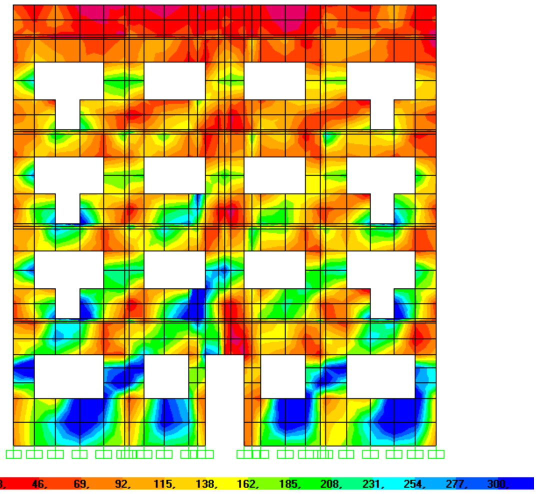


Figure 43 - Shear stress diagram for the masonry of the façade due to the seismic load combination.

The analysis of the shear stresses on the elements of the façade (Figure 43) shows that the stress resistance values are exceeded in the area of the ground floor between the openings and in the zone corresponding to the fictitious columns of masonry too. These values are three times higher than the maximum limits, so these elements reach their resistance values by more than half, for the seismic design load expected.

Most of the areas in the façade with high shear stress values are close to the openings in the two first floors, and the zones under the windows have a thickness smaller than the current thickness of the façade (0.35 cm instead of 0.70 cm). So these areas, as shown in the Figure 44 should have little importance for the resistance of the structure.

Thus, in a parallel study it was decided to remove the masonry elements in those areas of the model, ignoring their contribution in order to verify the difference in the behaviour of the shear stresses in the façade (Figure 44).

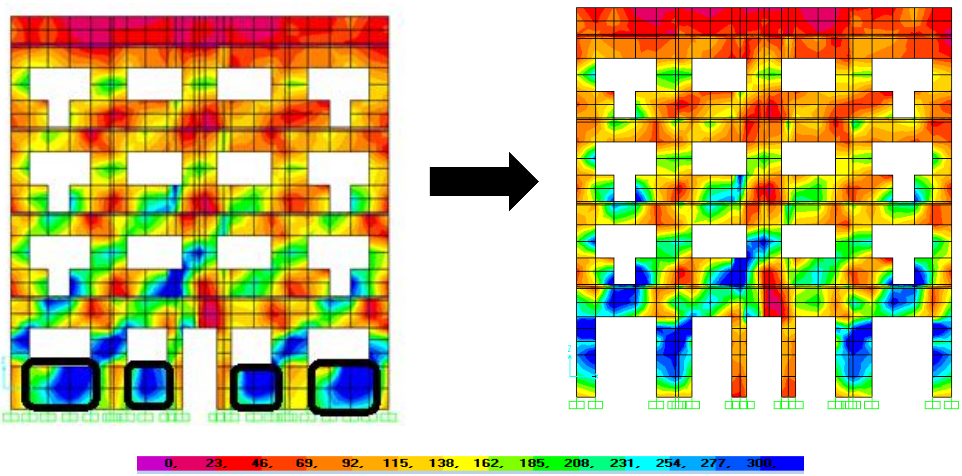


Figure 44 - Shear stress diagram for the masonry of the façade due to the seismic load combination – the areas with reduced thickness are neglected in the model.

However, the changes made in the model did not lead to significant improvements in relation to the analysis of shear stresses in the wall of the façade previously defined.

Besides, according to the Italian Seismic Regulation (2003, 2005) there is the possibility of reducing the level of seismic protection in cases of "rehabilitation interventions and reinforcement through a decrease in vulnerability of existing structures". In this case the code allows reducing the intensity of seismic load to 65% of the design one.

After reducing the intensity of the seismic action as suggested in Italian Regulation, a reduction of shear stresses around to 0.2MPa was verified, as shown in Figure 45. However the values are still higher in the base of the façade than the maximum strength (0.1 MPa).

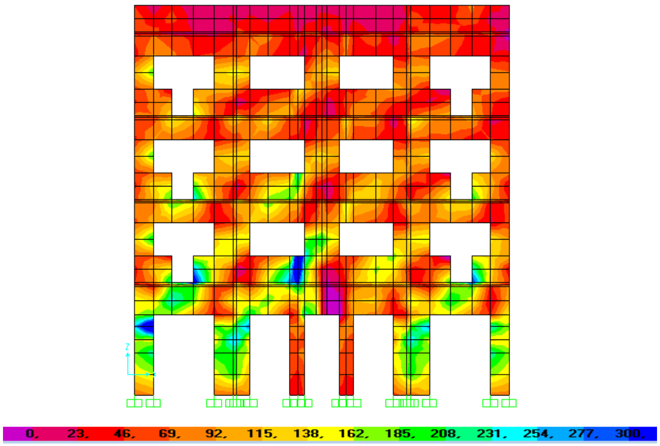


Figure 45 – Shear stress diagram for the masonry of the façade due to the seismic load combination with a reducing of 65% according the Italian Regulation.

The global collapse of the building will always depend on the reserve strength of these masonry elements as well as the real capacity of shear strength. The collapse or rupture by bending of the façades for out of your plan are the most common mechanisms of collapse in masonry buildings, and is usually due to poor quality of connection between the façade and to the gables walls (Cardoso, 2002).

4.5.2.2 Gable Walls and Exterior Walls on the Back of the Building

Other structural elements with major contribution to resist the horizontal loads are the gable walls of buildings, made of concrete blocks without openings. The analysis of shear stresses in these elements is presented in the following figures (Figure 46 to Figure 49).

In addition, the figures also show the stresses in the exterior walls at the back of the building, which are built of solid brick masonry.

Since the stress distribution in the both gable walls is equal, it was decided only show one side of the building.

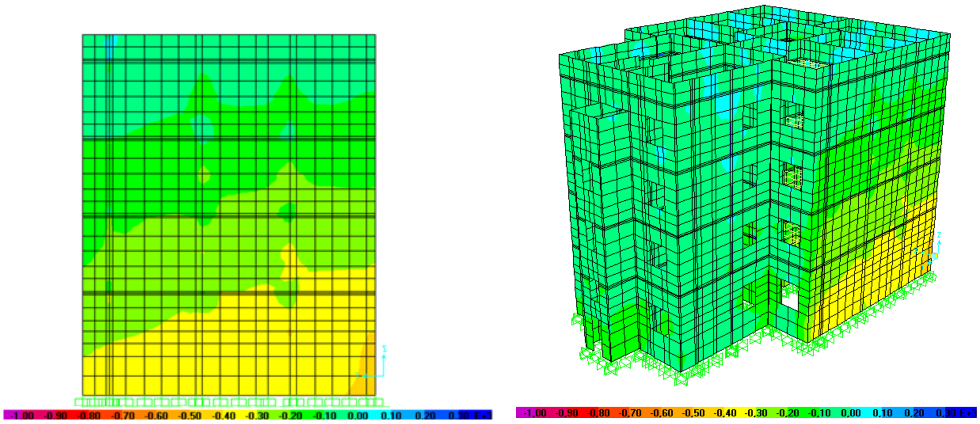


Figure 46 - Vertical stress diagram for the elements of the gable wall and exterior walls due to the quasi-permanent loads.

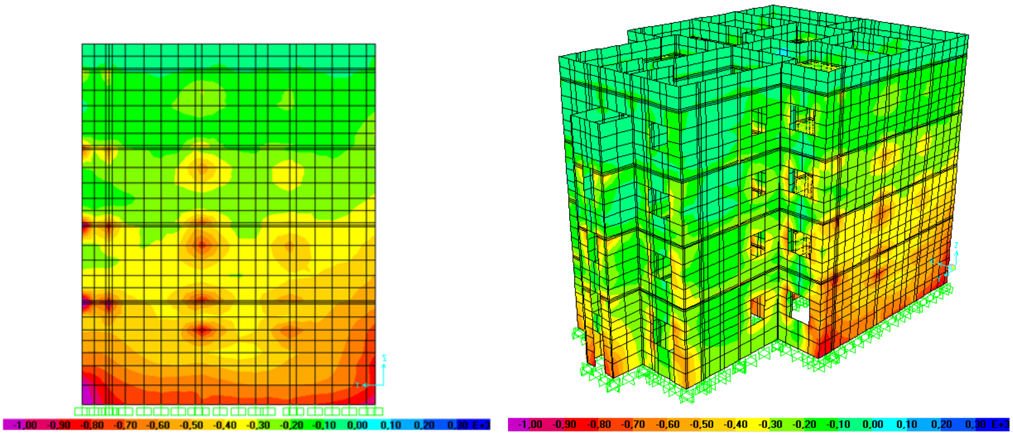


Figure 47 - Vertical stress diagram for the elements of the gable wall and exterior walls due to the seismic load combination (maximum compression stress).

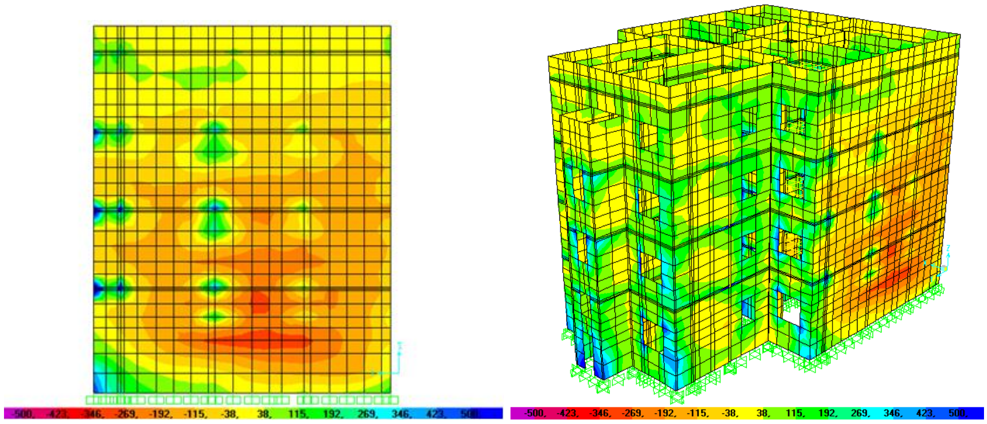


Figure 48 - Vertical stress diagram for the elements of the gable wall and exterior walls due to the seismic load combination (maximum tensile stress).

The previous figures (Figure 46 and Figure 47) show an increase of the vertical compression from the top to the base of the building, although there are some occasional disturbances due to connection between the interior walls to the gable wall and between the concrete beams to the exterior wall. However, the compressive stresses registered do not exceed the maximum capacity value (Table 2).

In general, the tensile stresses in the walls are reduced and do not exceed the maximum strength (with the exception of localized areas with no significant importance). Anyway, Figure 48 shows an increase of the stresses in the connection zone of the interior walls, also close to the windows and finally in the masonry elements near the reinforced concrete structure.

For the shear stresses, Figure 49 shows higher values than the resistant values in the area of the 1st floor in the gable wall.

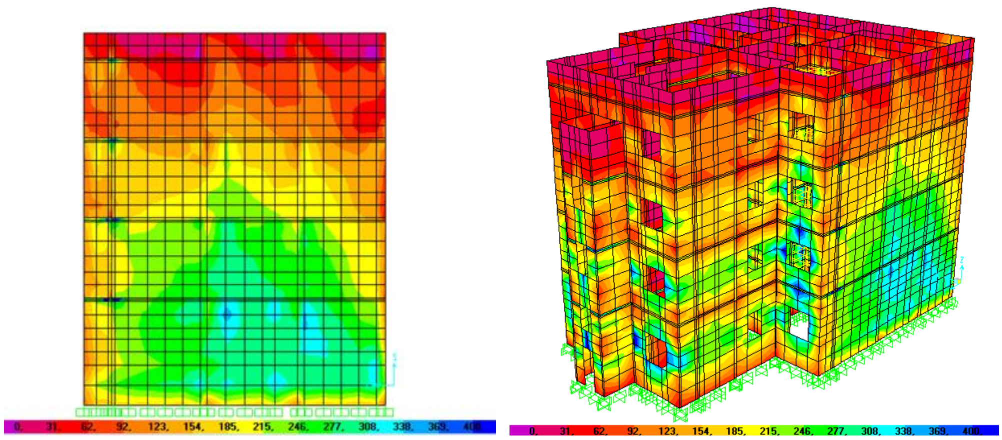


Figure 49 - Shear stress diagram for the elements of the gable wall and exterior walls due to the seismic load combination.

In this case, the first damage (cracking) in these elements will occur for lower values of the seismic intensity estimated. However, according to Oliveira (2009), the reduction in stifness of the gable walls associated with increase of the cracking due to increase of the seismic action, can give rise to the redistribution of the efforts in the structure, which by itself does not change significantly the behaviour of these elements.

Besides that, there are also high shear stresses in the exterior walls of brick masonry, mainly close to de windows that are near to the gable walls. However, in this part of the building there is also a reinforced concrete structure, so that it is not just the masonry walls which resisting to the seismic loads. The question remains whether the reinforced concrete structure is in good conditions and if it has the enough resistance.

4.5.2.3 Interior Walls

In this section are presented, from Figure 50 to Figure 53 the stress distribution in the interior walls of brick masonry. It is essential to refer again that these walls have a significant importance in the resistance of the structure, because there is not a load distribution of the floor due to the wooden floor neither vertical elements that supporting the vertical loads.

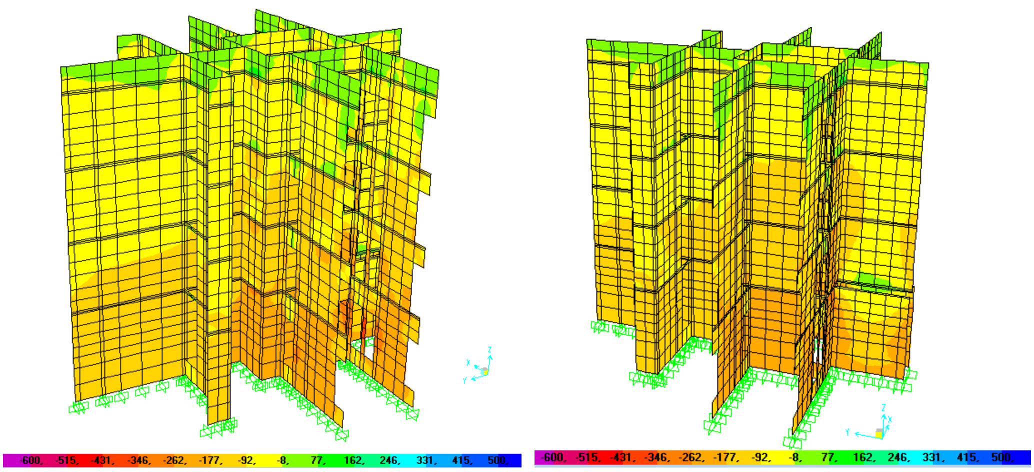


Figure 50 - Vertical stress diagram for the masonry of the interior walls due to the quasi-permanent loads.

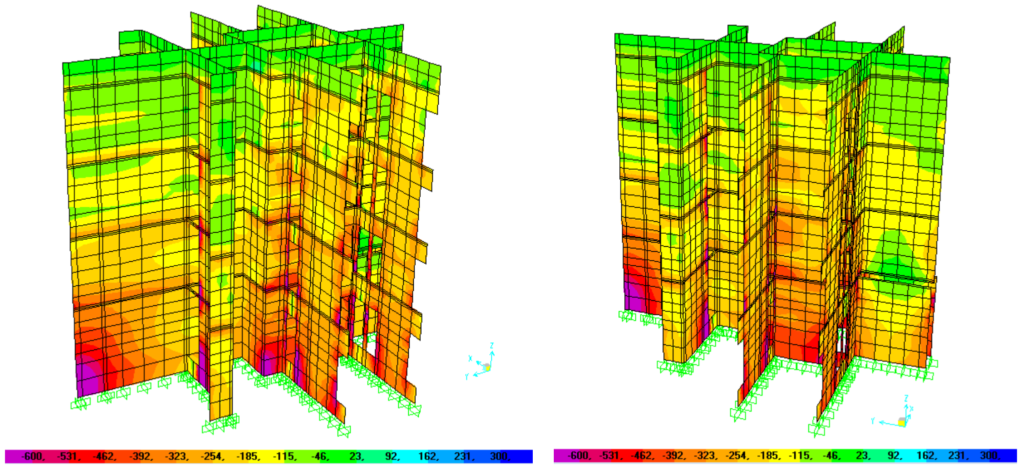


Figure 51 - Vertical stress diagram for the masonry of the interior walls due to the seismic load (maximum compression stress).

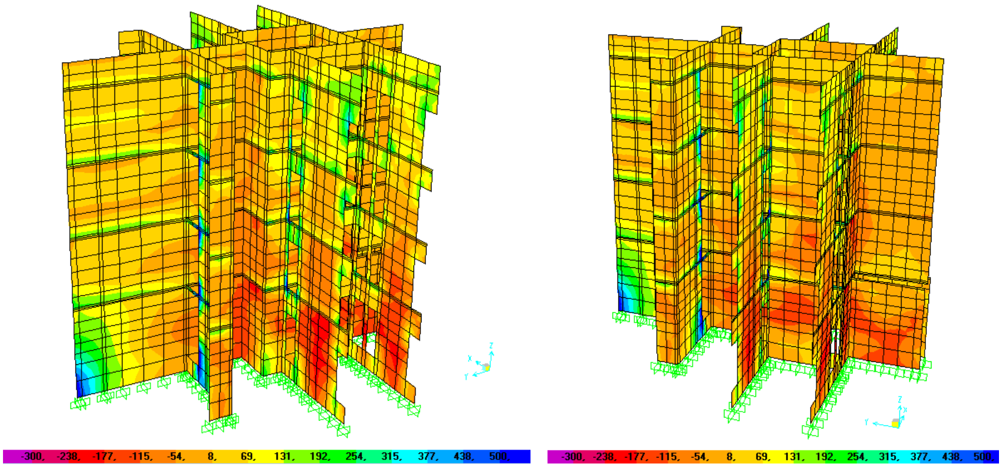


Figure 52 - Vertical stress diagram for the masonry of the interior walls due to the seismic load combination (maximum tensile stress).

The previous figures show that the interior walls with openings (windows and doors) are those that have higher stresses than the walls from the stairs.

In relation to the compression stresses, it is in the base of the walls that higher values are shown, as expected. However these values are smaller than the capacity ones. For the tensile stresses, there are high values in the area of the openings as well as the base of the walls; however these areas are not significant to the overall resistance of the structure.

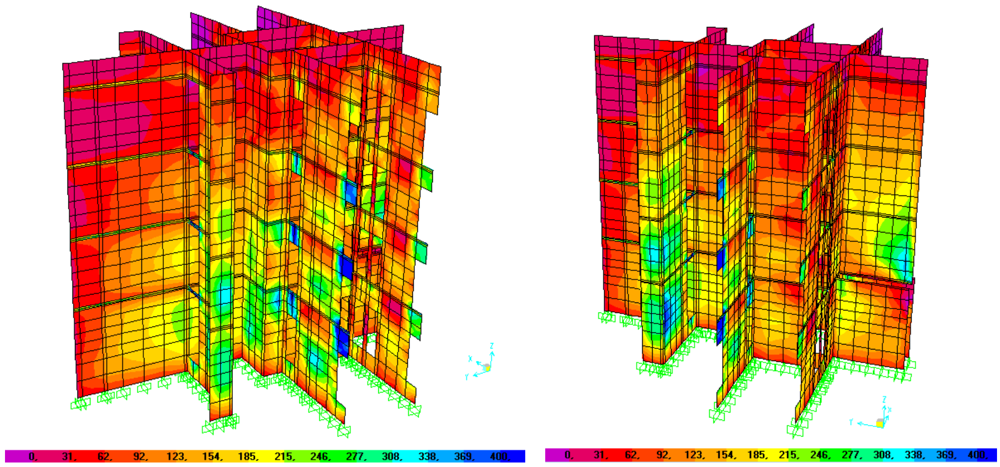


Figure 53 - Shear stress diagram for the masonry of the interior walls due to the seismic load combination

For the shear stresses, Figure 53 shows that the maximum values are exceeded mainly in the connection between the interior walls (parallel to the façade) to the gable walls. This happens essentially due to the weak connection between these walls because there are a lot of openings in these zones and the area of masonry that connects both walls is negligible.

This can be a real problem for several reasons: the connection area between the two walls is minimal, the connection quality between these perpendicular walls is not known, the wooden floor does not have a rigid behaviour in plan, so it cannot ensure the loads distribution, and finally the quality of the connection between the masonry and wood is also unknown.

4.5.2.4 Connections between the Elements of Timber Floor and Masonry Walls

The connection between the timber elements and the masonry walls was assessed for the direction perpendicular to the façade, since it is the direction of the timber main beams of the floors.

The strength of the connection depends essentially on the way and geometry between both elements in the contact area, as well the friction force that can be mobilized.

It is a brittle failure mode, because the yield strength corresponds to rupture. In this case, the adopted limit for the pull-out force was 10kN. The Table 10 shows the values of maximum forces in the connections along the height of the building.

Table 10 - Values of maximum forces in connections between the timber elements and masonry.

Floor	Maximum Force	% timber elements with more than 10kN
Coverage	21	44
3rd floor	21	55
2nd floor	20	44
1st floor	13	33

From the results presented in Table 10 it is observed there is a significant number of links that exceed their tensile strength for lower levels to the seismic intensity considered. Thus, it was decided realize again a parallel study, decreasing the intensity of seismic action for 65%, as suggested by the Italian code (2003 and 2005). The results obtained for this seismic intensity are presented in Table 11.

Table 11 - Values of maximum forces in connections between the timber elements and masonry (65% of seismic intensity).

Floor	**Maximum Force**	**% timber elements with more than 10kN**
Coverage	15	28
3rd floor	15	11
2nd floor	14	11
1st floor	10	0

Based on the results obtained, one can conclude that the percentage of timber elements exceeding the pull-out strength decreases more than half when is compared with the initially design response spectrum.

The analysis in the timber beams shows that the levels of force are considerable even with reduced intensity seismic. The maximum axial force that was obtained was 15 kN, in the roof, where the inertia forces are higher, verifying that the total proportion of elements with exceeded tensile stress is about 30%. Thus, it is concluded that the façade wall can be unsafe as far as to the out-of-plan collapse concerned. However, there are also the perpendicular walls that can keep the façade wall protected to the collapse.

4.5.2.5 Reinforced Concrete Structure

To verify the ultimate limit state, it was analysed if the strength capacity of structure is exceeded by means the internal forces (shear forces and bending moments) in the vertical structural elements and beams.

However, it is known that the best way to verify the seismic safety assessment of this existing reinforced concrete structure is by means of a static non-linear analysis (pushover analysis), since it is known the reinforcements in each element according the descriptive memory.

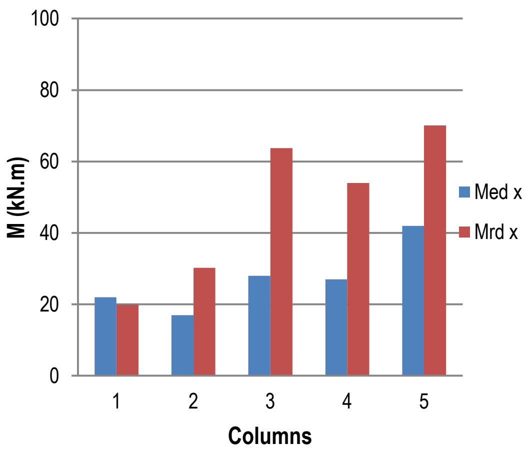
The results presented in the following are the largest absolute values obtained from the numerical model, considering the existence or not of the exterior walls of brick masonry.

Columns

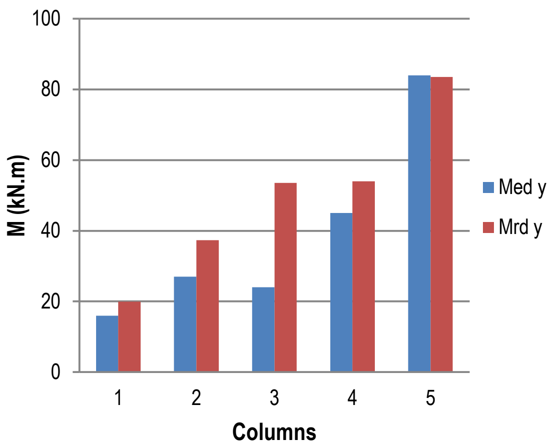
Table 12 shows the dimensions of the section and the reinforcements of each column. It is considered that these characteristics are constant in height (Descriptive Memory, 1939). In the following graphs (Graph 9 to Graph 11), it can be seen the difference between the values of the forces and moments observed from the building model with resistant values based on the reinforcement of each column.

Table 12 - Dimensions and reinforcements of each column (Descriptive Memory, 1939).

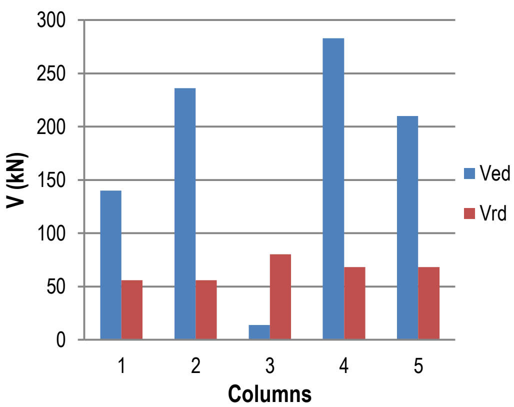
Column	b [m]	h [m]	Longitudinal Reinforcement	A_s [cm²]	Transverse Reinforcement	A_{SW}/s [cm²/m]
P1	0.25	0.25	4Φ1/2"	5.08	2Φ5/16" spacing by 15 cm	6.60
P2	0.3	0.25	4Φ5/8	7.96		
P3	0.3	0.35	4Φ3/4"	11.48		
P4	0.3	0.3	6Φ5/8"	11.94		
P5	0.35	0.3	8Φ5/8"	15.92		



Graph 9 - Bending moments in x-direction



Graph 10 - Bending moments in y-direction



Graph 11 - Shear force

From the analysis of previous graphs, in general, it can be stated that the bending moments are lower than the correspondent maximum capacity of each column (Graph 9 and Graph 10).

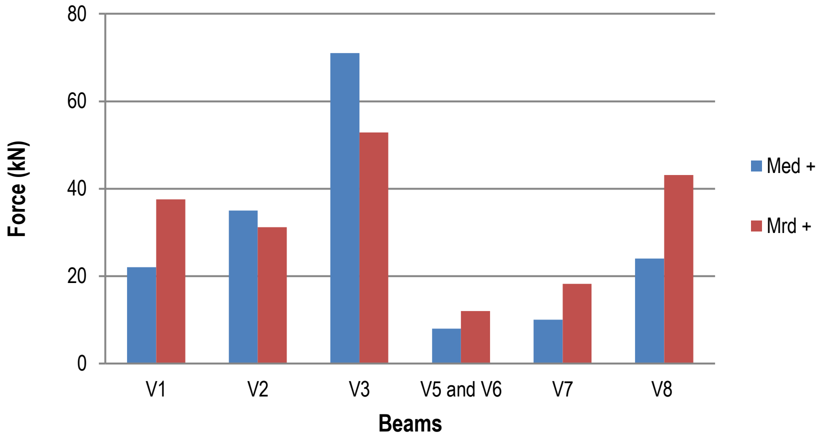
However, for the shear force (Graph 11), the values, which are obtained with the model defined, are significantly higher than the shear resistance. Moreover, according to the literature, the transverse reinforcement in much of the old buildings was quite deficient, and often did not even exist. Thus, it can conclude that the resistance of the columns to the shear force will be a serious problem for the possible collapse of the structure on the back of the building.

Beams

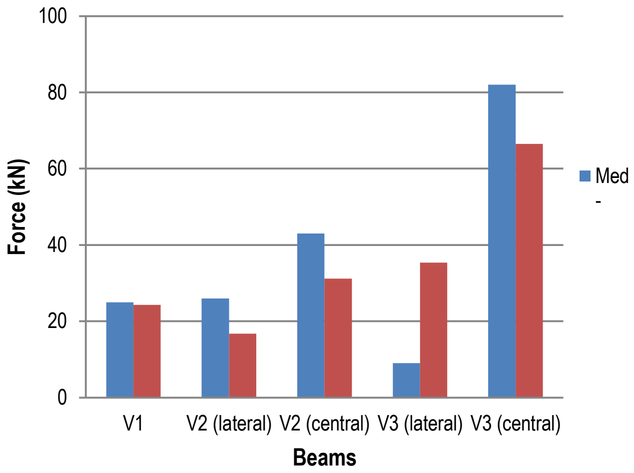
Table 13 depicts the dimensions and the reinforcements of each beam. In the following graphs (Graph 12 to Graph 14), it can see the difference between the values of the internal forces observed from the building model with correspondent resistant values based on the reinforcement of each beam.

Table 13 - Dimensions and reinforcements of each beam (Descriptive Memory, 1939).

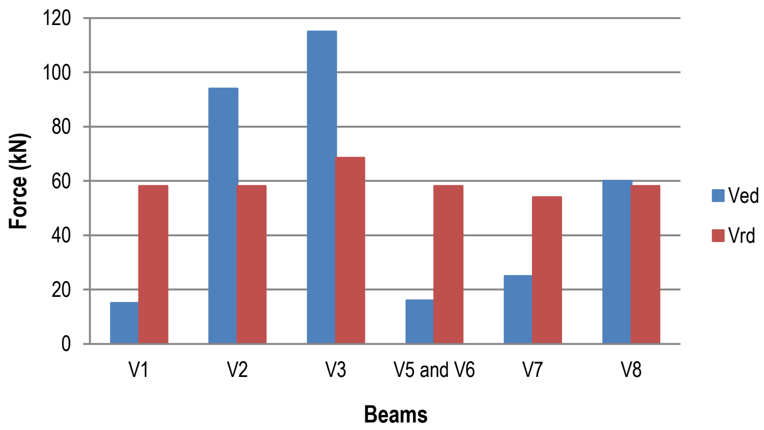
Beam	b [m]	h [m]	Place	Longitudinal Reinforcement	A_s [cm²]	Transverse Reinforcement	A_{SW}/s [cm²/m]
V1	0.23	0.3	M-	3Φ5/8"	5.97	2Φ5/16" spacing by 15 cm	6.60
	0.23	0.3	M+	5Φ5/8"	9.95		
V2	0.23	0.3	M- (lateral)	2Φ5/8"	398		
	0.23	0.3	M+	4Φ5/8"	7.96		
	0.23	0.3	M- (center)	4Φ5/8"	7.96		
V3	0.23	0.35	M- (lateral)	2Φ3/8"+ 2Φ3/4"	7.16		
	0.23	0.35	M+	4Φ3/4"	11.48		
	0.23	0.35	M- (center)	4Φ3/4"+ 2Φ5/8'	15.46		
V5 and V6	0.13	0.3	M+	3Φ7/16"	2.91		
V7	0.23	0.28	M+	5Φ7/16"	4.85		
V8	0.66	0.3	M+	8Φ1/2"	10.16		



Graph 12 – Positive bending moments



Graph 13 - Negative bending moments



Graph 14 - Shear force

In most of the beams analysed, the resistance capacity is not exceeded. The exception is mainly to V2 and V3 beams, because corresponding to the beams with a larger area of influence to the vertical loads and they are in the periphery of the building, so they are going to suffer greater efforts of the seismic action.

5. CONCLUSION

The three-dimensional dynamic response spectrum modal analysis of the quarter presented in this paper allows drawing some conclusions, not only on the dynamic characteristics of the building (and also of the aggregate buildings), but also in relation to the seismic performance of the existent structure studied. In particular, it can enumerate the following overall conclusions:

- First, it was possible to know more about the structure of the 'placa' buildings and their materials, because there are no other identical to this one, in relation to this type of buildings.
- This study shows that this last type of masonry buildings (‘placa’ buildings) was becoming stiffer, with higher frequencies, mainly due to the introduction of reinforced concrete elements on the buildings.
- Based on the mode shapes of vibration obtained, the behaviour of the quarter as a single structural unit does not exist in this structure, mainly due to the lack of rigidity of the floor to the distortion in the horizontal plane. However, the axial rigidity of the floor significantly contributes to the configuration of the main vibration modes in which the aggregate buildings move together in one direction. Another important aspect to emphasize in the context of linear analysis performed is the absence of significant displacements of the gable walls between the buildings in their own plans. The sharing of gable walls in different buildings greatly reduces the effects of global torsion at each building, because they cannot rotate separately from each other.
- The most rigid direction of the building corresponding to the transverse direction due to the gable walls, while in the other direction only the wall of the façade behaves as a resistance element. Besides that, the salient body on the back (corresponding to the reinforced concrete structure) works almost likes an isolated body, that has only one constraint which connects the central body, and it has more mass on the floors due to the concrete slab. So this body on the back is more flexible and therefore presents greater displacement.
- In façades, for the seismic load combination, the resistance values of shear stresses are exceeded especially in the vertical elements of the first floor. The results obtained showed that the first damage (cracking) by cutting in the façade elements can occur at less levels of seismic regulation. However the collapse, or not, of the structure depends on the global resistance of the other structural elements.
- For the vertical stress, tensile values in the base of the façade at the ground floor was obtained, signifying opening of tensile cracks in the joints accompanied by a reduction in the overall stiffness of the columns. However, the maximum values were not exceeded.
- On the other hand, the compression stresses in the columns of the façades are still far from the resistance values adopted.

- In general the gable walls and the exterior walls do not suffer high stresses. Only some higher values than the allowable stresses can be seen only in localized areas.
- The interior walls with openings (windows and doors) have higher stresses than the walls from the stairs. For the shear stresses, the maximum values are exceeded mainly in the connection between the interior walls (parallel to the façade) to the gable walls. This happens essentially due to the weak connection between these walls because there are a lot of openings in these zones and the area of masonry that connects the both walls is minimal.
- The analysis of the connection between the timber floor and the façade shows that the levels of force are considerable and it is concluded that the façade wall can be weakened in relation to the out-of-plan collapse.
- Finally, in relation to the reinforced concrete structure, on the back of the building, the main problem corresponds to the shear resistance of the columns.

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ANNEX 1 - Descriptive Memory

DES. N.º 2

Alterações ao Projecto inicial – eliminação do sagüão junto a escada principal – alargamento da sala de mesa – alargamento do atrio da escada – substituição das paredes duplas entre inquilinos por paredes de uma vez de tijolo – alteração do desenvolvimento da escada principal – alteração da forma esterior da escada de serviço – alargamento do vão interior da cosinha e eliminação do vão lateral da mesma dependencia.

O ARQUITECTO

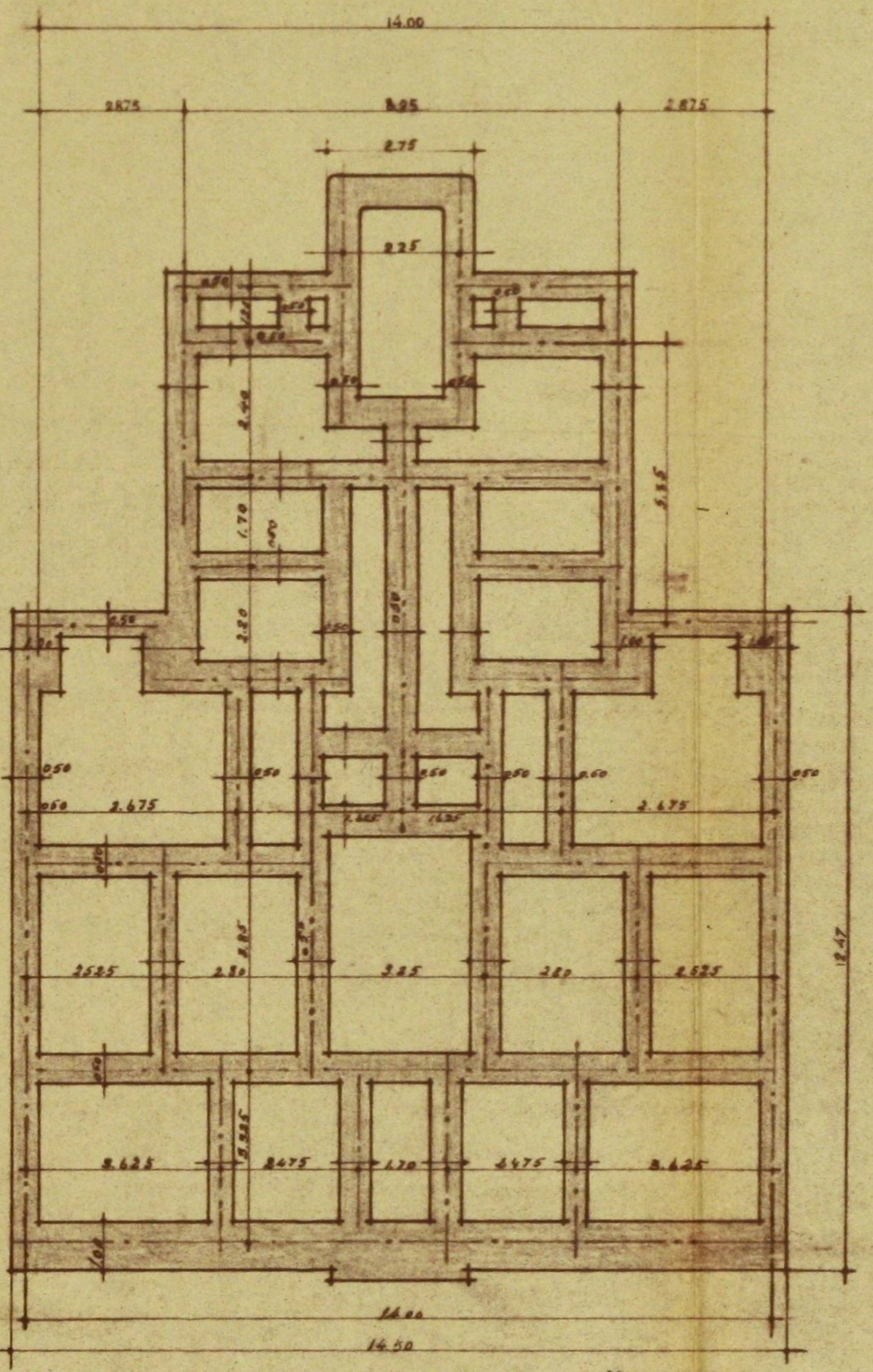
Foi paga a licença N.º 81
em 16 JUN 1939

BLOCO A CONSTRUIR NO TERRENO MUNICIPAL SITUADO ENTRE AS R.s LUCINDA DO CARMO CARLOS MARDEL, ACTOR IZIDORO E AUGUSTO MACHADO - LISBOA

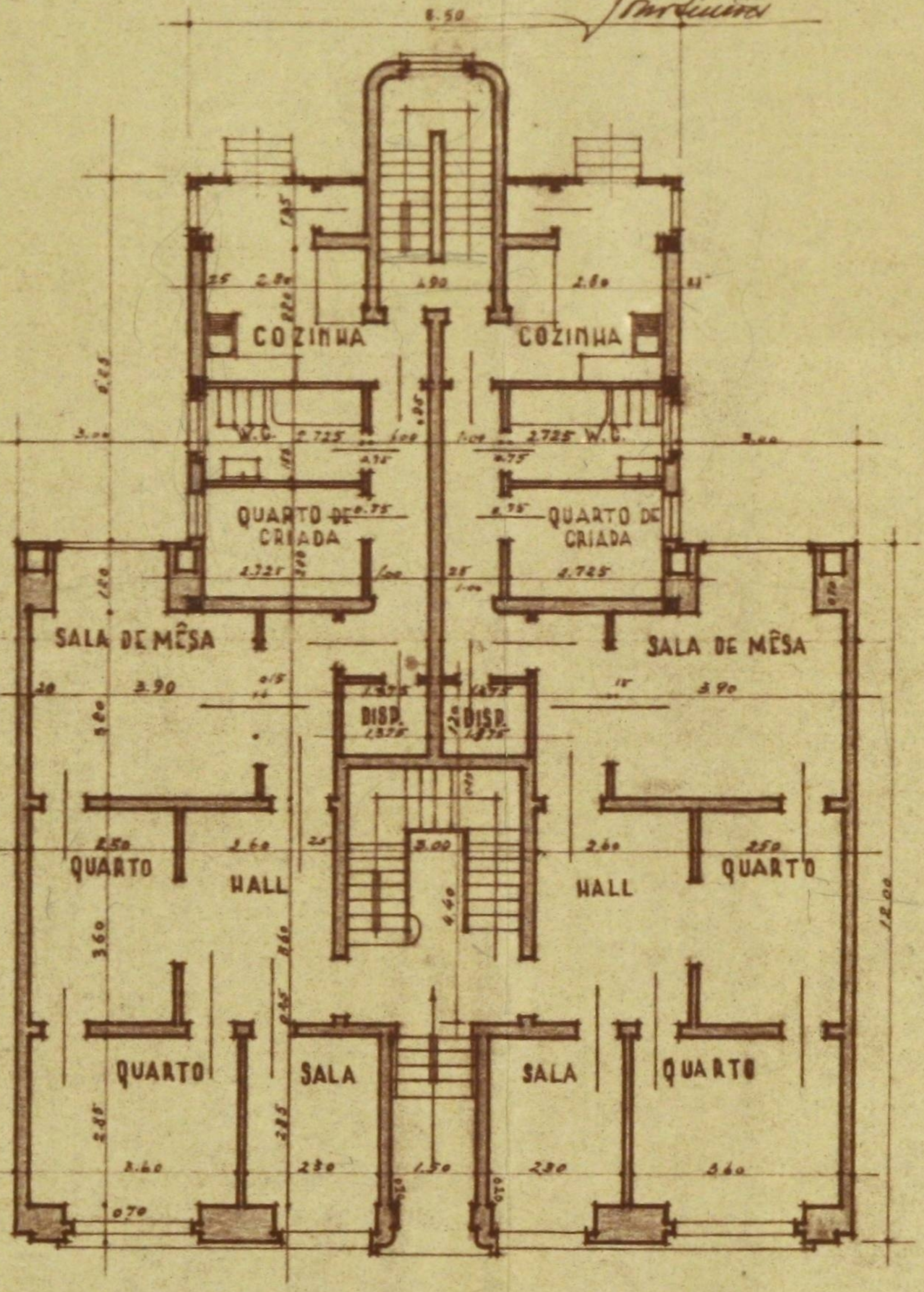
PREDIO TIPO B

TALHAO N.º 12

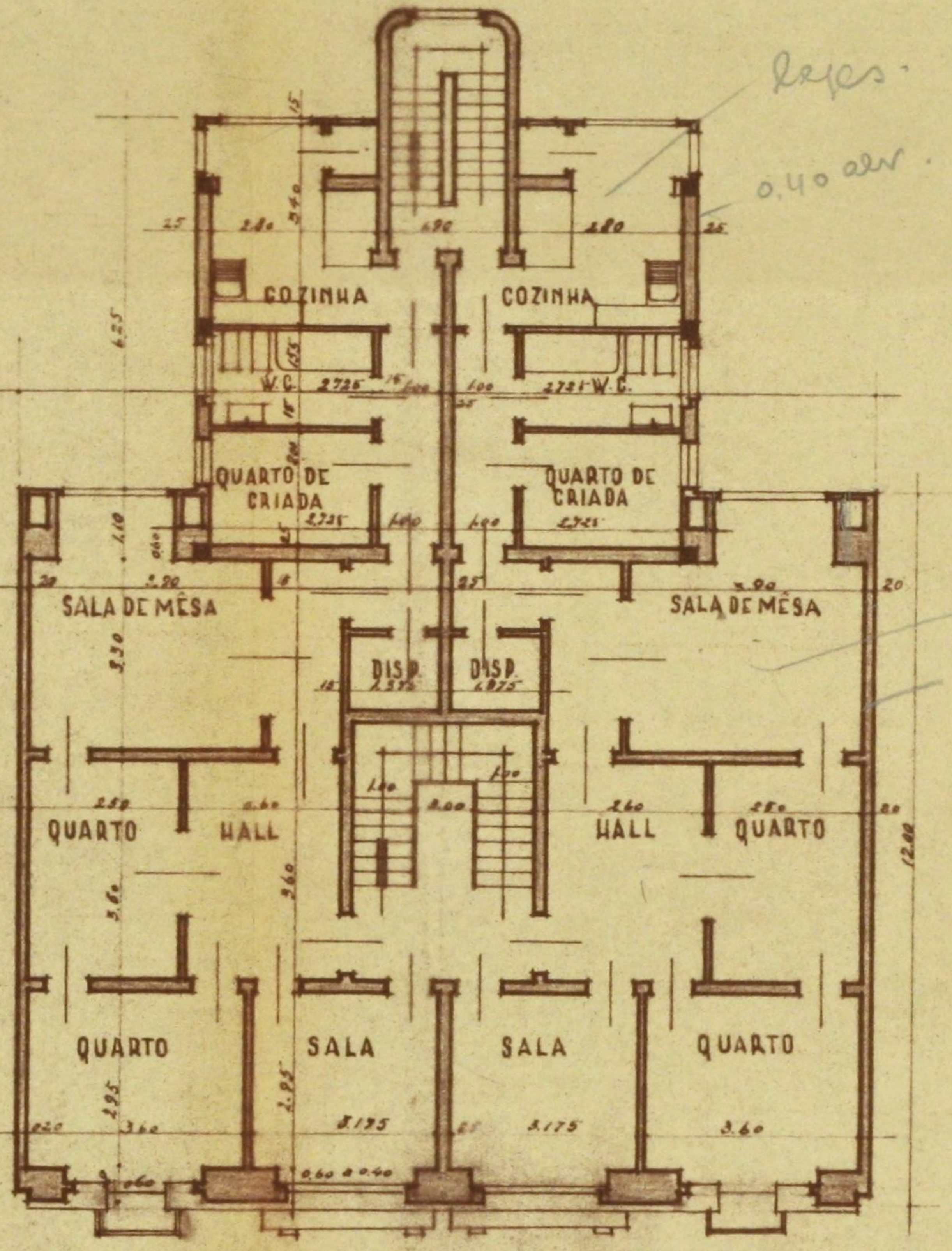
PLANTAS
ESCALA DE 1:100



PLANTA DAS FUNDAÇÕES



PLANTA DO REZ-DO-CHÃO



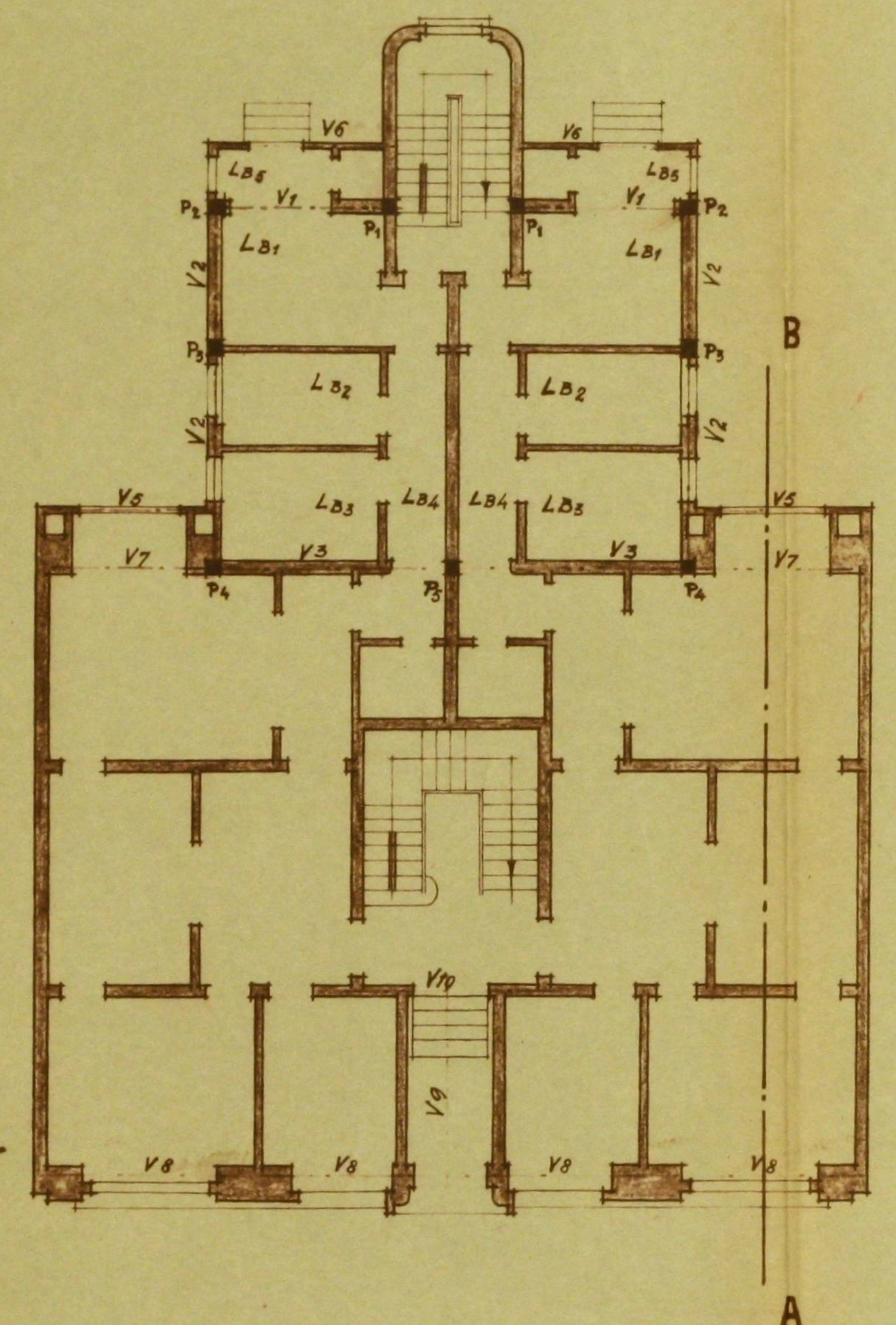
PLANTA DOS ANDARES

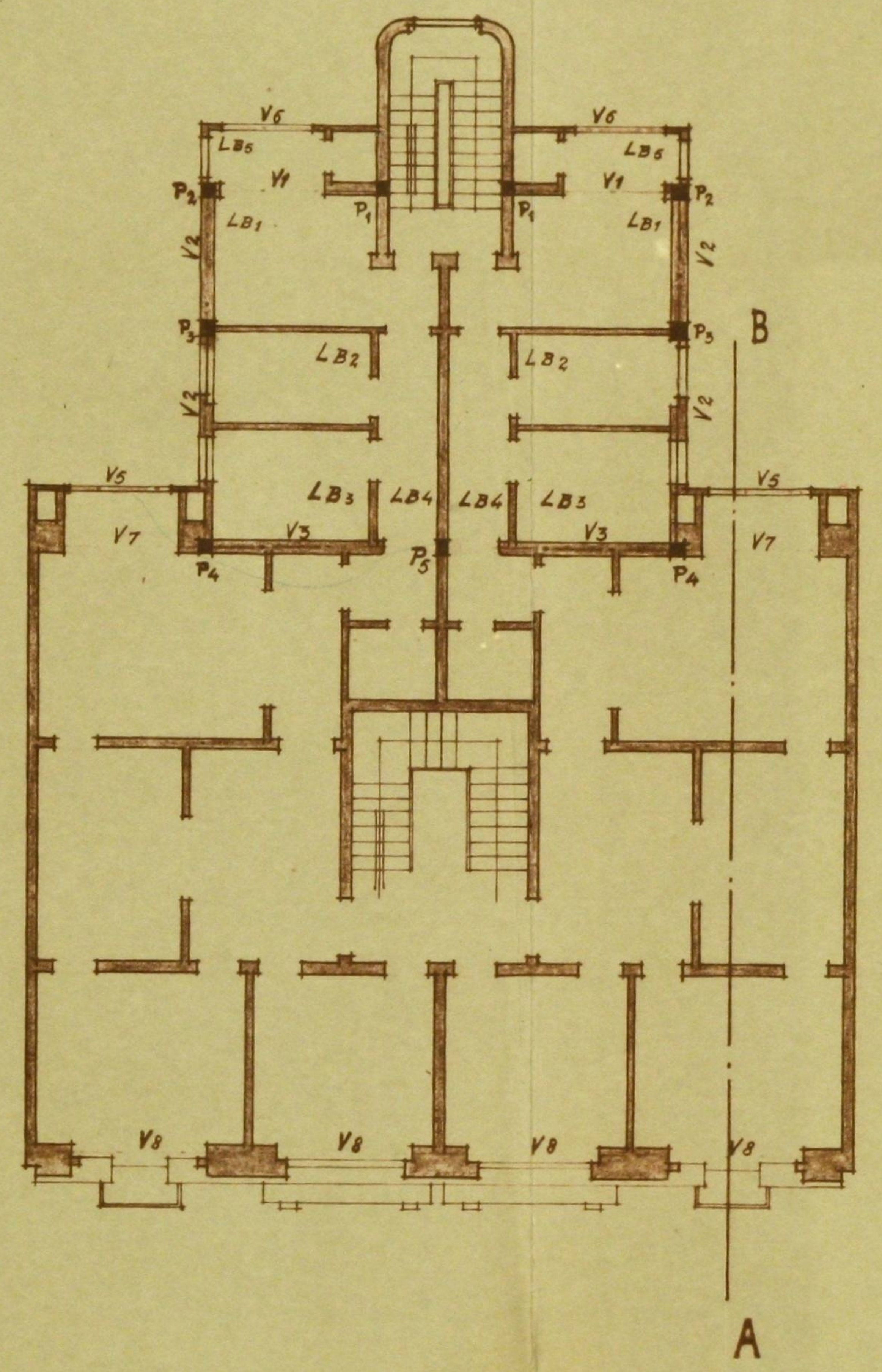
O ARQUITECTO

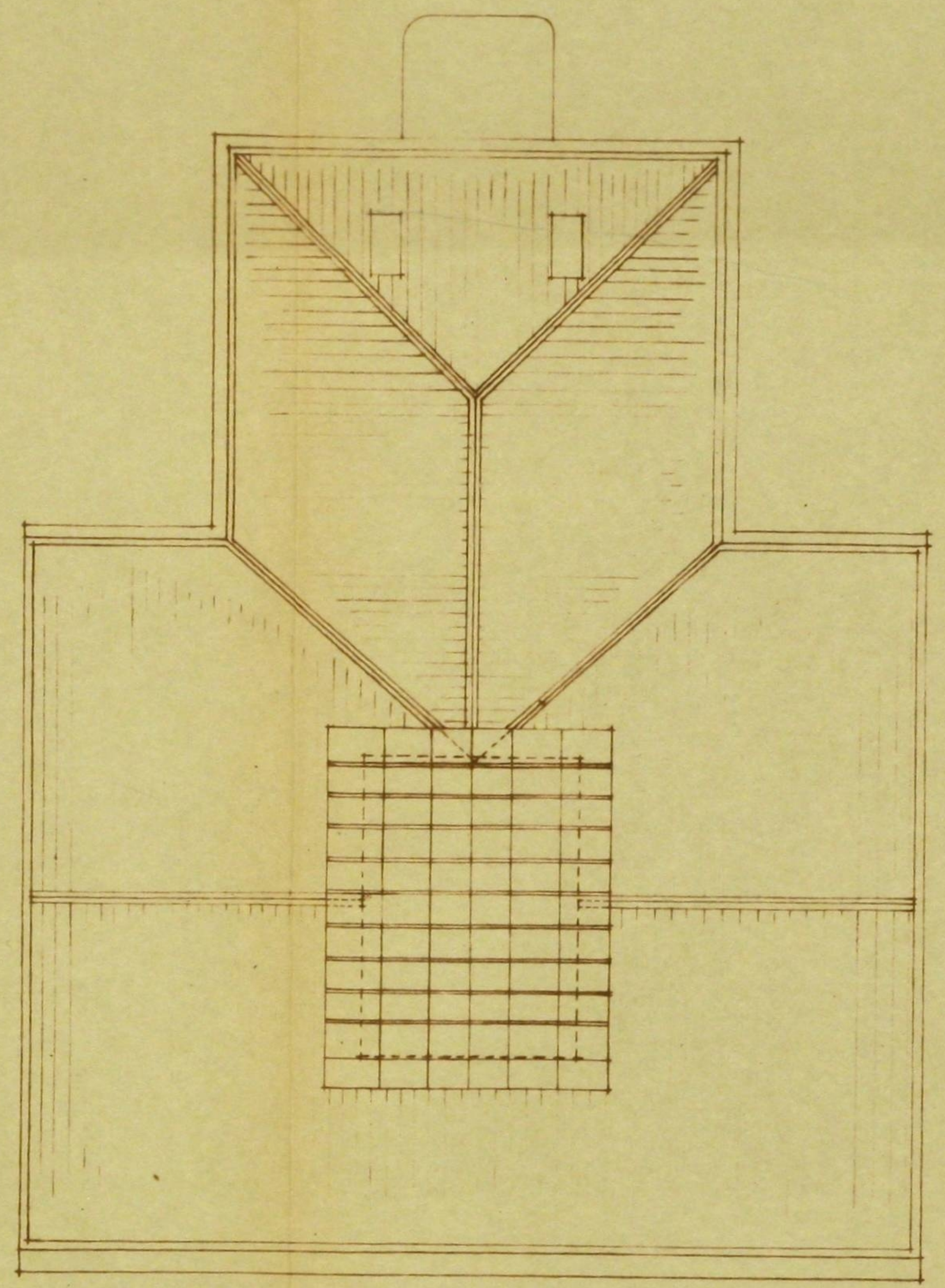
DES. n.º 3

Folha 6 / 11

Foi paga a licença N.º 81
em 16 JUN 1939







PLANTA DAS COBERTURAS

BLOCO A CONSTRUIR NO TERRENO MUNICIPAL SITUADO ENTRE AS R.s LUCINDA DO CARMO CARLOS MARDEL, ACTOR IZIDORO E AUGUSTO MACHADO - LISBOA

PREDIO TIPO B

TALHÃO N.º 12

PLANTAS
ESCALA DE 1:100

O ARQUITECTO

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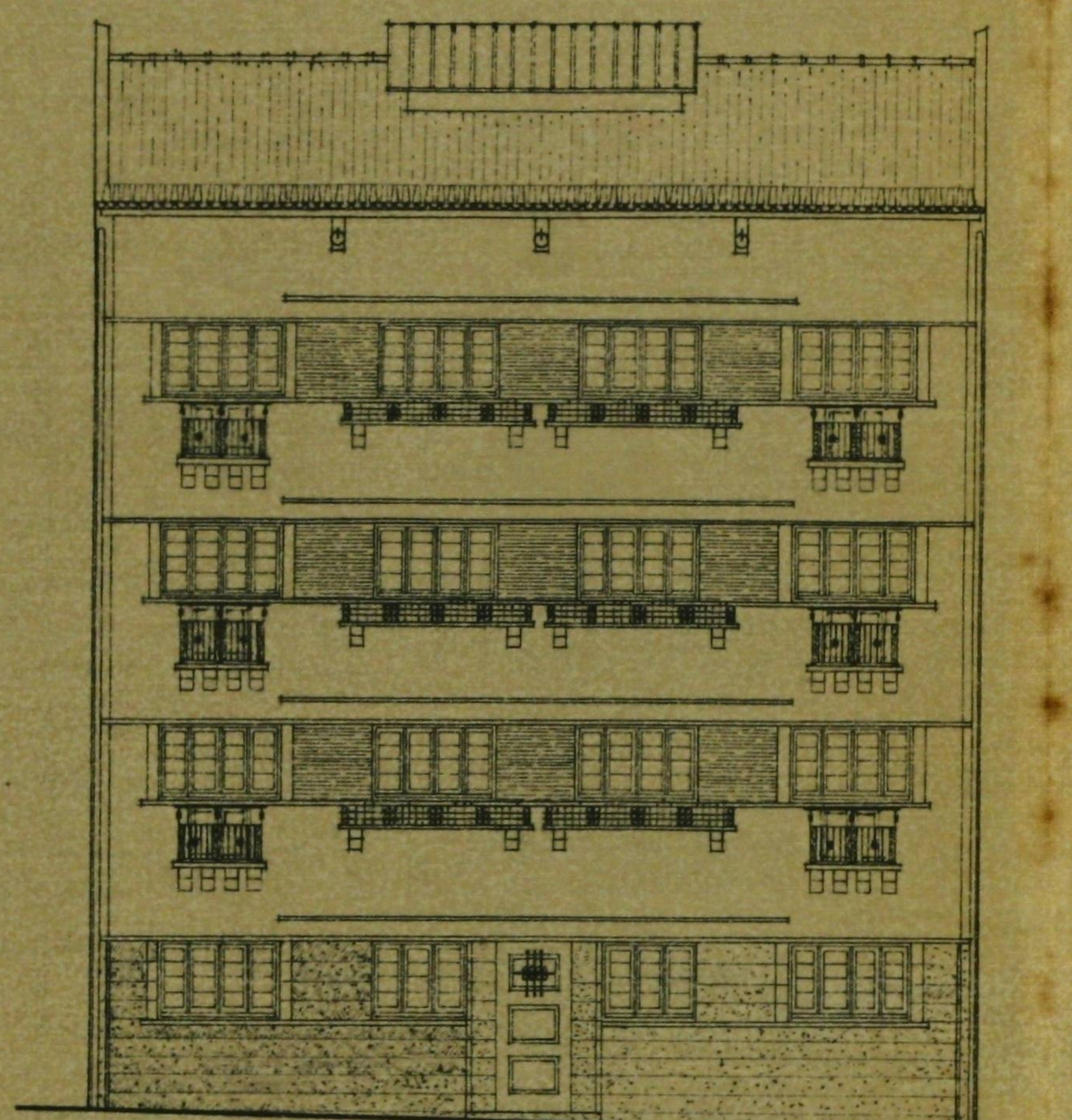
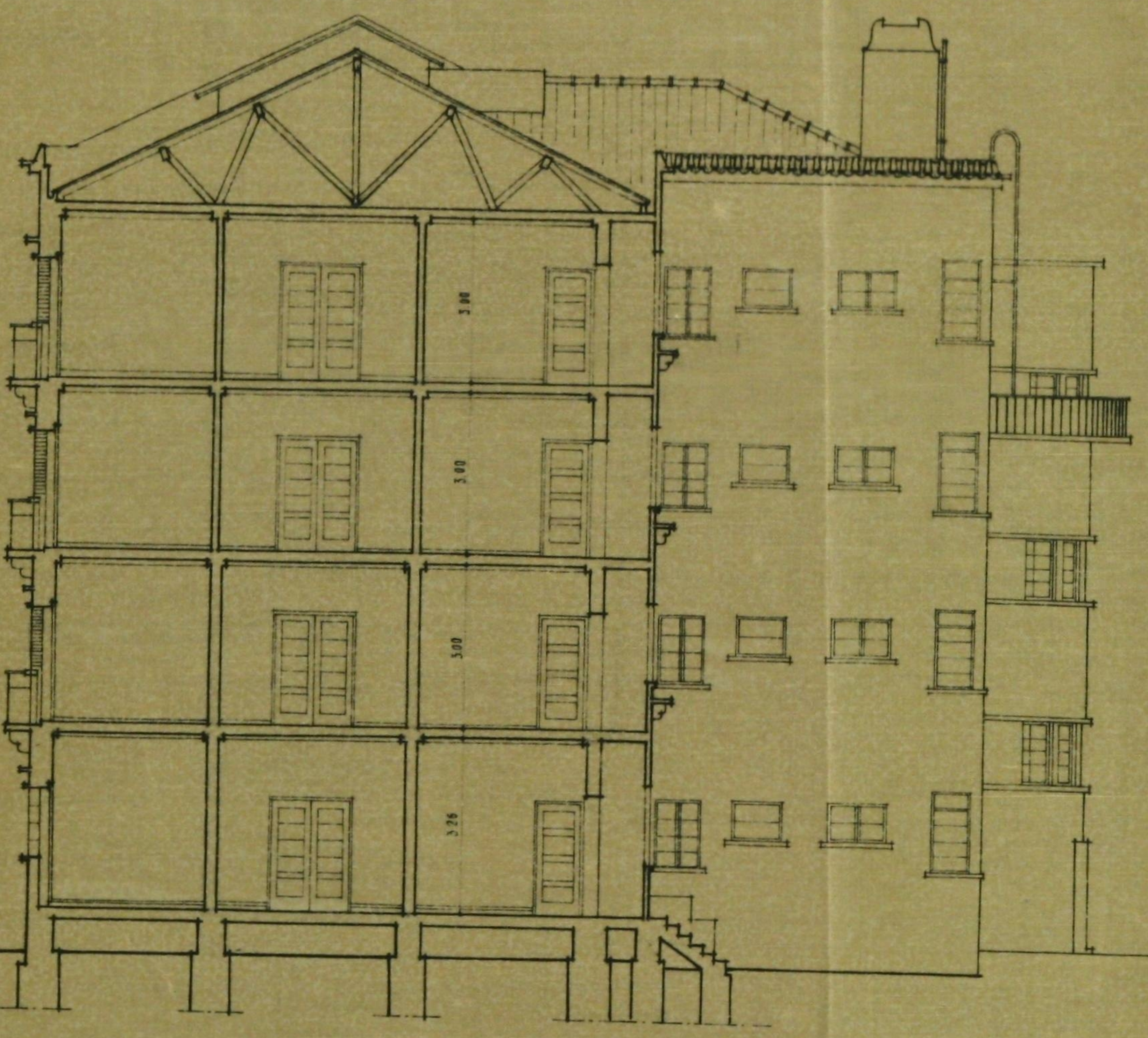
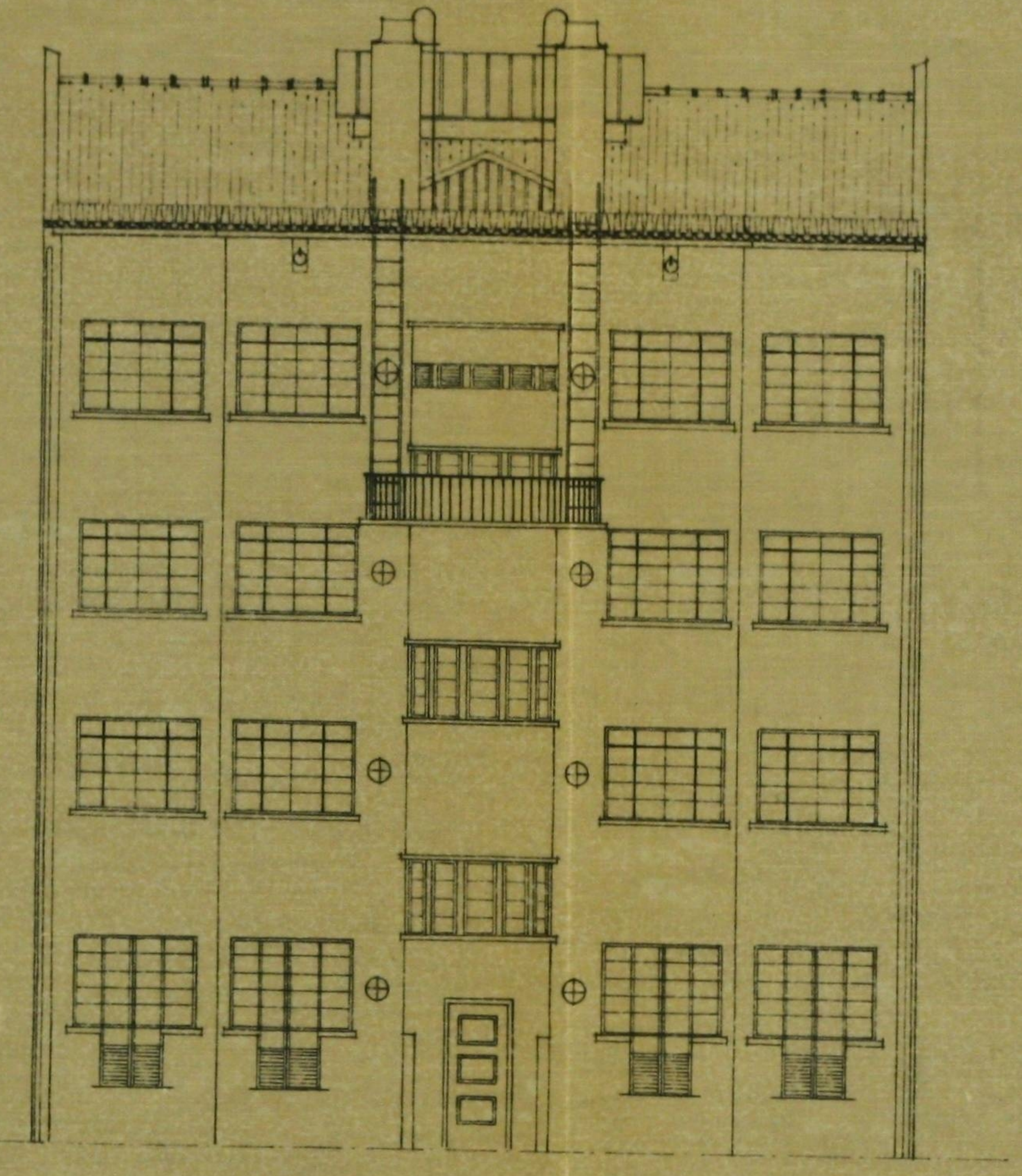


ALÇADO PRINCIPAL



CORTE POR A-B



ALÇADO POSTERIOR

DES. Nº1

Folha 6

8

BLOCO A CONSTRUIR NO TERRENO MUNICIPAL SITUADO ENTRE AS Rs LUCINDA DO CARMO, CARLOS MARDEL, ACTOR IZIDORO E AUGUSTO MACHADO-LISBOA.

PREDIO
TIPO
B
TALHÃO Nº 12

ALÇADOS E CORTE
ESCALA DE 1:100

O ARQUITECTO

Abril de 1937

ARQUITECTO - JOÃO SIMÕES
ATELIER: R. DR. ALEXANDRE BRAGA, 47 - TELEFONE 4 3367
L I S B O A

O Bloco é atravessado por uma passagem de serviço em toda a sua maior extensão, com duas entradas sob os prédios tipos D. e G. (ou lotes 8 e 16) que serve de comunicação do exterior para as escadas de serviço de todos os prédios.

Os prédios atravez dos quais se faz a passagem de serviço, teem, no rez-do-chão, lojas para estabelecimentos com montras de exposição sôbre a referida passagem, procurando fazer-se assim deste modo uma compensação vantajosa para esses mesmos prédios de onde se tirou a superficie necessária para a passagem de serviço.

Com a passagem de serviço colocada no interior do Bloco beneficiam todos os prédios que ficam providos desse serviço muito util e prático, sem prejuizo nem sacrificio da superficie necessária à construção.

Todos os prédios são compostos de rez-do-chão e três andares, sendo o ultimo andar recolhido nos prédios tipos D. e G. devido às dimensões das ruas onde são construidos. Na parte anterior dos andares recolhidos serão instalados terraços para logradouro dos respectivos inquilinos, sendo os referidos terraços providos de pergolas. Nestes prédios o rez-do-chão destina-se a estabelecimentos ao passo que nos restantes se destina a habitações.

Na rectaguarda de todos os prédios serão colocados pequenos quintais para utilização dos inquilinos do rez-do-chão.

Foi paga a licença N.º 104
em 29 AGO 1939

——— : ———

Os materiais a aplicar e a execução das varias partes destas construções deverão, de um modo geral, obedecer às seguintes indicações.

Todas as soleiras, degraus, hombreiras e vergas das portas principais, incluindo o sóco das fachadas também principais, serão de cantaria de lióz. Os peitoris dos vãos de janela, as lajes das sacadas e as ombreiras dos vãos correspondentes serão também de lióz. A cantaria dos peitoris será brunida e a restante trabalhada à bujarda.

Os vestibulos ou átrios das escadas serão revestidos de marmore, em média até à altura de 1,50 m. Os degraus dos mesmos átrios terão espelhos e pisos de marmore.

As paredes exteriores serão rebocadas e chapadas com argamassa de cimento e areia ao traço de 1:3.

As paredes interiores serão rebocadas com argamassa de cal e areia ou argamassa de cimento e areia.

Os estuques serão a massa de areia (aspero), com côr na massa, ou serão pintados a tinta de oleo, mate.

Todas as paredes exteriores serão guarnecidas de roboco tipo "simile" qque substituirá qualquer outro acabamento. O tom a aplicar será indicado pela fiscalização municipal. Os nembos centrais dos vãos dos andares serão guarnecidos de reboco de calcareo (projectado) na tonalidade conveniente. O sóco dos edificios poderá ser do mesmo material em substituição da cantaria de lióz se a Camara julgar conveniente ou aceitável essa substituição. Convindo esclarecer que essa substituição a verificar-se será geral.

Os pavimentos de todos os pisos à excepção das cosinhas e dependências sanitárias poderão ser de sôlho de madeira de pinho à inglesa. Os pavimentos daquelas dependências serão de mosaico hidraulico, cerâmico ou outro material que se julgar conveniente desde que a fiscalização municipal assim o entenda.

As cosinhas e dependências sanitárias levarão revestimento de azulejo até à altura das portas sendo a parede restante pintada a tinta de esmalte. Os lambris de azulejo deverão ser lisos sem quaisquer decorações. A parte restante das paredes destas dependências será pintada a tinta de esmalte.

Todas as engras quer da intercepção das paredes entre si quer destas com os tectos serão curvas. Os rodapés terão concavas e convexas, e não existem propriamente sancas mas apenas um pequeno balanço na continuação do tecto sôbre a perede para defenir o acabamento de cada uma das partes.

Os aros de caixilhos, guarnições de portas e roda-pés serão de madeira bem seca. Todos os caixilhos de vão de peito e de sacada serão de madeira de casquinha. As portas interiores e exteriores serão de madeira de casquinha almofadadas e para pintar.

Nos detalhes de carpintaria seguir-se-ão os indicados no projecto.

Todos os vãos de janela e de sacada à excepção das frestas das casas de banho, cozinha e pia s de despejo, levarão gelozias que enrolarão em caixas proprias de betão armado.

A escada de acesso aos andares será de madeira de pinho e levará corrimão de tubo de ferro para receber pintura.

As sacadas levarão grades de ferro e as cozinhas marquizes de ferro, envidraçadas.

As portas interiores, caixilhos, etc., levarão as ferragens necessárias que deverão ter bastante solidez e simplicidade de aspecto.

Para iluminação e arejamento das escadas serão construidos lanternins envidraçados e de ferro laminado.

Para acesso ao telhado de cobertura os prédios serão providos de escadas de ferro (portaló).

A canalização de esgôtos será dirigida dos prédios para as respectivas ruas confinantes, em ligação ao colector, ou será dirigida para um colector central a colocar ao longo da passagem de serviço que foi projectada no bloco a construir, se a Câmara assim o determinar.

Haverá rêde de canalização destinada a receber esgôtos das retretes águas sujas e pluviais. Os esgôtos das retretes serão de grés cerâmico vidrado com manilhas de 0,12 e de 0,17, os ramais destes esgôtos serão ligados a câmaras de inspecção evitando-se tanto possivel curvas para uma facil inspecção. Os esgôtos das águas pluviais serão de ferro galvanizado com a secção de 0,08 e serão embebidos nas paredes. Os tubos de ventilação será o de ferro galvanisado com a secção de 0,05. Será colocada a aparelhagem sanitária e higiénica que compreende, uma bacia de retrete com tampo pintado a esmalte, um bidet, um lavatório com o minimo de 0,60, um autoclismo de ferro fundido e uma banheira de ferro esmaltado assente e revestida de azulejo com uma caixa de ar envolvendo a referida banheira, com a necessária respiração ou ventilação.

Os tubos de distribuição para fornecimento de águas serão de chumbo e terão a secção suficiente para assegurar o abastecimento facil. Os varios pontos a alimentar são: nas casas de banho, 2 torneiras de serviço de 1/2" e três torneiras de passagem de 1/2" sendo uma para o autoclismo, uma para bidet, e outra para esquentador; nas cozinhas 3 torneiras de serviço. Estas torneiras serão de metal e as anteriores serão niqueladas.

Os prédios serão dotados de instalação ielectrica que deverá obedecer ao regulamento da repartição das Industrias Electricas, e cada habitação terá o seu quadro privativo e dois circuitos. A tubagem será interior e de tubo Bergman.

Serão previstas para cada dependência um ponto de luz. As portas da escada terão uma betoneira de campainhas para cada habitação e serão comandadas de cada habitação por meio de trinco electrico. Todas as portas de habitação serão providas de campainhas proprias.

Todas as carpintarias e serralharias serão pintadas a tinta de óleo depois de todos os trabalhos necessários e preparatórios. As paredes e tectos das casas de banho e cozinhas serão pintadas a tinta de óleo com uma demão de esmalte.

Em toda a caixilharia será aplicado vidro nacional de 3 m/m de espessura.

Em cada cozinha haverá um lava-louça de lioz branco e as pias de despejo serão também de lioz. Haverá também um armário e as despensas serão providas de parteleiras.

As coberturas serão de telha de tipo Lusa, com beiral à portuguesa sendo feita uma estrutura de madeira de pinho.

Todas as coberturas serão providas de algerozes de zinco nº12, recolhidos. As tubagens de esgôtos serão embebidas nas paredes.

ARQUITECTO – JOÃO SIMÕES
ATELIER: R. DR. ALEXANDRE BRAGA, 47 – TELEFONE 4 3367
L I S B O A

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Na construção será aplicada alvenaria de pedra, alvenaria de tijolo e betão armado.

A parede mestra principal será de alvenaria de pedra com argamassa de cimento e areia. Uma parte da parede mestra posterior será também de alvenaria da mesma qualidade. A parte restante destas paredes e que assenta em estrutura de betão armado será de tijolo macisso a 1 vez.

As paredes das escadas, saguões e tôdas as que suportam cargas serão de tijolo macisso a 1 vez.

As divisórias serão de tijolo furado a 1/2 vez, excepto porém as divisórias entre inquilinos que serão constituidas por parede dupla (duas paredes de tijolo furado a 1/2 vez com caixa de ar de o,o5) devidamente consolidadas e travadas entre si.

Pode na construção ser usado indistintamente o pavimento de madeira ou de betão armado, sendo unicamente obrigatório de laje de betão armado os pavimentos das cozinhas e casas de banho, assim como a estrutura das escadas que serão também de betão armado.

Parte dos elementos de suporte dos pavimentos foram calculados supondo estrutura de madeira. Se o proprietário optar por pavimento geral de betão armado, o Municipio obriga-se, caso o proprietário o deseje, a fornecer os detalhes dêste pavimento e bem assim proceder às rectificações que fôr necessário introduzir nos elementos acima referidos.

A espessura das empenas figura no projecto com o,2o, mas só deverá ser adoptada na construção no caso dos pavimentos serem de betão armado, devendo nêsse caso as ditas empenas serm de betão armado. Se o proprietário optar por pavimento de madeira deverá a espessura das empenas aumentar, de acordo com as disposições do R.G.C.U e com as indicações fornecidas pela fiscalização Municipal.

Para as fundações previram-se paredes continuas de alvenaria hidraulica para apoio das paredes, e sapatas de betão para apoio dos pilares.

O terreno foi considerado em condições favoraveis, e se porém a sua natureza indicar outro sistema de fundações, deverá ser apresentada à aprovação Municipal o respectivo estudo, pelo proprietário.

Previu-se que parte do desaterro a fazer com as fundações poderá ser aproveitado para aterro e nivelamento dos logradouros de acordo com o projecto realizado.

Em tôda a parte omissa será respeitado o R.G.C.U., em vigôr.

Acompanha a presente memória a justificação dos calculos de estabilidade elaborados por engenheiro civil diplomado.

Lisboa, 26 de Abril de 1939

Dispensa fiscalização Paguese.

O ARQUITECTO,

C. M. L. - Direcção dos Serviços Centrais
1.ª REPARTIÇÃO (Central)
1.ª SECÇÃO - Expediente
30 JUN. 1939
Registado sob o N.º 28208
DESTINO: Direcção dos Serviços de Obras

IMPOSTO DO SELO 2$50

Fôlha 1
Proc.º 28208

ARQUIVE-SE
16 FEV. 1940
O Director dos Serviços Centrais

Exmo Snr Presidenete da Camara Municipal de Lisboa

Os abaixo assinados proprietarios dos terrenos do Bloco Municipal situado na Rua Carlos Mardel, Augusto Machado, Actor Isidoro e Lucinda do Carmo, veem muito respeitosamente pedir a Va Exa se digne conceder-nos algumass alterações á memoria discritiva as quais não afectam as construções dos predios.

1º ESCADA PRINCIPAL: Em virtude de se construir os pisos em vigamento excepto cosibhas e casas de banho, de harmonia com a memoria discritiva ser substituida a construção de cimento armado por estrutura de madeira.

2º ESCADA DE SERVIÇO: Substituir a construção de cimento armado por escada de ferro, como tem sido autorizddo pela Exma Camara em outras construções e de harmonia com o artigo 178º do R.G.C.U.

3º EMPENAS : No projecto indicanos a espessura de o,2o para as empenas o que para os pisos de vigamento não é premitido. O regulamento autoriza a substituição por alvinaria de pedra com a espessura de o,4o ou em blocos com a espessura de o,3o nos très andares inferiores e o,2o no andar superior, o que nos vai esteitar as larguras das casas. Pedimos para que nos seja autorisado as construções destas em alvinaria de tijolo massiçõ a uma vez com argamassa de cimento ao traço de 1.3 o que nos dá uma melhor ligação e travamento com as divisorias e ocupar menos espaço e ainda por ser mais resistente que a enpena a blocos de cimento que era a que nos roubava menos espaço e, tambem por os predios serem construidos em conjunto formando um bloco unico.

Deferido nos termos da informação
2-VIII-39
Carvalho

4.ª REPARTIÇÃO
Enviado ao "D.º Municipal" em 3 / 8 /1939
Publicado no B.º n.º 1272 de / /19

A II e IV Repartição
para informar

e volte em ___/___/___
___/___/___
O Eng.º-Director dos S. U. O.

4º ESTUQUE A ASPERO NAS PAREDES : Substituição por estuque liso o qual pode ser lavavel e não formar deposito de poeiras o que é anti-higienico.

5º CASAS DE BANHO E COSINHAS: Para que o azulejo fique na altura de 1,5o conforme indica o regulamento G.C.U no seu artigo Nº 134º

6ºBANHEIRAS: Não serem colocadas as banheiras em virtude de a maioria dos inquilinos que habitam casas desta natureza as possuirem e tambem por ser anti-hegienico, por o inquilino que se mudar e o que entrar não estar de acôrdo em se banhar, por não saber se o outro inquilino tinha qualquer doença contagiosa.

7º ESTORES OU GELOSIAS: Substituição por janelas, por estas serem de melhor vedação e resitencia do que a gelosias.

Pedindo a benovolencia de Vª Exa , e o bom acolhimento desta petição.

Pedimos deferimento

Lisboa 27 de Junho de 1939

Os proprietarios dos terrenos.

a)

Obra 52866 João Guimarães V
" 52867 Rodrigo Guimarães V
Adolfo Gonçalves
Pela Sociedade de Trabalhos Industriais, L.da
Paulo Fernandes
52863 / 52864 Manuel Domingues Roza
52865 José Maria Fernandes V

CÁLCULOS DE ESTABILIDADE

Prédio Tipo B

Cimento armado:

O estudo referente a esta construção consta de lajes de pavimento e vigas em cimento armado.

Nos calculos dos diferentes elementos adoptaram-se os seguintes principios.

Sobrecargas: as previstas pelo Regulamento de Betão Armado em vigôr ou sejam:

Lajes .... 200 kg/mq.
Escadas .... 250 kg/mq.

Os momentos flectores e esforços transversos foram calculados nos casos previstos pelo R.B.A. segundo os valores nêles consignados ou ainda segundo o metodo das linhas de influência exposto no curso de B.A. de G.Magnel.

Os materiais empregados obedecerão aos preceitos citados nos Artºs 5º e 9º do R.B.A.. O cimento será Portland Normal de fabrico nacional. O Betão a empregar será plastico, conforme as indicações do R.B.A. e a sua dosagem será nos casos correntes, 300 kgs. de cimento, 400 l. de areia e 800 l. de brita.

As fundações sob paredes terão a largura necessária a não exceder 2 kg/cmq. e sob os pilares adoptar-se-á o sistema indicado nos desenhos anexos.

A alvenaria sôbre vigas exteriores do corpo posterior deste edificio terá como função a de simples enchimento, e repousarão sôbre as vigas aos niveis dos diferentes pisos.

Estas paredes serão em tejolo burro ou também poderão ser executadas em alvenaria de tijolo mixto (furado e burro) observando-se sempre o principio que as fiadas cujo tôpo está dirigido para o exterior do edificio, serão sempre em tijolo burro, afim de evitar toda e qualquer infiltração. Este ultimo processo mantendo o isolamento caracteristico deste material alivia a construção.

Lisboa, 26 de Abril de 1939

O Engenheiro

[signature]

Insc. n.º 77

BLOCO A CONSTRUIR NO TERRENO MUNICIPAL ENTRE AS RUAS LUCINDA DO CARMO, CARLOS MARDEL, ACTOR IZIDORO E AUGUSTO MACHADO

----oOo----

TIPO B TALHÃO Nº

- Cálculos justificativos -

LAJES

LAJE: B 1

1) - Vãos:

Útil ...... 2,20 x 2,80
Teórico ... 2,40 x 3,00

2) - Cargas:

Sobrecargas	200 kgs/mq.
Pêso próprio	240 " "
Revestimento	60 " "
p =	500 Kgs/mq.

Redução pela dupla flexão $a = \frac{3,00}{2,35} = 1,27$

$p1 = 0,710 \times 500 = 355$ kgs/mc.
$p2 = 0,290 \times 500 = 145$ " "

3) - Momentos flectores

a)-No sentido do vão menor
a meio e nos apoios

$$M_{ap} = M_m = 1/10 \times 355 \times \overline{2,4}^2 = 205 \text{ kgm.}$$

b)-No sentido do vão maior
a meio e nos apoios

$$M_{ap} = M_m = 1/10 \times 145 \times \overline{3}^2 = 130 \text{ kgm.}$$

4) - Dimensões e armaduras

a)-No sentido do vão menor

e = 10 cm. h = 8,5

A meio e nos apoios:

$$B = \frac{8,5}{\sqrt{205}} = 0,555 \qquad R'b = 28 \text{ kgs/cmq.}$$

$$Aa = 0,00305 \times 8,5 \times 100 = 2,6 \text{ cmq.}$$

Adoptam-se: 9 Ø 1/4" p.m. em tracção

b)-No sentido do vão maior:

$$e = 10 \text{ cm.} \qquad h = 7,5 \text{ cm.}$$

A meio e nos apoios:

$$B = \frac{7,5}{\sqrt{130}} = 0,6 \qquad R'b = 26 \text{ kgs/cmq.}$$

$$Aa = 0,00266 \times 7,5 \times 100 = 2 \text{ cmq.}$$

Adoptam-se: 9 Ø 1/4" p.m. em tracção

5) - Reacções nos apoios

a)- No sentido do vão menor:

$$r_1' = 1/2\ (3+0,6)1,2 \times \frac{200}{3} = 147 \text{ kgs/m.c.}$$

$$r_1'' = 1/2\ (3+0,6)1,2 \times \frac{300}{3} = \underline{220} \quad " \quad r_1 = 367 \text{ kg/mc.}$$

b)- No sentido do vão maior:

$$r_2' = 1/4 \times 200 \times 2,4 = 120 \text{ kgs/mc.}$$

$$r_2'' = 1/4 \times 300 \times 2,4 = \underline{180 \quad "} \qquad r_2 = 300 \text{ kg/mc.}$$

LAJE: B 2

1) - Vãos: Útil .... 1,60 x 2,65
Teórico ..... 1,70 x 2,80

2) - Cargas:

$$p = 500 \text{ kgs/mq.}$$

Redução pela dupla flexão

$$a = \frac{2,80}{1,70} = 1,6$$

3) - Momentos flectores

No sentido do vão menor
a meio e nos apoios

VIGA: B 3

1) - Vãos: 2 tramos continuos de $_1 = \; _2 = 4,20$

2) - Cargas

Nos dois tramos

a)- Sobrecargas:

Reacção das lajes	129	kgs/mc.
Reacção do pavimento	200	"
s =	329	"

b)- Permanentes

Reacção das lajes	193	kgs/mc.
Reacção do pavimento	140	"
Pêso de 1 pano de tijolo burro a 1 vez	380	"
Pêso proprio	200	"
p =	1913	"

3) - Momentos flectores (ver B.A. II de G.Magnel)

A)-No apoio central:

$$M_s = -0,125 \times 329 \times \overline{4,2}^2 = - 726 \text{ kgm.}$$
$$M_p = -0,125 \times 1913 \times \overline{4,2}^2 = - 4220 \text{ "}$$
$$M\,(s + p) = 4946 \text{ "}$$

B)-Nos tramos (ameio)

$$M_s = + 0,0938 \times 329 \times \overline{4,2}^2 = 545 \text{ kgm.}$$
$$M_p = + 0,0938 \times 1913 \times \overline{4,2}^2 - 0,025 \times 1913 \times \overline{4,2}^2 = 2323 \text{ kgm.}$$
$$2868 \text{ "}$$

4) - Esforços transversos

$$T''_{As} = T'_{Cs} = + 0,433 \times 329 \times 4,2 = 598 \text{ kgs.}$$
$$T''_{Ap} = T'_{Cp} = +0,433 \times 1913 \times 4,2 - 0,04 \times 1913 \times 4,2 = 3158 \text{ "}$$
$$3756 \text{ "}$$

$$T'_{Bs} = T''_{Bs} = -0,567 \times 329 \times 4,2 - 0,049 \times 329 \times 4,2 = 852 \text{ kgs.}$$
$$T'_{Bp} = T''_{Bp} = -0,567 \times 1913 \times 4,2 - 0,048 \times 1913 \times 4,2 = 4949 \text{ "}$$
$$5801 \text{ "}$$

5) Dimensões

$h_T = 35$ $h = 31$ $b = 23$ $b_o = 4,5 \times 10 + 23 > 50$

6) - Armaduras

A)-No apoio central (h_t = 45   h = 41)

$$B = \frac{41}{\sqrt{\frac{494600}{23}}} = 0,279 \qquad R'_b = 50 \text{ kgs/cmq.}$$

$$\gamma = 0,9$$

Aa = 0,01282 x 41 x 23 = 12,1 cmq.

A'_a = 0,9 x 12,1 = 10,9 cmq.

Adoptam-se: 4 ∅ 3/4 + 2 ∅ 3/8 (11,46+1,42=12,88 cmq.) em tracção
4 ∅ 3/4 (11,46 cmq.) em compressão

B)-Nos tramos

$$B = \frac{31}{\sqrt{\frac{286800}{50}}} = 0,4 \qquad R'_b = 41 \text{ kgs/cmq.}$$

Aa = 0,0058 x 31 x 50 = 9 cmq.

Adoptam-se: 4 ∅ 3/4 em tracção (11,46 cmq)

7) - Estribos   Tensões tangenciais

Apoios extremos:

$$t = \frac{3756}{0,75 \times 31 \times 23} = 7 \text{ kgs/cmq.}$$

Apoios central:

$$t = \frac{5801}{0,75 \times 41 \times 23} = 8,2 \quad "$$

Momentos resistentes:

No vão: M_r=11,46x0,887x31x1200 = 3780 kgm.

No apoio: M_r=12,88x0,872x41x1200 = 5540 "

$(T/z)_{max}$ = 8,2 x 23 = 190 kgs/cm.

(T/z)betão = 4 x 23 = 92 "

(T/z)2 ∅ 3/4= $\frac{5,7 \times 1200}{28}$=245 "

Adoptam-se estribos de 2 ∅ 5/16 afastados de 18 cm.

8) - Reacções dos apoios

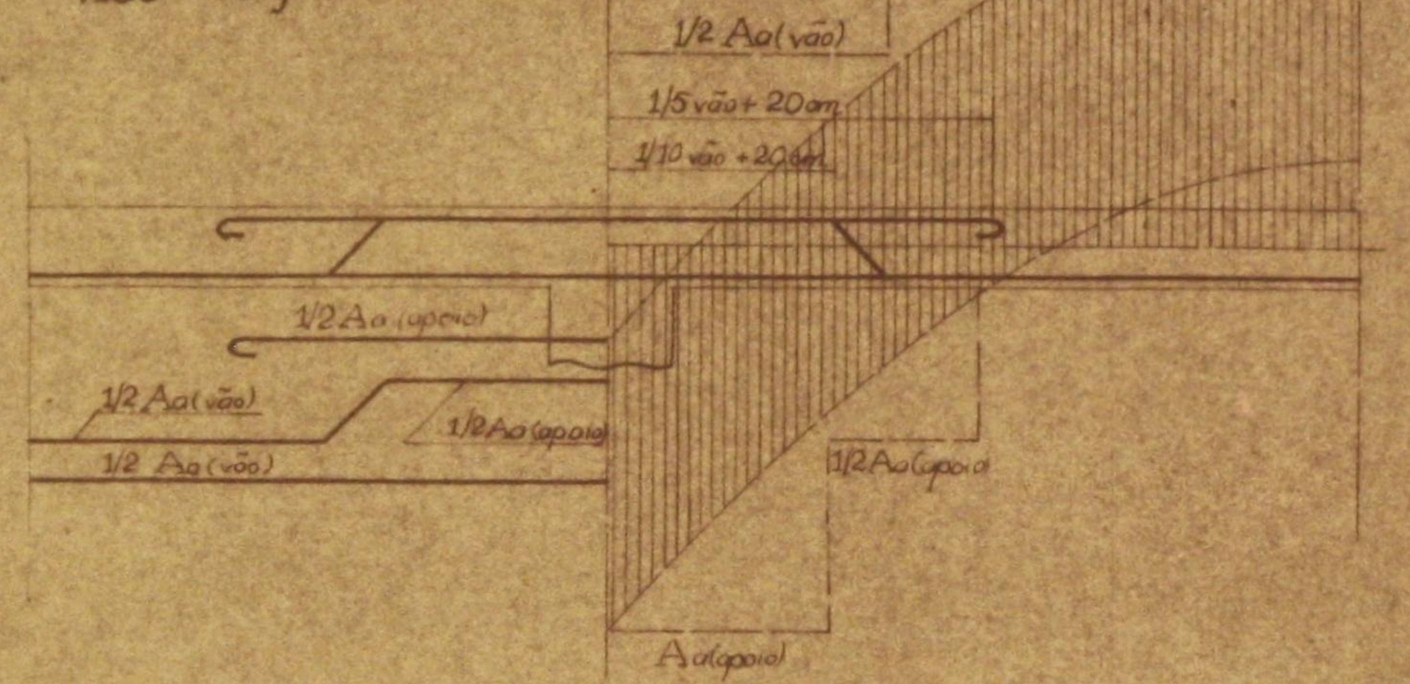
$R_A = R_C$ = 3756 kgs.

R_B = 11602 "

QUADRO SINOPTICO DAS LAGES

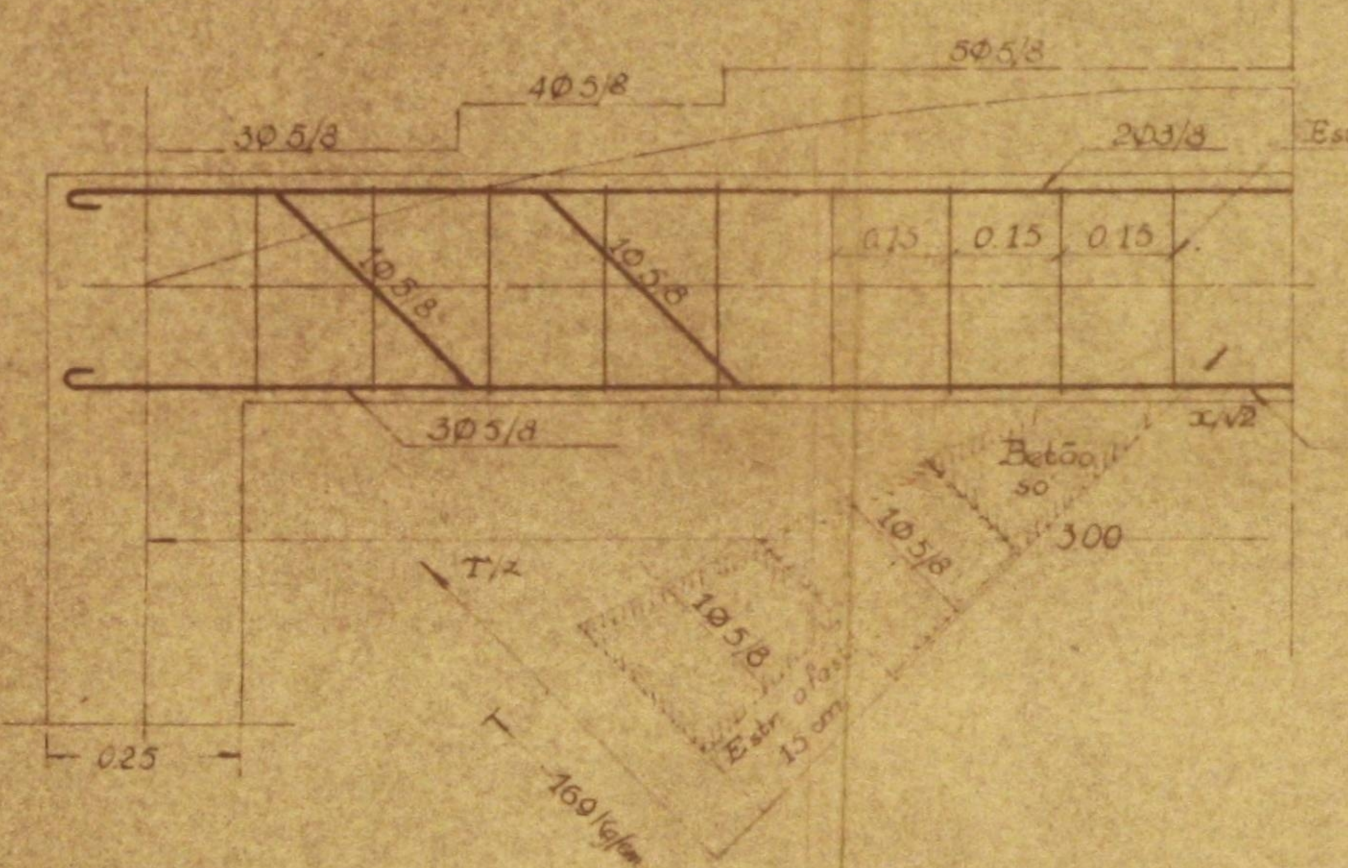
Lages	Vãos axb	Espessura	Altura util	Armadura vão menor		Armadura vão maior		Observ.
				no meio	nos apoios	no meio	nos apoios	
B_1	2.40x3.00	10	8.5	9Φ1/4	9Φ1/4	9Φ1/4	9Φ1/4	
B_2	1.70x2.80	10	8.5	9Φ1/4	9Φ1/4	8Φ1/4	8Φ1/4	
B_3	2.00x2.80	10	8.5	9Φ1/4	9Φ1/4	8Φ1/4	8Φ1/4	
B_4	1.10x5.00	10	8.5	8Φ1/4	8Φ1/4	7Φ1/4	7Φ1/4	
B_6	1.10x3.10	10	8.5	8Φ1/4	8Φ1/4	7Φ1/4	7Φ1/4	

Acondicionamento das armaduras nas lages



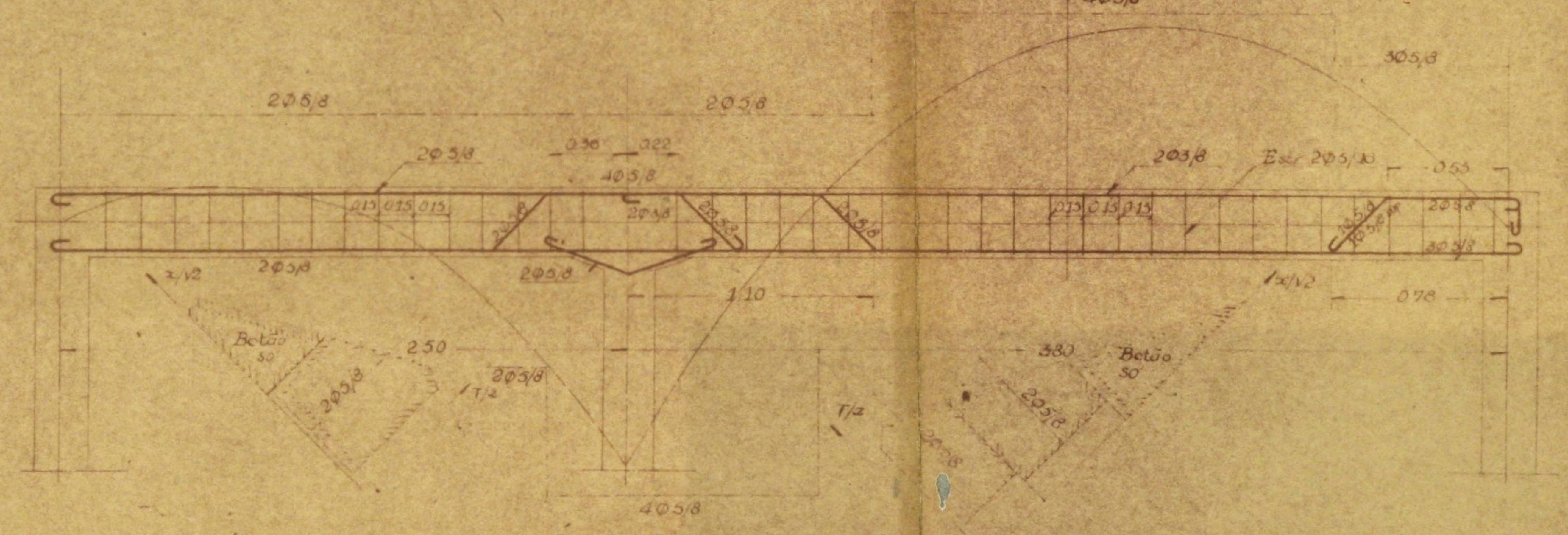
VIGA B_1

h_t = 30 cm b = 23 cm



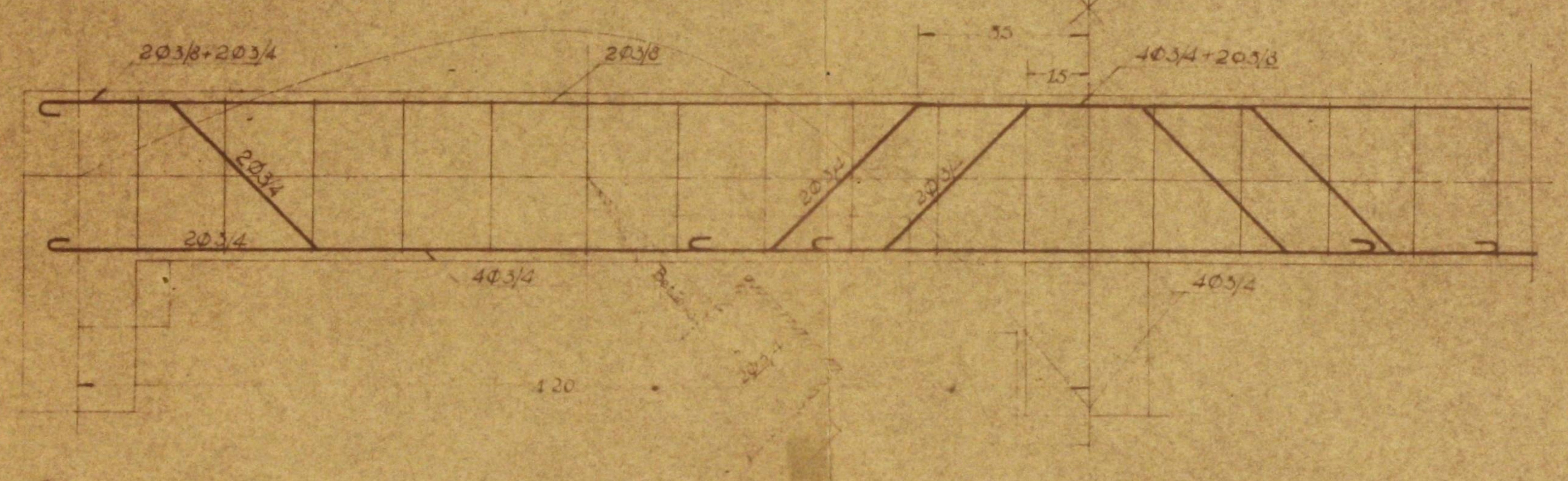
VIGA B_2

h_t = 30 cm b = 23 cm



VIGA B_3

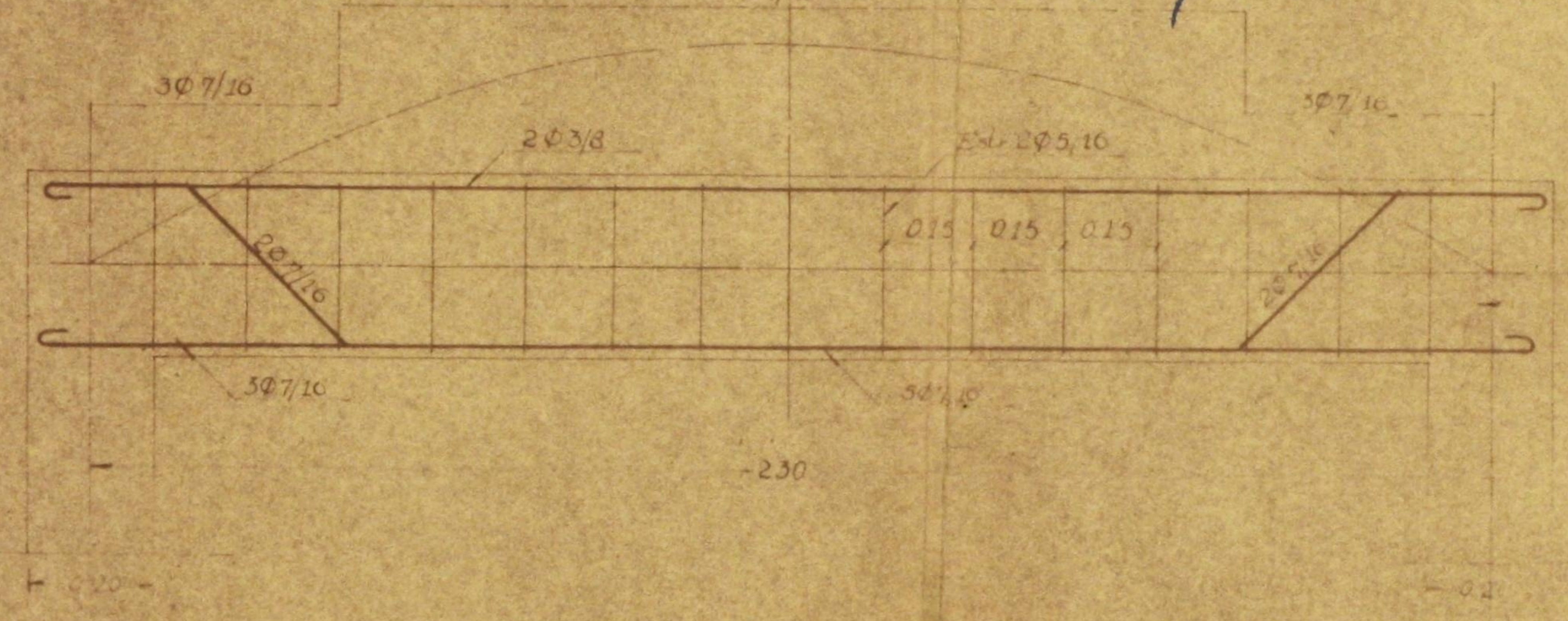
h_t = 33 cm b = 23 cm



VIGA B_4

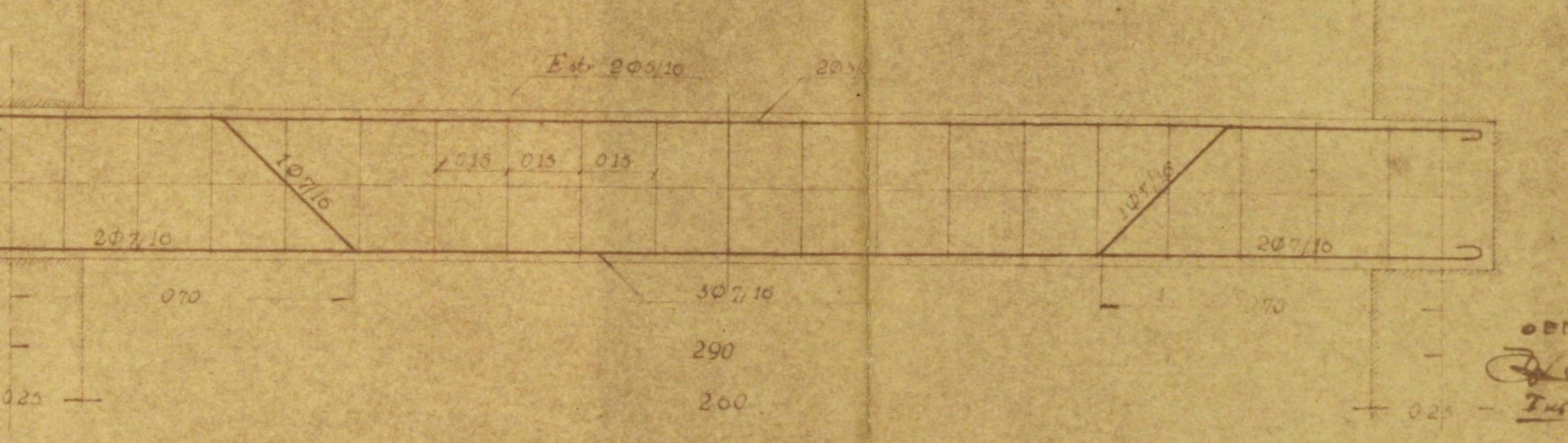
Segue
Suprimida
8/VI/39

h_t = 30 cm b = 23 cm



VIGAS B_5 e B_6

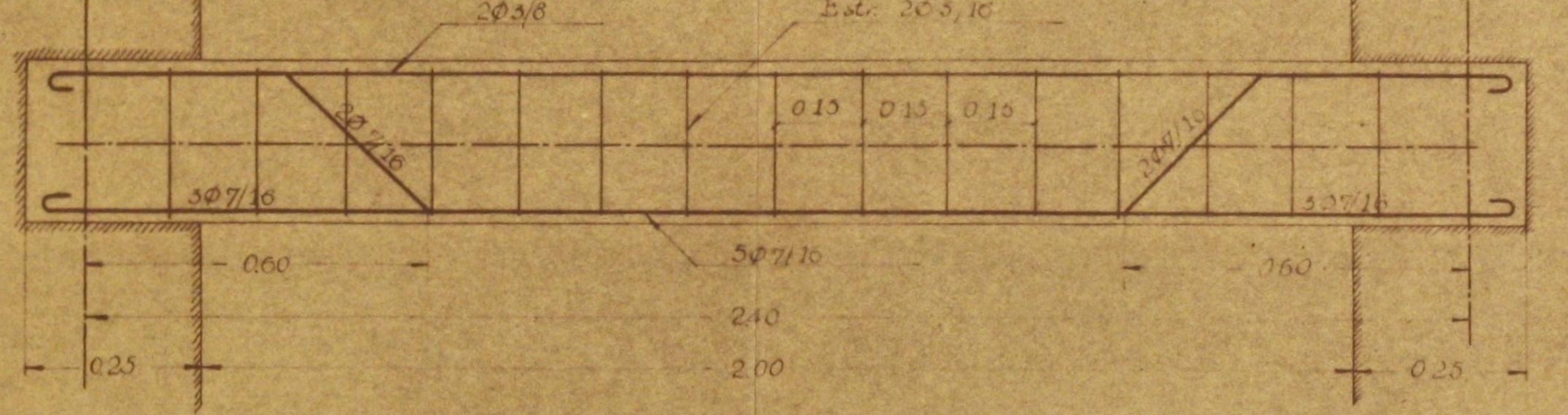
h_t = 30 cm e b = 13 cm



O Engenheiro
Insc. nº 77

VIGA B_7

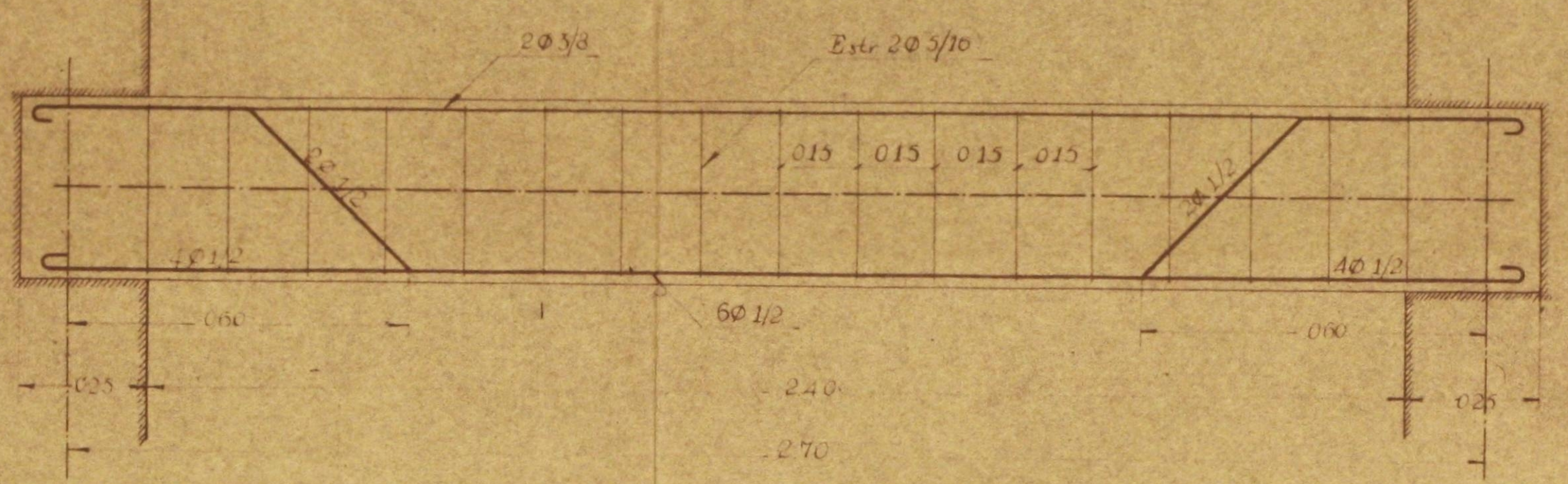
h_t = 28 cm b = 23 cm



VIGA B_8

3° Andar

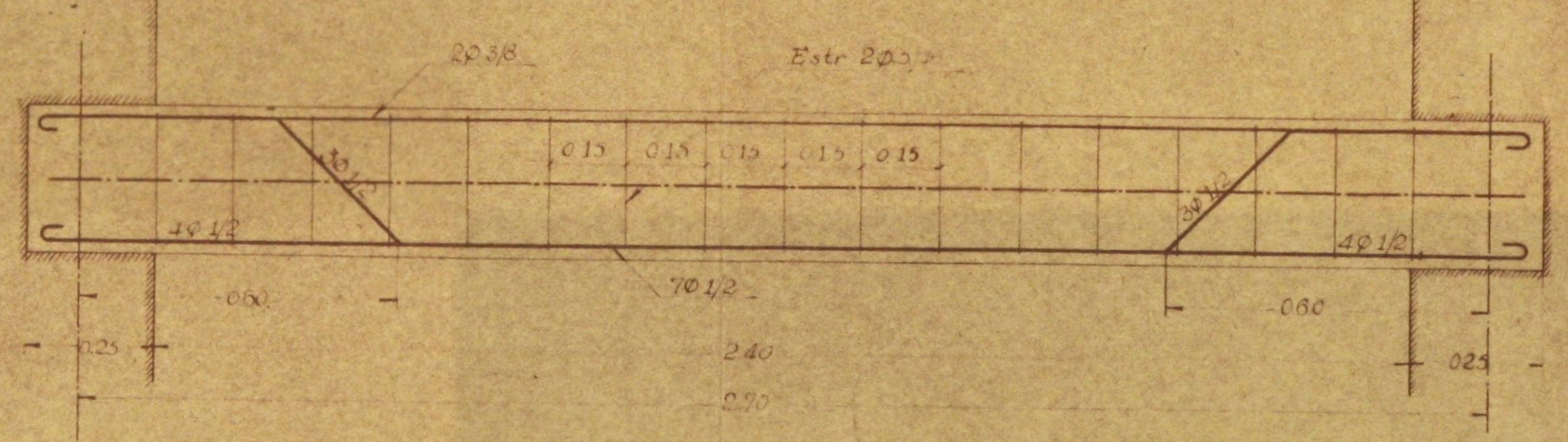
h_t = 35 cm b = 36 cm



VIGA B_8

1° Andar

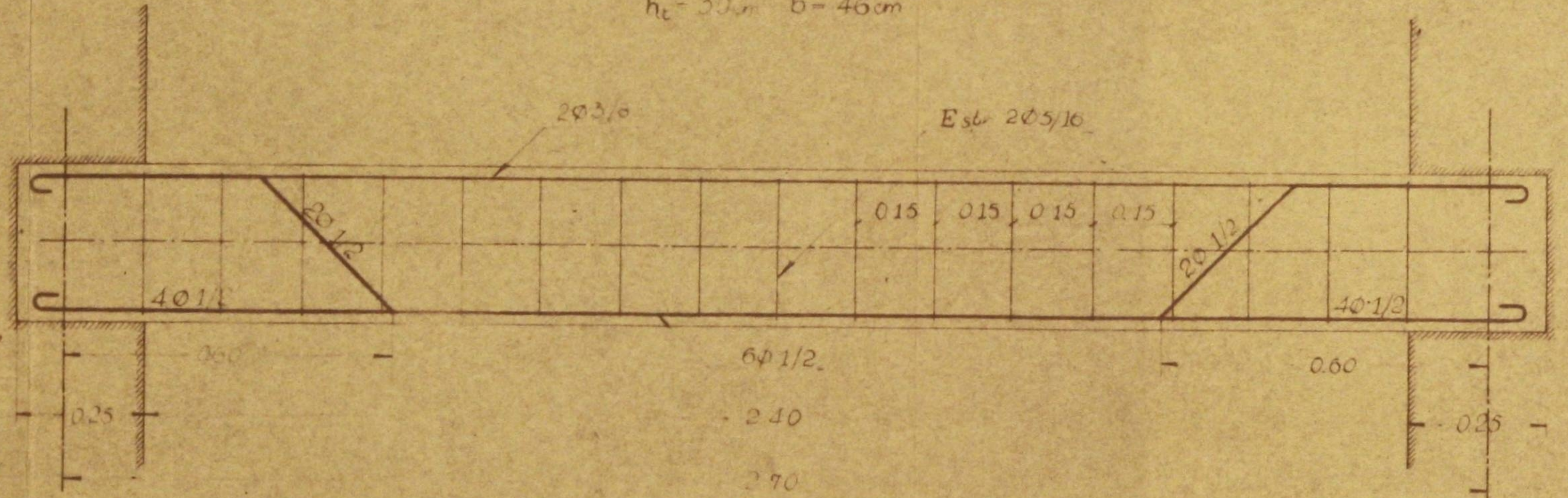
h_t = 28 cm b = 50 cm



VIGA B_8

2° Andar

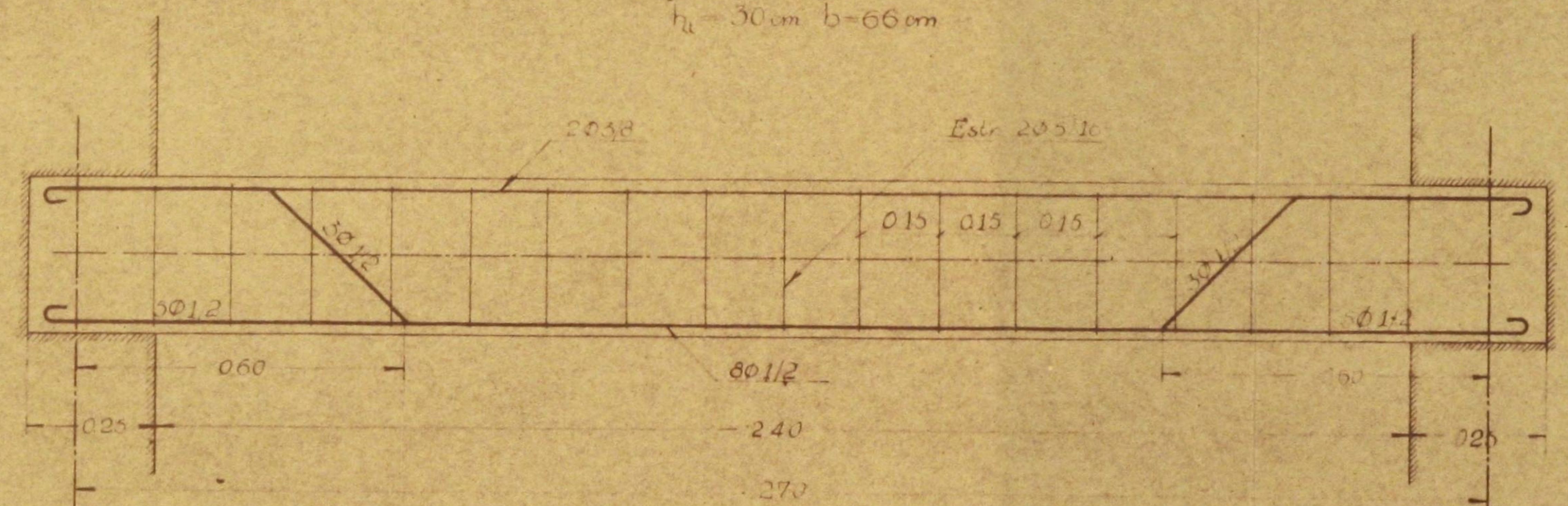
h_t = 30 cm b = 46 cm



VIGA B_8

R.C.

h_t = 30 cm b = 66 cm

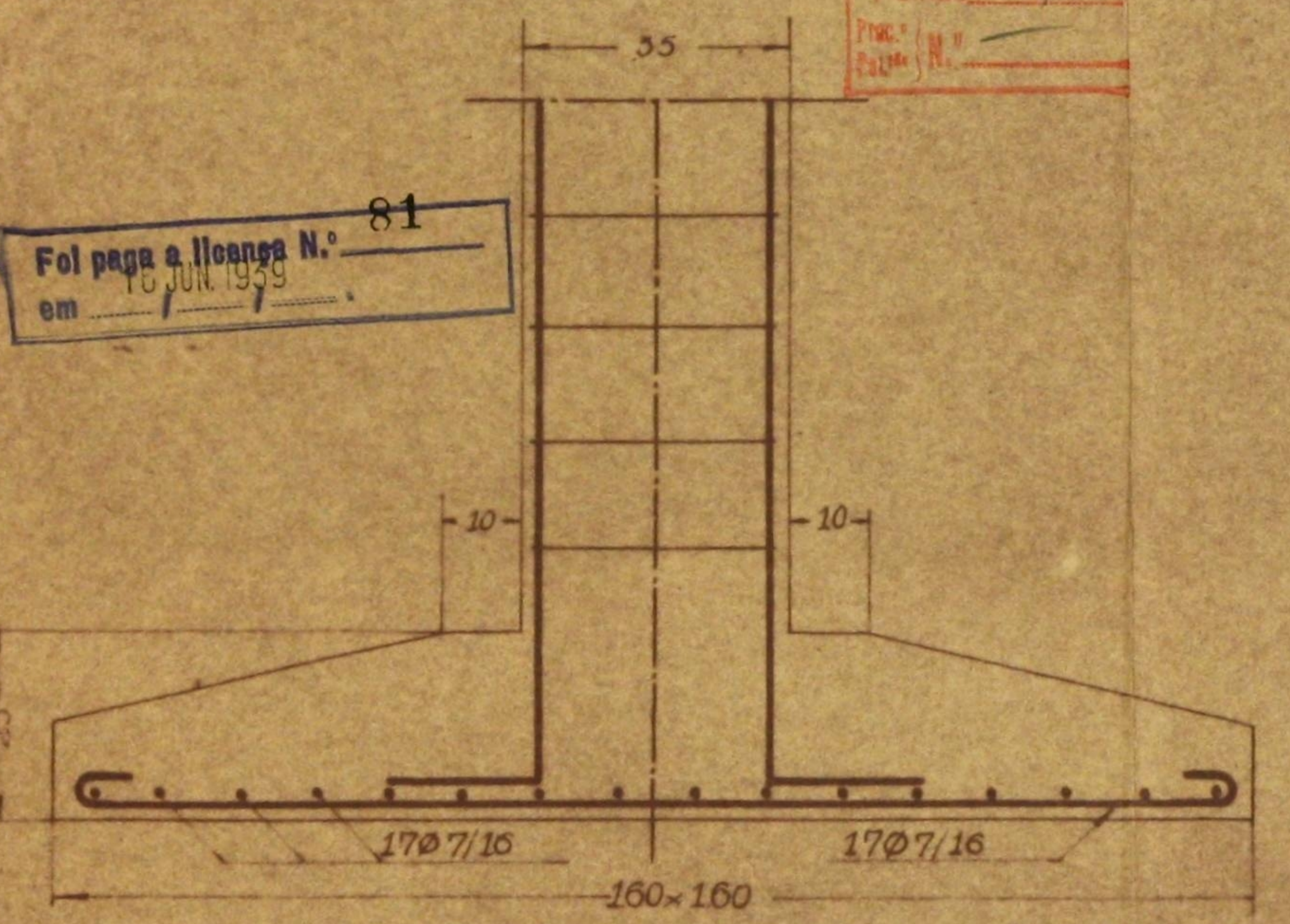


O ENGENHEIRO:

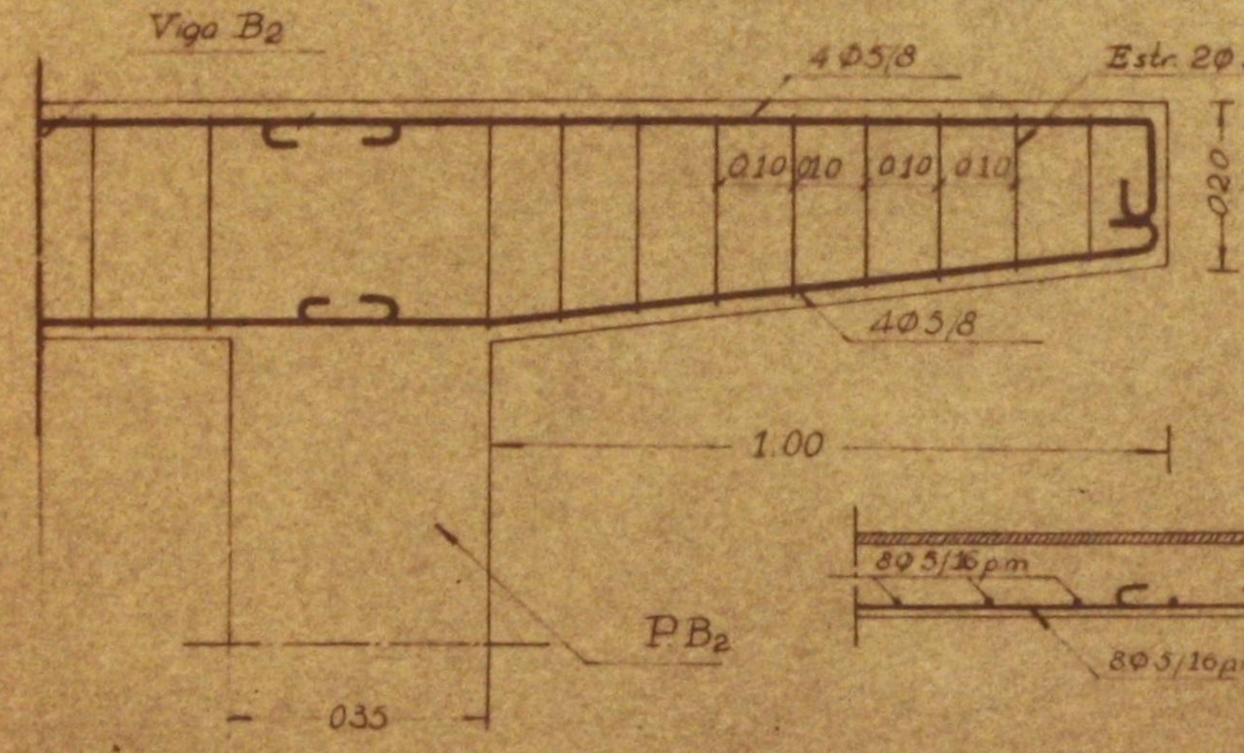
Insc. n.° 97

Fôlha 2

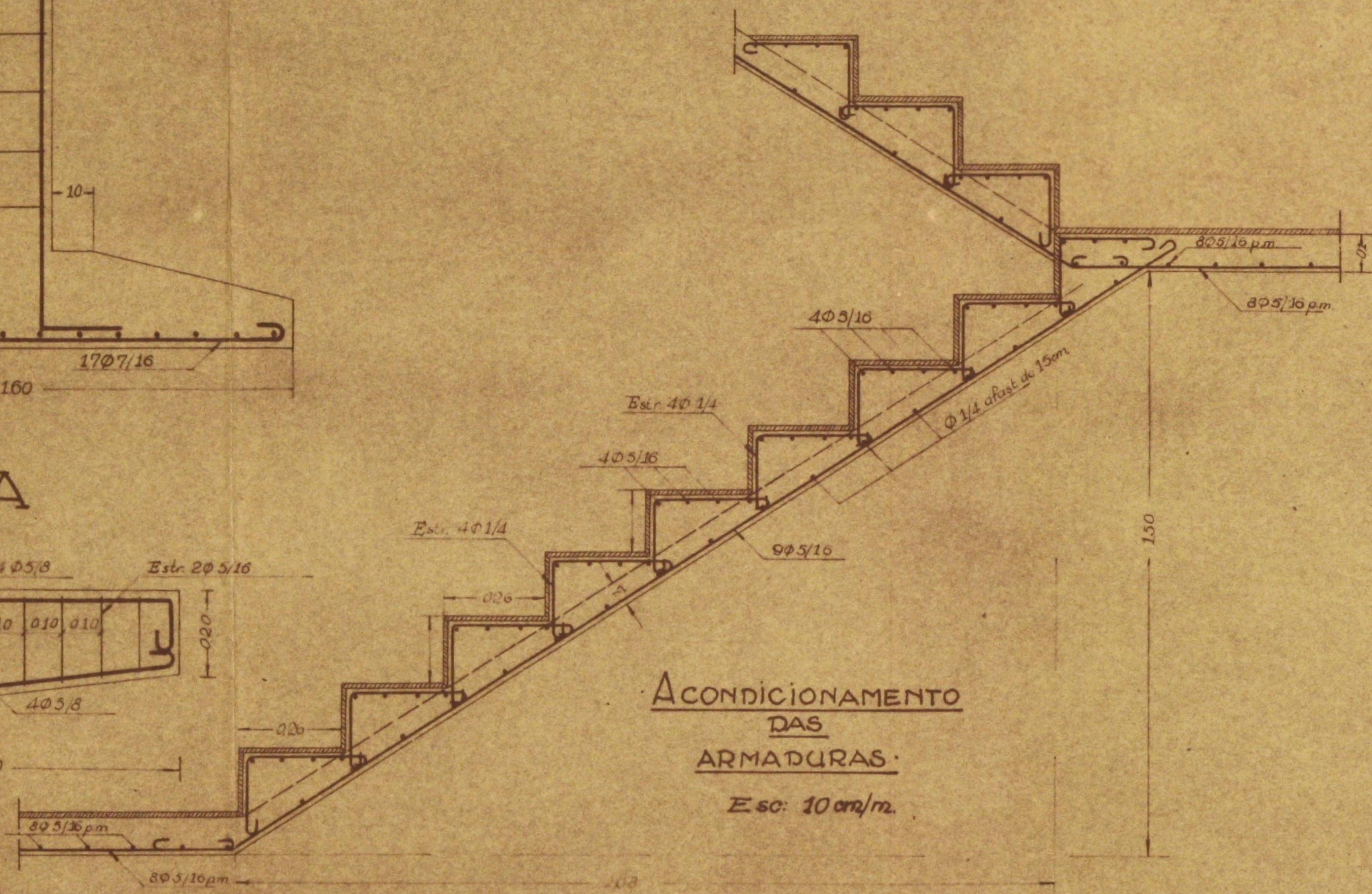
SAPATA TIPO



CONSOLA



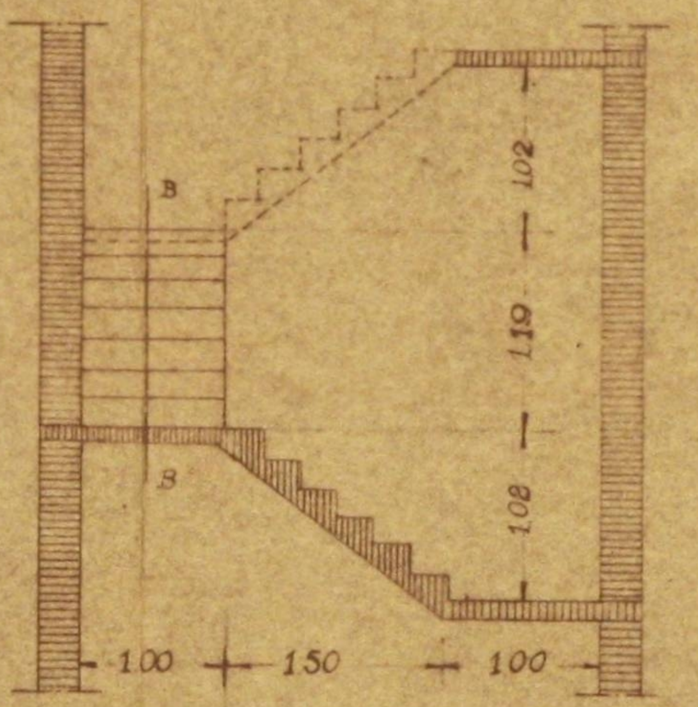
ESCADA DE SERVIÇO



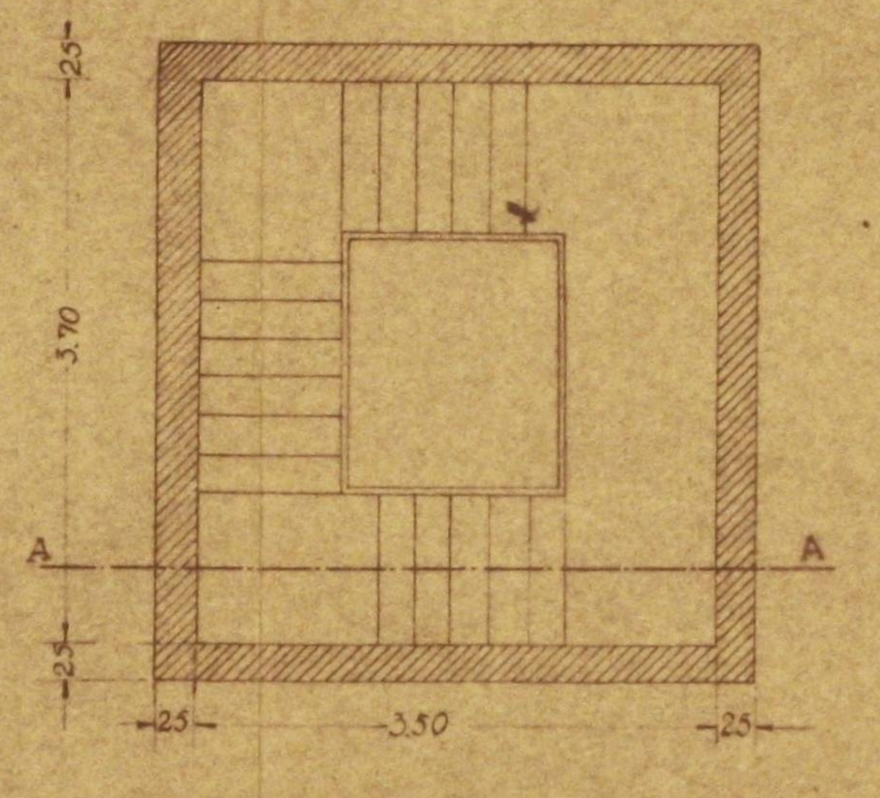
ACONDICIONAMENTO DAS ARMADURAS

Esc: 10 cm/m.

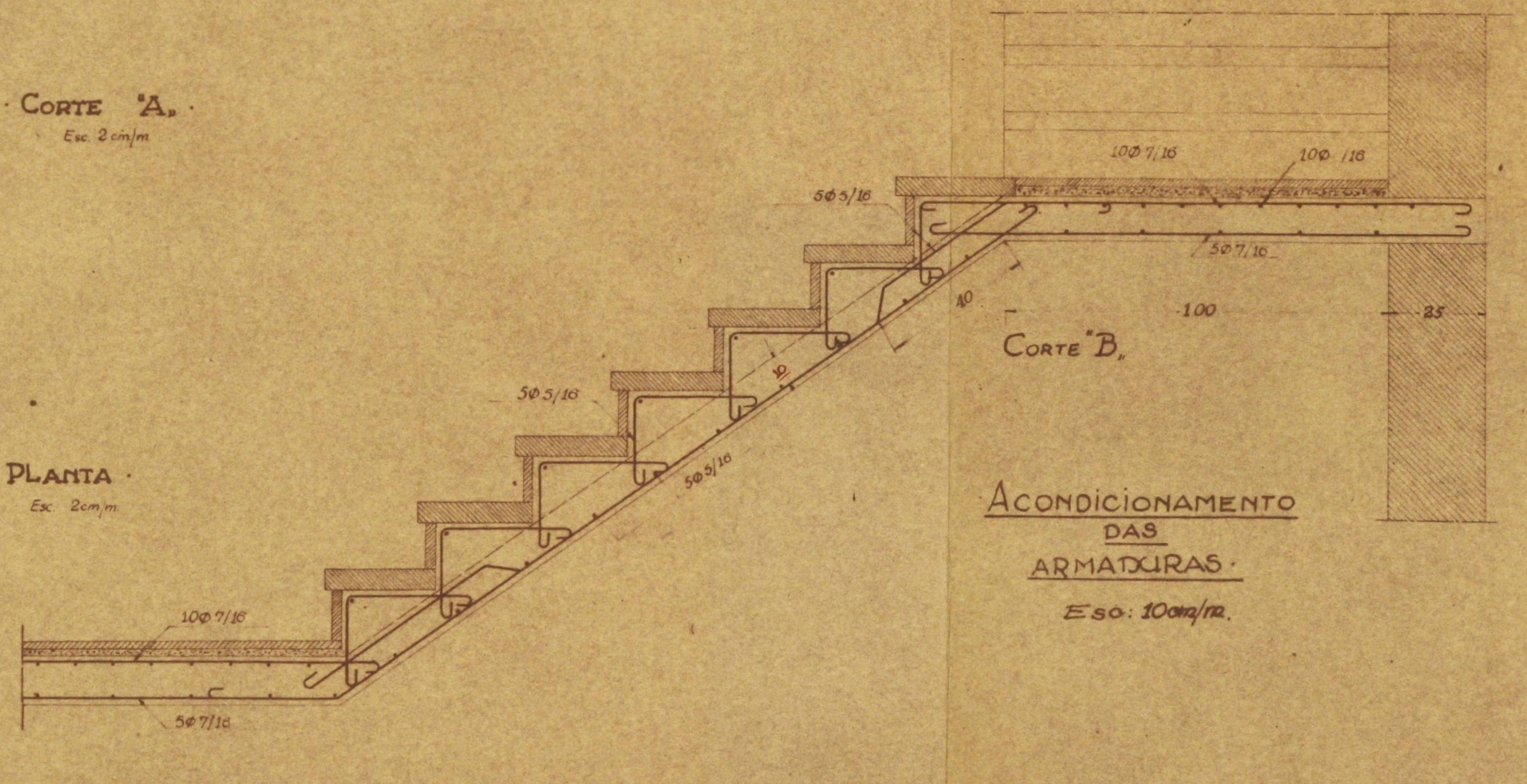
ESCADA PRINCIPAL



CORTE "A"
Esc. 2 cm/m



PLANTA
Esc. 2 cm/m



CORTE "B"

ACONDICIONAMENTO DAS ARMADURAS

Esc: 10 cm/m.

O ENGENHEIRO:

Ins. nº 77

ANNEX 2 – Processing of the results from the dynamic characterization tests.

The digital measuring device supplies a series of discrete-time data. In the case where the function is not periodic, it is difficult to make the analysis through of a Fourier series (n linear combination of sinusoidal functions) (Eq. 1). Thus in case of a discrete sample data it is possible determine a Fourier Transform (DFT - Discrete Fourier Transform), where the integral is determined numerically. The expression that defines the DFT is as follows (Eq. 2).

$$f(v) = F_t[f(t)](v) = \int_{\infty}^{\infty} f(t) e^{-2\pi i v t} dt \quad \textbf{(Eq. 3)}$$

$$F_n \equiv \sum_{k=0}^{N-1} f_k \, e^{-2\pi i v/N} \quad \textbf{(Eq. 4)}$$

In this study we used the method of Fast Fourier Transform (FFT - Fast Fourier Transform), which is an algorithm for determining the DFT of a discrete sample data. This method reduces the number of multiplications N^2 to $N.\log_2 N$ for the conventional algorithm, where N is the size of the register. This algorithm has the advantage of requiring less processing time.

One of the limitations of the algorithm FFT is that require samples whose size is integer power of 2.

Several tests were performed as mentioned in Chapter 3, whose signal obtained is shown in the graph above. Each set of signals is divided into groups 2^{12} (4096) elements. The signals are separated by intervals of time of 0.004 seconds, with a total duration of 16.384 sec.

With the program Microsoft Excel, was calculated DFT function for the same duration of signal (16.384s).

For the design of the Fourier spectrum was necessary to convert the time intervals in frequency ranges in order to be able to identify the natural frequencies of vibration. These are identified by the maximum of the function for the lowest frequency.

For each step of the Fourier spectrum function is defined frequency ranges according the following equation:

$$Df_n = \frac{1}{n \times 0{,}004} \quad \textbf{(Eq. 5)}$$

Where ‘n’ is the step.

After realize the different tests, it was possible to run the average of the values of the Fourier spectrum function for each frequency. This process helps to eliminate effects of ambient noise in the individual estimates. So now it is possible to obtain the graph of the function for Fourier spectrum after removal of noise and fluctuations (Proença, 1989) and (Guerreiro and Azevedo, 2004).

ANNEX 3 – Other studies

There are still very few studies regarding the ‘Placa’ buildings, because, although there are many examples of these buildings in Lisbon, this transition period was very fast, less than 20 years.

However, there are some documents that show some characteristics of buildings on the same time of construction and with the same type of structure, which support the frequency values obtained in the dynamic characterization tests realized and also the mechanical material properties of the elements used.

The Figure 1 shows the location of ‘placa’ buildings that knows the fundamental dynamic characteristics, based in dynamic characterization tests. All of these buildings are also ‘Rabo de Bacalhau’ buildings and include the building in study.

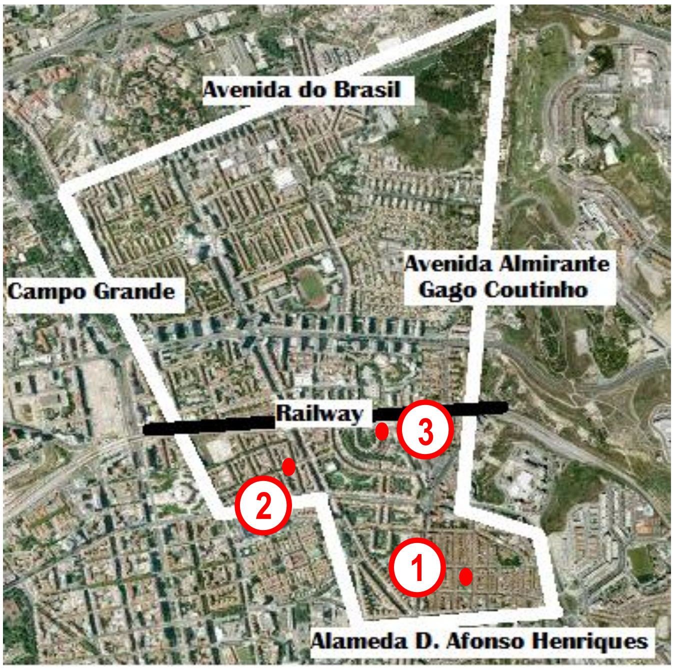


Figure 1 - Location of ‘placa’ buildings who know the dynamic characteristics (Google Maps)

Table 1 - Dynamic characteristics of ‘placa’ buildings.

‘Placa’ Building	Date	Floors	f_x (longitudinal)	f_y (transverse)
Rua Actor Isidoro, nº 13	1940	4	4.45	5.1
Rua Augusto Gil nº 10	1940	5	4.1	4.3
Praça Afrânio Peixoto nº 2	1950	6	4.2	4.4

In the following a brief description of the characteristics of the building located on the Rua Augusto Gil (nº10) is presented.

Rua Augusto Gil, nº 10 (Oliveira, 1997; Oliveira and Navarro, 2010)

Since the early nineteen seventies Oliveira and Navarro (2010) have been measuring the in-situ dynamic characteristics of the different structures built in Portugal, essentially based on ambient vibration and using expedite techniques. A *data-base* containing not only the fundamental dynamic characteristics of those structures but also their most important geometric and constructive properties has been created with the aim of setting correlations between construction typologies and fundamental periods or frequencies, and damping characteristics, and calibrate numerical modelling of those structures.

In these data-base there is one 'Placa' building in Lisbon was also completed in 1940 and is located at Rua Augusto Gil No. 10, belonging to the S. João de Deus district (Figure 2). This building is inserted into a quarter, but the buildings have not all the same characteristics.

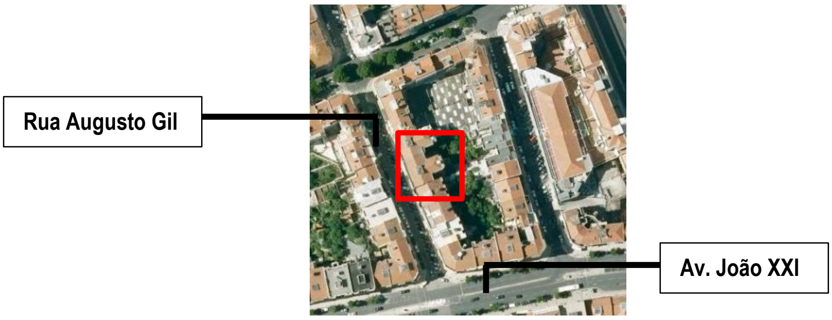


Figure 2 - Location of the 'Placa' Building in Rua Augusto Gil (Google Maps)

This example corresponds also to the 'Rabo de Bacalhau' buildings, which is the most representative in this period (1940-1960). The plant characteristics (Figure 56) are almost the same in relation to the previous building in Rua Actor Isidoro, n. 13.

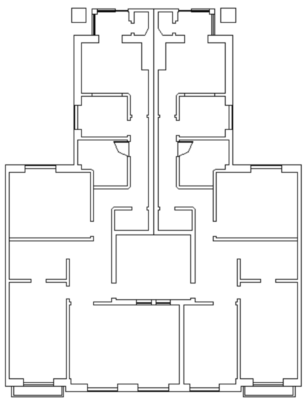


Figure 3 - Plant of the High Floors

According to the information this building was targeted of rehabilitation works recently. However these works were not structural works, were just at the level the repair of the façades, roofs and staircase (Figure 4). The building in study has five floors (ground floor and four floors high), as shown in Figure 5.

Figure 4 – Façade of the building before the rehabilitation works

Figure 5 - Façade of the building after the rehabilitation works

This building has the same structural elements than the building on study. The façade wall is made by rubble masonry, the gable walls by concrete blocks and the interior walls made by brick masonry. In relation to the floors, on the front there is a wooden floor and on the back there is a concrete slab supported on the reinforced concrete frame.

During the rehabilitation works it was possible take some photos of the buildings elements (Figure 6 to Figure 9).

Figure 6 – Photograph of the gable wall.

Figure 7 - Photograph of the stair wall on the coverage.

Figure 8 - Wooden structure on the coverage

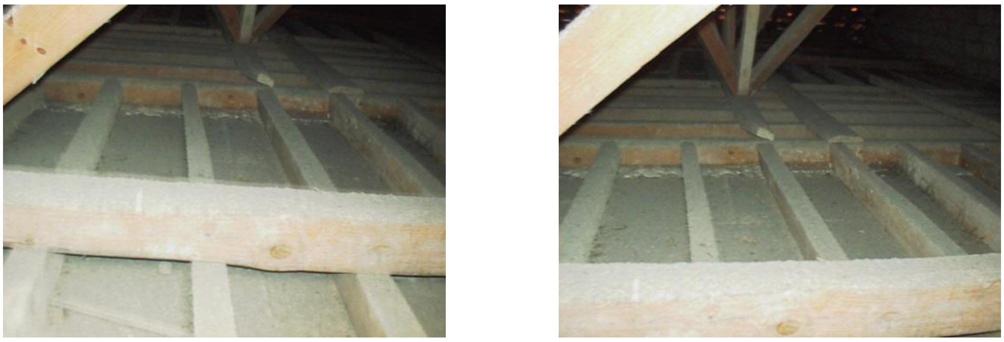


Figure 9 - Wooden floor on the roof

Initially, it was built the model of the building (considering also the effect of the quarter), as shown in the next figure (Figure 10), but only when was performed the dynamic characterization tests in the building is that it was faced with the high frequencies, as obtained for the building under study.

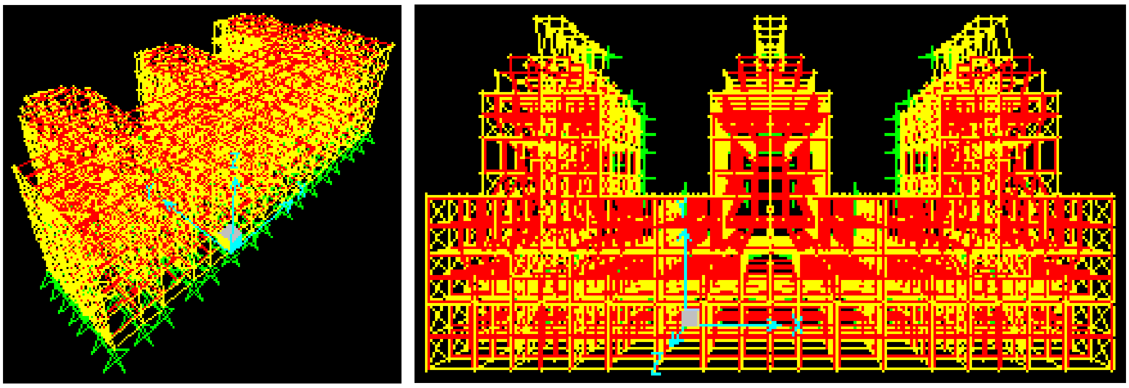


Figure 10 – Building Model considering the effect of the quarter

The Table 2 presents the comparative results between the model and the experimental values.

Table 2 - Comparison between the fundamental frequencies of the model and experimental tests

Description	Ux [Hz]	Uy [Hz]	Rz [Hz]
Experimental	4.1	4.3	5.6
Model	2.28	2.91	2.94

In addition to the buildings above mentioned, there are another school buildings that can be used to support this study. Although it is not a building of habitation is a good example for comparison with the study case since the structure has the same date of construction, has the same characteristics and has a similar behaviour to quarter, since it is a long building in plan.

These schools (D. João de Castro and Sá da Bandeira) were targeted interventions within the scope of the national schools' modernization programme.

D. João de Castro School, Lisbon (Proença and Gago, 2011; Gago et. al, 2008; Gago and Proença, 2009)

The D. João de Castro School is in the Alto de Santo Amaro, parish of Alcântara, in Lisbon, and is located halfway up a natural slope, facing south. The building was designed by José Costae Silva in 1945. It was built between 1946 and 1949 and originally had three floors, one of which was partly underground (Figure 64 and Figure 65).

The modernization of the D. João de Castro School by Parque Escolar, EPE took place of the Schools Modernization Programme for Secondary Education.

Figure 11 - Escola de D. João de Castro

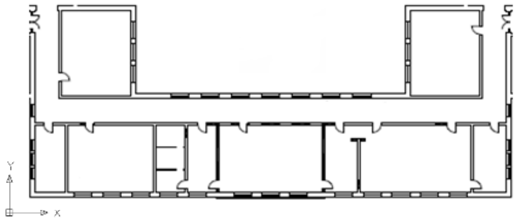


Figure 12 – Main Building

Description of Existing Construction

Basically, the structure of the original building consisted of load bearing walls made of stonemasonry (the exterior ones) and voided concrete blocks (the interior ones), which support the horizontal elements, beams and slabs, of reinforced concrete. The building's exterior outline and the earth retaining walls are made of stone masonry and are between 35 and 80cm thick (Figure 67). The interior voided concrete block masonry walls are 25 to 35cm thick (Figure 66). The floor slabs are of reinforced concrete and 12 to 15cm thick, and the roof slab, also made of reinforced concrete, incorporates precast reinforced concrete joists, voided concrete blocks and cast in place concrete screed (Figure 15).The floor slabs and the roof slab, reinforced in one direction with smooth steel rebars, are supported on the exterior walls (stone masonry), on the interior walls (voided concrete blocks) and on the reinforced concrete beams, with these last being supported on the exterior masonry wall sand interior walls. On the ground floor, in the covered courtyard areas and the main entrance of the original building, where space had to be freed up to facilitate movement, the masonry walls were replaced by reinforced concrete columns, which support reinforced concrete beams with a generous cross-section, and these in turn support the voided concrete block masonry walls of the upper floors. The foundation ground exhibits good strength characteristics and so the foundations are direct.

Figure 13 – Voided concrete block interior masonry walls

Figure 14 – Exterior Stone Masonry Walls

Figure 15 – Roof slab supported on inverted reinforced concrete beams

Experimental tests

The strength behaviour of the voided concrete block and stone masonry walls was assessed by experimental tests carried out. They consisted of applying horizontal and vertical loads to stretches of the two kinds of wall in both the original state (Figure 16).

Figure 16 - General view of the two experimental models, voided concrete block and stonemasonry walls

The following table shows the results obtained:

Table 3 – Results of the experimental tests

Wall	Section [m^2]	Axial Force Applied [kN]	Compression Stress Applied [kPa]	Ultimate Axial Force [kN]	Ultimate Shear Stress [kPa]
Rubble masonry	1.05 x 0.48	230.62	768.73	131.46	438.20
Voided concrete block	1.5 x 0.2	414.74	822.90	158.80	315.08

Besides, it was also done the dynamic characterization tests in the building. The main purpose of these tests consists identify the fundamental frequencies and the mode shapes of the building.

Based on the testes the fundamental frequency is 4.45Hz in X-direction and it is 5.1Hz in the Y-direction.

Structural Analysis

Taking into account the information collected in situ, the experimental tests and in the original design drawings, it was considered the following mechanical characteristics of the elements:

Table 4 - The mechanical characteristics of the elements considered.

Wall	Weight density (kN/m^3)	Modulus of Elasticity (MPa)	Poisson's ratio v
Rubble masonry	23	2.000	0.49
Voided concrete block	13	2.000	0.3
Reinforced Concrete	25	30.000	0.2

The **Erro! A origem da referência não foi encontrada.** presents the comparative results between the two models (isolated model and the aggregate buildings), where it can check the proximity to the experimental values.

Table 5 - Comparison between the fundamental frequencies of the model and experimental tests

Description	Ux [Hz]	Uy [Hz]	Rz [Hz]
Experimental	5.1	5.8	-
Model	5.6	4.9	5.8

The construction of a computational finite element model of the building (Figure 17), calibrated with the results from experimental modal identification tests conducted for the purpose, made it possible to understand the dynamic behaviour of the building and how to quantify the forces in each wall element. When defining the model analysis dimensions of the structural elements, taken from the geometric survey, the mechanical characteristics identified in inspections and the in situ experimental tests' results were taken into consideration. These characteristics were adjusted for the numerical model's results to be compatible with the experimental modal identification tests' results. The actions and structural safety checking criteria were defined in compliance with the regulations in force, i.e., the RSAEEP (1983).

The results of the analyses showed that most of the masonry walls, both the exterior stonemasonry ones and the interior block ones, exhibited internal forces that exceeded their bearing capacity and strength, thus requiring generalised structural strengthening of these walls. Other problems of a structural/seismic nature were identified in the same model, including insufficient strength in the reinforced concrete columns and risk of the gable toppling onto the building's main entrance.

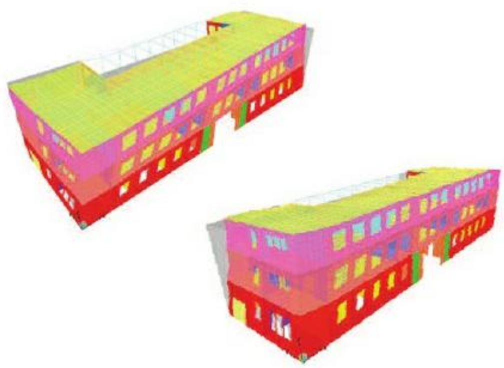


Figure 17 – Structural Analysis Model. Mode 1: translation along y-axis; Mode 2: translation along x-axis.

Seismic Strengthening Intervention

The promising test results led to this reinforced plastering mortar solution being used in the building's seismic refurbishment project. So if the exterior and interior masonry walls were identified in the numerical studies as those with the poorest strength or offering the greatest seismic risk, as Experimental tests conducted were as follows: is the case of the interior concrete block walls supporting the floors of the classrooms and corridors, they were strengthened using the proposed solutions.

In addition to the generalised strengthening of the walls, the exterior stone masonry ones were strengthened at the building's capping by improving the fixing of the reinforced concrete slab and roof slab beams to the stone masonry walls (Figure 72). This reinforcement consisted of reinforced plastering mortar with polymer mesh (the option of stretched steel mesh was discarded for reasons of durability), and the inverted beams of the ceiling slab were nailed to the masonry walls through anchorage plates. To supplement the reinforced plastering mortar strengthening of the walls the reinforced concrete columns of the covered courtyards were jacketed (Figure 71) and the gable was stabilised by being fixed to a steel frame attached to the roof slab structure.

Figure 18 – Strengthening of a reinforced concrete column by jacketing. Cross section and detail

Figure 19 – Strengthening of a reinforced concrete column by jacketing. Cross section and detail

The Escola Secundária de Sá da Bandeira is in a residential part of Santarém called São Bento. Construction of the present buildings started in 1939, following the official model for school buildings promulgated by the New State (Estado Novo). The building was opened in 1943 under the name 'Liceu Nacional Sá da Bandeira'. The original building has an approximately square plan, with an interior courtyard and comprises two floors (Figure 20).

Figure 20 – View of the school in 1943

Description of Existing Construction

Basically, the structure of the original building consisted of stone masonry walls (the exterior ones) and clay brick walls (the interior ones) that supported the horizontal elements, beams and slabs, of reinforced concrete. The floor slabs and stairs were made of reinforced concrete and there was no roof slab, just a false ceiling over the classrooms, corridors and gym.

The original building looked robust and was in a good state of repair, both structurally and in constructive terms, so efforts were made to retain the original structural philosophy and strength en only the elements that were found to have inadequate resistance

The structural behaviour of the original Sá da Bandeira school building was characterized with the aid of a computational finite element model of it, developed by ICIST (Figure 21). The model took into account the typical mechanical characteristics of the materials found in situ and it was calibrated with the experimental modal identification test results, also carried out by ICIST.

Taking into account the information collected in situ, the experimental tests and in the original design drawings, it was considered the following mechanical characteristics of the elements:

Table 6 - The mechanical characteristics of the elements considered.

Wall	Weight density (kN/m^3)	Modulus of Elasticity (MPa)	Poisson's ratio v
Rubble masonry	23	2.500	0.49
Brick masonry	13	2.500	0.3
Reinforced Concrete	25	29.000	0.2

The Table 7 presents the comparative results between the model and the experimental values.

Table 7 - Comparison between the fundamental frequencies of the model and experimental tests

Description	Ux [Hz]	Uy [Hz]
Experimental	7.73	10.87
Model	8.35	10.67

The results of the numerical model, i.e., the internal forces and stresses due to seismic action (computed according to RSAEEP), showed that in general the load bearing walls lacked resistance. The brick masonry walls inside the building, unbraced on top, presented a significant seismic risk.

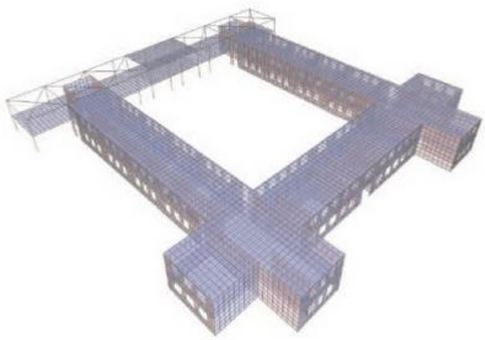


Figure 21 – Numerical analysis model

In brief, the structural refurbishment consisted of: a) strengthening the stone and clay brick masonry load bearing walls with plastering mortar reinforced with stretched steel mesh fixed to the slabs with anchor bolts; b) building reinforced concrete walls in the interior courtyard areas which had been identified as exhibiting deficient seismic behaviour; c) adding steel structures at roof slab level to connect and brace the top ends of the walls; d) and then complemented by a peripheral beam along the top which was sealed horizontally to the prefabricated cornice and vertically to the roof slab and wall.

ANNEX 4 – Mechanical Properties of Materials

Table 8 to Table 24 summarize the main mechanical properties of the structural materials obtained from the case studies referred in the precious section and other bibliographic related with old masonry buildings.

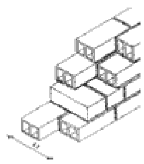
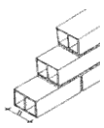
Table 8 - Rubble Stone Masonry Mechanical Properties.

Structural Material	Mechanical Properties		Author of the Study
Rubble masonry *	γ = 23 kN/m^3 E = 2.000 MPa υ = 0.49	σ_t = - 0.26 MPa τ = 0.44 MPa	Proença and Gago (2008 e 2011)
	γ = 23 kN/m^3 E = 2.500 MPa υ = 0.49	--	Proença and Gago (2009)
Rubble Stone Masonry	γ = 24.6 kN/m^3 E = 15000 MPa ξ = 5%	σ_c = 4.00 MPa σ_t = - 0.40 MPa τ = 0.14 MPa	Costa and Oliveira (1989)
	γ = 22 kN/m^3 E = 1000 MPa ν = 0.2	σ_c = 4.00 MPa σ_t = - 0.40 MPa τ = 0.14 MPa	Branco (2007)
Good Quality Rubble Stone Masonry	γ = 22 kN/m^3 E = 4000 MPa ν = 0.2	σ_c = 8.00 MPa σ_t = - 0.20 MPa τ = 0.40 MPa	Jesus (2007)
Medium Quality Rubble Stone Masonry	γ = 22 kN/m^3 E = 600 MPa ν = 0.2	σ_c = 0.90 MPa σ_t = - 0.10 MPa τ = 0.10 MPa	
Rubble Stone Masonry with air lime mortar	E = 563 MPa G = 57 MPa G = 92 MPa	σ_c = 7.4 MPa τ = 0.024 MPa	Milosevic et al. (2012)
Self compacting bentonite-lime concrete	γ = 19.1 kN/m^3 E = 750 MPa υ = 0.20	σ_c = 0.70 MPa	Candeias (2008)

* These results are the only values obtained in tests on the building typology ('placa' buildings) of the building under study.

	γ = 19.1 kN/m^3 E = 750 MPa G = 250 MPa	σ_c = 0.70 MPa τ = 0.125 MPa	Salvado (2008)
	γ = 19.10 kN/m^3 E = 779 MPa υ = 0.2	σ_c = 0.80 MPa G_c = 1.25 N/mm f_t = 0.125 MPa G_t = 0.125 N/mm	Mendes and Lourenço (2009)
Limestone Masonry	γ = 22.54 kN/m^3 E = 25000 MPa	--	Technique Tables (2003)

Table 9 - Brick Masonry Mechanical Properties.

Structural Material	Mechanical Properties		Author of the Study
Brick Masonry †	γ = 13 kN/m^3 E = 2500 MPa υ = 0.3	--	Proença and Gago (2009)
Brick Masonry	γ = 15.7 kN/m^3 E = 5000 MPa	--	Technique Tables (2003)
	γ = 14.60 kN/m^3 E = 5000 MPa	--	Costa and Oliveira (1989)
 'a uma vez'	γ = 14.60 kN/m^3 E = 500 MPa ν = 0.2 ξ = 5%	--	Branco (2007)
	γ = 3.75 kN/m^3 E = 3200 MPa ν = 0.2	σ_c = 5 MPa σ_t = - 0.10 MPa τ = 0.20 MPa	Jesus (2007)
 'a meia-vez'	γ = 2.10 kN/m^3 E = 3200 MPa ν = 0.2	σ_c = 5 MPa σ_t = - 0.10 MPa τ = 0.20 MPa	Jesus (2007)

In what concerns the values in the tables, it should be noted that almost all the references corresponding to the mechanical properties of the masonry correspond to a 'gaioleiro' buildings, because, as mentioned before, there are a very few studies about the 'placa' buildings and its materials.

So, it was tried to find some works related with buildings in the same period of construction and with the same structural elements, like the 'placa buildings'.

That is why in the Annex 3 it was mentioned the examples of the studies done by Proença and Gago (2008, 2009 and 2011) about school buildings, due to the schools modernization programme for secondary education.

† These results are the only values obtained in tests on the building typology ('placa' buildings) of the building under study.

Nevertheless, the results listed in Table 21 to Table 23 for the masonry (rubble stone and brick) are highly changeable and dispersed, revealing the need of specific and reliable information about the structural materials of 'Placa' buildings.

Table 10 - Wood Mechanical Properties.

Structural Material	Mechanical Properties		Author of the Study
Pine Wood (Floors)	γ = 6 kN/m^3 E = 6000 MPa ν = 0.2	--	Branco (2007)
	E = 8000 MPa ν = 0.2	--	Jesus (2007)
	γ = 5.80 kN/m^3 E = 12000 MPa	--	Candeias (2008)
	γ = 7 kN/m^3 E = 8000 MPa ν = 0.2	σ_c = 50 MPa σ_t = - 90 MPa τ = 5 MPa	Tabelas Técnicas (1998)
	--	$F_{arrancamento}$ = 10 kN	Lopes (2004)

Table 11 – Concrete Block Mechanical Properties.

Structural Material	Mechanical Properties		Author of the Study
Voided concrete block	γ = 13.00 kN/m^3 E = 2.000 MPa ν = 0.3	σ_t = - 0.53 MPa τ = 0.32 MPa	Proença e Gago (2008 e 2011)
	--	σ_c = 3.5 a 7.5 MPa	Tabelas Técnicas (1998)
	--	σ_c = 4 a 5 MPa	LNEC (1986)

In relation to the mechanical properties of reinforced concrete used in that time, first the table shows the values corresponding to the studies made by Proença and Gago (2008, 2009 and 2011) in the school buildings, after the values in the descriptive memory of the building in study, and finally the values in the regulations.

Table 12 – Reinforced Concrete Mechanical Properties.

Structural Material	Mechanical Properties			Author of the Study
Reinforced Concrete *	γ = 25.00 kN/m^3 E = 30.000 MPa ν = 0.2	--		Proença e Gago (2008 e 2011)
	γ = 25.00 kN/m^3 E = 29.000 MPa ν = 0.2	--		Proença e Gago (2009)
	γ = 24.00 kN/m^3	--		Descriptive Memory (1939)
Reinforced Concrete (Regulations)	--	f_{ck} > 18 MPa f_{cd} > 5 MPa	f_{yk} > 370 MPa f_{yd} > 220 MPa	RBA (1935)
	--	B180 f_{ck} = 18 MPa	A24 f_{yk} = 240 MPa	REBA (1967)
	γ = 24.00 kN/m^3 E = 29.000 MPa ν = 0.2	C16/20 f_{ck} = 16 MPa f_{cd} = 10.7 MPa	A235 f_{yk} = 235 MPa f_{yd} = 204 MPa	Eurocode (2008)

ANNEX 5 – Survey of the Reinforced Concrete Elements in the Structure and its Steel Reinforcement

The main purpose of this test it was to confirm the existence of reinforced concrete elements in the structure as well as to confirm their position according with the Descriptive Memory (1939). So some experimental tests were conducted using the cover meter. This equipment is a sophisticated device for the non-destructive location of reinforcement and for the measurement of concrete cover, using the 'Eddy Current Theory' with pulse induction as the measuring method.

Figure 22 – Cover Meter

First, it was possible to confirm the difference between the two types of floors existing in the building (concrete slab and wooden floors). At the front of the building, the device did not detect any metallic element as was expected and on the back, the equipment detected the presence of reinforcement mesh in the zone of the concrete slab. The concrete cover for steel bar recorded ranged from 2 to 3 cm.

Subsequently it was confirmed the location of reinforced concrete elements possible to identify. Based on the descriptive memory, the great doubt corresponded to the location of the beams on the façade of the building, because it did not specify if the beams were located at the level of each floor or in the upper area of the openings of the windows. Thus, the cover meter used allowed identify the existence reinforced concrete lintels on the front windows.

In relation to the concrete cover for steel bar,as is shown in Figure 23.

Table 13 presents the values recorded in some reinforced concrete elements, as is shown in Figure 23.

Table 13 - The values recorded of the concrete cover for steel bar in some reinforced concrete elements (beams and columns)

Beams	Concrete Cover (cm)	Columns	Concrete Cover (cm)
V1	3 – 4	P1	3
V6	3	P2	3
V8	3 – 4	P5	3

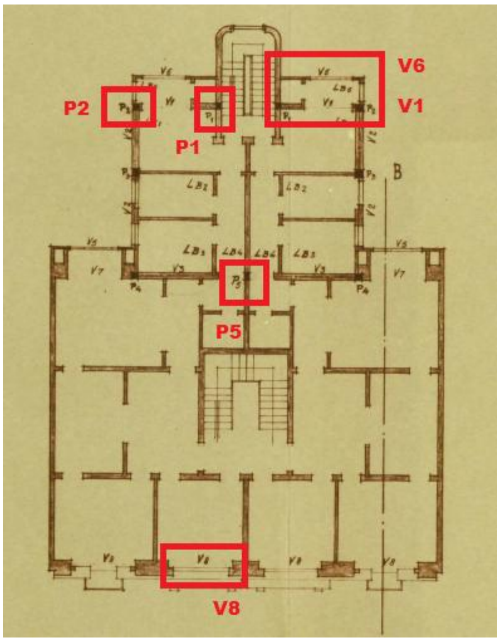


Figure 23 - The reinforced concrete elements (beams and columns) where it was made the tests

Although the values recorded for the concrete cover are not high (except for the area of concrete slabs) they are reasonable when compared with the minimum values required in the actual regulation [EC2, 2010].