The ‘Pombalino Frontal’ design and its structural efficiency

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Abstract. After November 1st, 1755, earthquake which hit and destroyed Lisbon, the Portuguese military architects innovated the constructive techniques and conceived a ‘new construction system’ with anti-seismic characteristics, the so called ‘Pombalino building’, often improperly named ‘pombalino cage’. Inserting in the ‘Pombalino cage’ a vertical ‘frontal’ element, made up of Santo André Crosse’s lattices, impresses the intellectual boldness, at that time, demonstrated by the military architects of the Kingdom, concerning the effectiveness and efficiency of the structural performance of the ‘Pombalino building’. It should be noted that we were in 1755, so the concept of stress and deformation had not yet appeared, and, therefore, mankind did not yet discover the structural calculus theory, so the concepts of bending moment and normal effort were not known at that time. Although they did not know it, the frontal element design suggests their engineers’ designers had an accurate knowledge about the phenomenology involved in their behavior, and, above all, they knew the optimal geometry to achieve structural efficiency to resist rupture of the ‘Pombalino frontal’ when an earthquake occurs. In this article we present structural calculations performed on a ‘frontal pombalino’ standard, used in the ‘Pombalino cage’, which demonstrate that the military architects, who developed the Pombalino system, had not only a perfect understanding of the St. André cross potential, but they also experimented with models to scale, because the ‘pombalino frontal’ geometric dimension is optimized in the plane of structural efficiency.

1. Introduction

The ‘Pombalino system’ was developed in the 18th century by the Portuguese Military Engineers as a structural response to the disastrous consequences of the great earthquake of 1755, given the widespread collapse suffered by most of the buildings in Lisbon, which caused thousands of casualties/deaths. The prime minister at that time was the Marquis de Pombal, a very enlightened and cult man, who took on the task of handling the reconstruction of the city of Lisbon after the earthquake, as the terrified king ran away from the capital, taking refuge in a countryside tent, named ‘royal barrack of Ajuda’.

The Marquis of Pombal, by his name Sebastião José de Carvalho, was a pragmatic and rational personality, little given to esoteric and religious explanations to justify natural disasters. The Marquis pronounced a prophetic phrase to the kingdom's military architects: “the consequences of the earthquake are not a divine punishment; it is necessary to find a constructive solution to stand a new earthquake action”. Immediately after the earthquake, he ordered the police to close the entire destroyed area, barring access to the public for two years, including homeowners, transforming the entire area into a laboratory, where architects of the kingdom could interpret, understand and elaborate theories justifying the causes of the collapses.

As a corollary of this attitude and command of the Marquis, the kingdom's military architects innovated and created a new constructive system with capacity for seismic resistance - the ‘pombalino cage’. It was the first man-made world-wide system to withstand the action of an earthquake [1].
Observations and interpretations *in loco*, given that the destroyed area was transformed into a real laboratory, carried out by the military architects, headed by the Kingdom's master engineer, Manuel da Maia, allowed the acquisition of knowledge about the reasons for the collapse of those buildings under an earthquake action - the *phenomenology of collapse*.

The kingdom’s architects learned from the ruins; they produced science. This knowledge led to the creation of a strong and potent structural system in the area of seismic resistance. The military architects rehearsed and developed a set of constructive, formal and material dispositions, seeking the new design of the ‘Pombalino building’ to achieve a greater capacity seismic energy dissipation and a better resistance to the horizontal actions.

Nowadays, the academic community in Portugal knows, with a reasonable degree of reliability, the functioning and structural behavior of the ‘Pombalino building’ existing in downtown Lisbon, however, we still know little, at the historical level, what technical and scientific assumptions are rational reached and assumed by the military architects of the ‘Marquis of Pombal’ in the development of the ‘pombaline cage’. Due to the cultural, historical and economic importance of ‘BaixaPombalina’ (downtown Lisbon) for the city, and the country, and also for humanity, in the field of seismic science, it is necessary to understand the evolution of the conceptual creative process that led to the definition and design of the pombalino cage constructive system.

In this paper we present the hypotheses and conclusions achieved through structural analysis and calculations based on the mechanical behavior of a standard frontal of the ‘pombalino cage’, which allow us to evaluate those military engineers’ degree of comprehension, led by military engineer Manuel da Maia, on the effectiveness of the performance of the frontal present in the Pombalino building.

### 2. The Pombaline System

#### 2.1. Constitution of the ‘Pombalino System’

At the formal level the main characteristic of the Pombalino buildings is their regular geometric in plan and height, where it follows the building’s symmetrical axis, respecting the two main characteristic axes in plan.

In the constructive dispositions they emphasize, among others, essentially two characteristic elements:

1. The ‘*Pombalino frontal*’, consisting of a vertical truss of Santo Andre crosses, filled and covered with a mortar impregnated with pieces of ceramic material (clay mortar), connecting the facade walls in masonry, parallel to each other;
2. Masonry exterior facade walls, with thickness on the order of the meter, containing a wooden structure inside (in the masonry interior), and reinforced on the corners of the building with high quality stone.

To such set (internal ‘frontal’ connecting parallel walls of facades in masonry) it is usually called ‘*pombalino cage*’. The designation of ‘cage’ comes from the three-dimensional arrangement of the wood elements, forming a space truss. This three-dimensional wooden structure, forming a cage, is considered by historians to be the great innovation of the ‘Pombalino construction system’. Due to its great ductility, and given that the main task of a seismic structure is balancing and dissipating energy, transmitted by an earthquake, then, it is associated to the ‘Pombalino frontal’ the principal task of resisting an earthquake, and it is considered irrelevant and subordinate the presence of the outer walls in stone masonry. However, research shows that, in the overall functioning of the system, masonry walls’ thickness plays a decisive role in the effectiveness of the system [2]. In fact, the power of the ‘Pombalino system’ results from the aggregate effect of the masonry walls, interconnected with each other, by the vertical wood frontals. The connection between the masonry walls, provided by the frontal walls, is vital for the static balance and entire building endurance. The military architects of the ‘Marquis of Pombal’ even carried out a seismic test of a model of the ‘Pombalino building’ on a natural scale in the ‘Largo do Paço’ (Downtown Lisbon) [3].

The existence of vertical walls made of solid wooden truss inside in the interior of the building—the frontal, in addition to simple partition walls, without any special structural function, arranged in the two orthogonal directions of the rectangular plan of the building, carefully connected to the external
walls, the main masonry exteriors, the master-walls, and the floors, clearly demonstrates the intention of equipping the Pombalino building with capacity to support horizontal seismic forces.

2.2. Performance and behavior of a masonry
The ‘Pombalino system’ uses the materialization of the exterior walls, the facades, in a masonry constructive solution. It is, however, a material with a brittle and non-ductile behavior, therefore, with very low capacity to deal with tensile stresses, so it has a poor dynamic behavior, very susceptible to an earthquake action. It is known that in the case of structural masonry, as in the case of ‘pombalino system façade walls’, the resistance, to be guaranteed, essentially consists of the capacity to provide compressive strength and shear strength to the wall element, since these are the internal efforts to which the masonry is apt to work, given the poor ability to deal with traction by the masonry (traction is a great obstacle to masonry as a structural material).

The resistance to the seismic action of masonry buildings then comes essentially from the thickness of the constructive shape itself. Thus, massive walls acquire resistance to the traction provided by their own weight, as this cancels the vertical traction caused by a horizontal force (this is an artificial way of pre-stressing the wall).

In structural masonry buildings, the sturdy walls are implemented in the two main directions of the building plan, forming a three-dimensional structural system and developing a structural effect called by ‘box effect’. In order to ensure the stability of the masonry building, it is important that the façade parallel walls of the building are strongly interconnected, in order to perform synchronous movements, ie, the oscillatory movement away from the plane in a given direction of a given wall, must be accompanied by equal movement, value and direction, executed by the other parallel opposite facade wall - the oscillatory movement must be absolutely equal in both walls, parallel to each other [2].

Forcing the facade walls parallel to each other to perform equal movements in value and meaning, was one of the discoveries that Marquis of Pombal’s architects discovered in their on-site observation campaigns in the real laboratory of the ‘Downtown of Lisbon’ collapsed at the earthquake of 1755. They invented the wooden frontal, with crosses of Saint Andrew, to obtain this effect of equality in the oscillatory movement, between the facade walls parallel to each other. In the ‘Pombalino building’, this coordinated and equal interaction, between the facade walls parallel to each other, is ensured by the vertical wood truss joining ‘face to face’ parallel walls - ‘the frontal’. The investigation carried out by A. Morais showed the determinant and specific role of the ‘Pombalino frontal’ in the seismic resistance of the ‘Pombalino building’[2].

The presence of the frontal, by joining the outer walls of the two parallel facades, forces the oscillatory movement of these walls to move out of their plane, locking them mutually, consequently resulting not only in less displacement of the blocks of masonry, especially at the top of the walls, after the traction breaks in the joints, but forcing equal movement, so the internal pavements don’t collapse. With this mechanism, the ‘box effect’ necessary to the overall operation of a masonry building is, although partially, developed and strengthened.

From the referred study [2], it is observed the ‘Pombalino frontal’ presence interferes to the dynamic building behavior, however it is acknowledged that this influence does not have the relevance sometimes attributed, although its enormous deformation capacity, meaning a high ductile behavior. It is concluded from this study, that the ‘Pombalino frontal’ is not the preponderant element to the seismic resistance of the building availability; that is essentially assured by the masonry external wall façade’s thickness, i.e., using the dead load as a resistance method - a self-weighting mode of resistance. The Pombalino frontal when forcing the facade walls to move together, prevents the floors of the building from falling off the walls and collapsing themselves. This is the principal role played by the frontal in the Pombalino building.

3. Structural study of the design of the pompelino frontal

3.1. Frontal design
In historical terms, the design of the Pombalino frontal closes some doubts about the real knowledge held by the military architects about the effective functioning of the frontal and, especially, its formal and geometric design (crosses of Santo André and thickness of the wooden elements) [4]. At that time,
there was no structural calculation, nor the concept of stress had been created by humanity. It is surprising that, in the absence of the theoretical tools of structural calculation, the architects of Pombalino could invented a structure and a structural system with anti-seismic characteristics, where the frontal is presented with an optimized design for the performance of the seismic function in the Pombalino building.

This research intends to contribute to clarify the extent to which the architects of the Pombalino were aware of the efficient and effective frontal design they invented. Three models were then conceived for structural analysis, with of three different geometric designs, which was performed using the SAP 2000 program. The first model does not contain crosses from Santo André; only has vertical and horizontal elements. The other two models are identical in the formal plane, showing as difference the spacing between the vertical elements, which in the standard case of the Pombalino building is 1.0 meter while, in this study, the third model presents a spacing of 1.5 meters.

3.2. Characterization of seismic action in the frontal
The research methodology adopted to analyze the effects of the seismic action on the frontal consisted of an equivalent static analysis. For the scope and objective of the investigation, this approximation is reasonable because it is a regular structure in plan and height, with predictable dynamic behavior. Under these conditions, and because the EC8 [5] allowed it, the effects of the seismic action were determined by applying to the structure a set of horizontal forces $F_i$, acting in parallel with the seismic action adopted and in the center of the respective masses, which in this study were concentrated at the level of the respective floors.

According to EC8 5.3.2, part 1.1, its values are given by the expression $F_i = \varepsilon_0 w_i$.

Where:

- $S_d(T_1)$ - ordered from the calculation spectrum for period $T_1$;
- $T_1$ - period of fundamental vibration for the movement considered;
- $m$ - mass of the building, corresponding to the area of influence of the front;
- $\lambda$ - correction factor ($\lambda = 0.85$).

In order to characterize the seismic action, the maximum acceleration of the terrain ($a_g = \gamma_1 a_{gr}$) and taking into account the values for the maximum reference acceleration in the various seismic zones in mainland Portugal, if there is a Type 2 seismic action in a 2.3 seismic zone (Lower Pombalino), we will have $a_{gr} = 1.7 \text{ ms}^{-2}$. It was considered the importance factor corresponding to current buildings, importance II $\gamma_1 = 1$. Thus, $a_g = 1.7 \text{ ms}^{-2}$.

The calculation spectrum $S_d(T)$ for the study conditions, considering the values of $T_B = 0.10$ and $T_C = 0.3$ obtained in EC8 (Table 3.3) $T_B \leq T \leq T_C$, is defined by the following expression:

$$S_d(T) = a_g S \frac{2.5}{q} \cdot \left[ \frac{T_C}{T} \right]$$

Where:

- $a_g$ - calculation value of the acceleration at the ground surface ($a_g = 1.7 \text{ ms}^{-2}$);
- $S$ - soil coefficient ($S = 1.8$);
- $q$ - coefficient of behavior ($q = 2.5$) EC8 (table 8.2);
- $T_C$ - upper limit of the period at constant spectral acceleration plateau;
- $T$ - vibration period of a linear system with a degree of freedom

$$T = C_t \cdot H^3$$

$C_t$ - 0.050 according to EC8 4.3.3.2.2 (3);
$H$ - front height.

3.3. ‘Frontal Panel’ geometric characterization
Since there were no elements to accurately characterize the geometry and constructive details of the original of the ‘frontal wall’, the structure presented in a paper at the conference “2nd ENCORE”, in Lisbon 1994, (Coias and Silva, Lourdes Alvarez, Victor Gomes, Fernando Domingues) [6] as a geometric proposal for the frontal structure (Figure 1) [6]. For the overall dimensions of the
‘frontal wall’, a framework with 12 x 6 m² was admitted, characterizing a building of 4 floors including the ground floor.

The first model reproduces a frontal wall without crosses of Saint Andrew (Figure 2). The other two models are identical in the formal geometric plane, adopting both locking of the panels with crosses of Santo André. The difference between the latter two models lies in the dimension of the orthogonal mesh of the lattice structures, consisting of diagonal locking posts and crossbars in each of the panels. In one model, spacing of 1.5 meters between uprights and crossbeams were adopted (Figure 3); the sections of all bars being enlarged to obtain a total mass identical to that of the standard structure of the ‘Pombalino building’. In the other model, spacing of 1.0 meter were used between uprights and crosspieces (Figure 4). This last model corresponds to the standard structure of the ‘Pombalino building’, according to the ones made in several works of ‘BaixaPombalina’.

The results obtained in these three models show the level of knowledge acquired by the architects of the Pombalino, since, as we shall see, the standard solution adopted in the Pombalino building is the optimized one in the seismic resistance chapter. It is surprising how these military architects, without theories of calculation, managed to reach this level of scientific knowledge, only using experimentalism observation.

Figure 1. Geometry of the frontal structure
3.4. Frontal inertia forces

For the determination of the inertial forces acting on the frontal wall and for the EC8 [5], the model that substitutes the deformed of the fundamental mode by a linear configuration, considering the equivalent forces, at the level of the floor girder (Fi) resulting from the weighting of the basal force (Fb) by the parameter hi x mi, which results respectively from the height and mass corresponding to the level of the force i, obtaining for each of the floors the equivalent force given by the following expression:

\[ F_i = F_b \times \frac{h_i \times m_i}{\sum h_i \times m_i} \quad \text{(EC8 4.3.3.2.3)} \]

c/ \[ F_b = S_d(T_1)m \lambda \]

In the determination of the shear force at the base, we considered the masses of the frontal wall, of the houses and overloads, corresponding to the area of influence of the frontal. In this analysis, the mass values of the elements were obtained from the usual actions for dimensioning structures, according to the following assumptions:

a) Frontal wall (c = 6m; h = 3m)

It was considered that the frontal wall structures in analysis consisted of profiles of pine wood totaling 1 m³ per floor.

\[ W_{pf} = 1 \, m^3 \times 6 \, kNm^{-3} = 6kN \]

b) Flooring

Assuming an area of influence of 6 x 5m² and considering the own weight of the wood structure 0,5 kNm⁻² and the overload 3 kNm⁻² we will have that, at the level of each floor, the force resulting from the weight of the floor and its overload will be:
From chapter 3.2 we obtain spectrum calculus $S_d(T)$

\[ S_d(T) = a_g \cdot \frac{2.5}{q} \cdot \left[ \frac{T_c}{T} \right] = 1.7 \times 1.8 \times \frac{2.5}{0.3} \approx 2.8 \]

Considering the previous components, it results that the cutting force in the base, according to the gravitational forces considered, will be:

\[ F_b = S_d(T) \cdot \lambda = 2.8 \times \frac{F_g}{g} \times 0.85 = 0.24 \cdot F_g = 0.24 \times 444 = 106.56 \text{ kN} \]

\[ F_i = \frac{h_i \cdot m_i}{\sum h_i \cdot m_i} \]

The forces of inertia $F_i$ in the different floors are indicated in the following table (Table 1)

| Floor | $h_i$(m) | $F_{qi}$ (kN) | $F_{qi} \cdot h_i$ (kNm) | $F_i$ (kN) |
|-------|---------|---------------|-------------------------|------------|
| 4     | 12      | 111           | 1332                    | 42.62      |
| 3     | 9       | 111           | 999                     | 31.97      |
| 2     | 6       | 111           | 666                     | 21.31      |
| 1     | 3       | 111           | 333                     | 10.66      |
| R/C   | 0       |               |                         |            |
| $\Sigma$ | 444      | 3330          | 106.56                  |            |

3.5. Characteristics of materials
We consider for wood (forces considered // to fibers) the acceptable values:

\[ \sigma_{Cmax} = 33 \text{ MPa} \]
\[ \sigma_{Tmax} = 33 \text{ MPa} \]
\[ \nu = 0.4; \varphi = 0.6 \text{ kgm}^{-3} \]
\[ E = 10 \text{ GPa} ; \alpha = 5 \times 10^{-6} \]

4. Results

4.1. Horizontal and vertical displacement at the top of the frontal

Based on the results obtained in the structural analysis of the three models considered in this investigation, described above, the displacements, the maximum stresses and the tensions in the wooden frontal bars were calculated. The calculated maximum lateral and vertical elastic displacements are observed at the ends of the frontal end, as expected:

- Frontal wall, with section 12 x 6 m$^2$, diagonals and reticulated mesh 1.0 x 1.0 m$^2$;
  \[ U_x = 0.232 \text{ m}; U_z = -0.008 \text{ m} \]

- Frontal wall, with section 12 x 6 m$^2$ with diagonals and reticulated mesh of 1.5 x 1.5 m$^2$;
  \[ U_x = 0.012 \text{ m}; U_z = -0.006 \text{ m} \]

- Frontal wall, with section 12 x 6 m$^2$ with diagonals and reticulated mesh of 1.0 x 1.0 m$^2$;
  \[ U_x = 0.009 \text{ m}; U_z = -0.006 \text{ m} \]

From the results it can be seen that the military architects of the ‘Pombalino’ had knowledge about the effectiveness of the insertion of the crosses of Santo André inside the frontal, between the crosspieces
and the props, although at that time humanity still did not develop any rational theory of calculation and design of structures (not even the concept of state of stress and deformation had been invented). Note that not only the maximum horizontal displacement decreases from 23.2 mm to 9 mm, when the St. Andreas crosses are introduced, as also the vertical displacement decreases from 8 mm to 6 mm. With a horizontal displacement of only 9 mm, we realized that the military architects of Pombalino achieved a remarkable achievement for that time, in the 18th century, as they succeeded in achieving the desired effect of equalizing the oscillating movements of the façade, walls parallel to each other, with a introduction of vertical trellis incorporating the crosses of Saint Andrew.

But they reached another level of knowledge; with a spacing of 1,0 meters the horizontal displacement is still significantly lower, since it goes from 1.2 cm to 0.9 cm. That is, they optimized the trellis design for the desired effect. This optimization effect is again evident in the calculation of the maximum stresses, as shown in the following paragraph.

The results obtained confirm the assertion expressed in the initial summary of this article. The military architects of the Pombalino evolved in their knowledge, as they watched the collapsed buildings in the Downtown Lisbon. From this visual observation, and from the consequent reflection on the real causes of the collapse, they fabricated science and acquired knowledge about the real functioning of buildings during an earthquake. It was not a coincidence the innovation of the vertical frontal connecting the facade walls parallel to each other. They knew the importance of setting the parallel walls of the façade to oscillate in synchronous motion - it was necessary to avoid collapsing the interior floors. They achieved this with the insertion of the crosses of Saint Andrew inside the trellis. The front not only ensures the movement of both walls together, but also drastically reduces the horizontal displacement, forcing a drastic reduction of stress on the facade walls in masonry.

4.2. Efforts and tensions on floor 1 of the building

The tensile stresses were calculated only for vertical truss models with spacing of 1.0 m and 1.5 m, between uprights and crosspieces. In summary, the values obtained for the stresses and tensions in the pieces that present themselves with higher tensions are presented (Table 2), which are, as expected, at the level of floor 1.

| Element | Section | Dim. (cm2) | Bar | N (kN) | V2 (kN) | M3 (kNm) | σ(MPa) | τ (MPa) |
|---------|---------|------------|-----|--------|--------|---------|-------|--------|
| Beam    | 1X1     | 17x14      | 596 | 64.3   | 11.7   | -2.3    | 6.1   | 0.7    |
|         | 1.5x1.5 | 20x20      | 68  | 44.3   | 16.2   | -4.2    | 4.3   | 0.6    |
| Plumb   | 1X1     | 14x10      | 621 | -95.4  | 0.01   | -0.01   | 6.9   | 1.5x10^{-3} |
|         | 1.5x1.5 | 17x10      | 106 | -125.5 | 0.02   | 0.06    | 9.6   | 0      |
| «Travessanho» | 1X1 | 10x10      | 663 | 53.3   | 0      | 0.02    | 5.4   | 0      |
|         | 1.5x1.5 | 14x10      | 81  | 27.4   | 0.2    | 0.06    | 2.1   | 2x10^{-2} |
| Diagonal | 1X1    | 10X8       | 729 | -45.4  | 0.2    | 0.02    | 5.8   | 3.1x10^{-2} |
|         | 1.5x1.5 | 10x10      | 129 | -43.6  | 0.1    | 0.02    | 4.4   | 1x10^{-2} |
The results confirm the high structural knowledge in the field of seismic resistance achieved by the architects of Pombalino. We know from the various historical records that, in order to optimize the operation, have carried out tests on physical models in the search for the efficient and efficient design of the frontal. Only the experiment allowed them to reach the standard 1.0 m spacing at the frontal. As shown in the above calculations, with the standard spacing of 1.0 m, the maximum tension in the plumb bob is 6.9 MPa, whereas in the case of the 1.5 m spacing it is 9.6 MPa.

The values obtained in the calculations confirm the efficiency of the Pombalino frontal design, since the obtained tensile values, although high, are below the rupture values of the wood material, confirming the high knowledge acquired by the architects of the Pombalino, although they did not have any theory of calculation and sizing, only had the observation of the collapsed buildings and the experimentation.

5. Conclusion

Considering the results obtained, it is concluded that those military engineers who made the ‘Pombalino cage frontal’ design, had a clear understanding of the overall structural and seismic behavior of the building, especially the need to interconnect and limit the displacements of the exterior walls. Although, they did not have theories of calculation and sizing, those military engineers of the ‘Pombalino’ design realized the necessity of introducing crosses of Santo André inside the vertical lattice of the frontal and managed to optimize the geometry and sections of this vertical trellis.

The ‘Pombalino Building’ innovation was a relevant intellectual leap. It was the first anti-seismic system engineered by mankind and, surprisingly, even today it follows the national seismic regulation now in force in Portugal.

6. References

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