Structural efficiency of the ‘Pombalino Frontal’ infilled with mortar

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Abstract. On November 1st, 1755, occurred a huge earthquake in Portugal which hit and destroyed Lisbon. In a previous paper, where we presented some research about pombalino system, we referred and noted that we were just in 1755, when the creation and development of the ‘pombalino building’ occurred, so the concept of stress and deformation had not yet appeared, but, despite this, the military engineers of the kingdom were able to create a remarkable construction system with anti-seismic characteristics. In the vertical frontal truss of the Pombaline System, the spaces between the vertical and horizontal plumbs are filled with mortar that contained small elements of stone and ceramic residues. This form of filling was not carried out in order to close the space, because at the time, even in the Pombaline building, the solution adopted to close space was different. This filling was designed by the engineers to provide the building with an additional energy dissipation system, although at the time humanity has not yet developed the concepts and theory of structural analysis and, obviously, the characterization and definition of the dynamic behaviour of a structure had not yet been achieved. The frontal element design suggests their engineers’ designers had at that time an accurate knowledge about the dynamic phenomenology involved in the seismic behaviour of the pombaline building. In this article we present structural calculations performed on a ‘frontal pombalino’ standard, used in the ‘Pombalino cage’, which demonstrate that the military engineers had not only a perfect understanding of the St. André cross potential, but they also know the role played by the filling mortar in the seismic response of the Pombaline building. The infill cracks when a seismic occurs and a large number of slits are formed, and each crack acts as an energy dumper (a power sink).

1. Introduction

Ordered by the prime-minister Marquis of Pombal, the military engineers of the kingdom designed the pombaline system with the main purpose of the pombaline building having an earthquake-resistant capacity. The design was done with the main intention of having the capacity to resist an earthquake, as violent as it was. Its design and creation was made with the main objective of being an anti-seismic structural system. And this purpose has been achieved. [1] As noted before, the insertion of the vertical frontal element in the ‘Pombalino cage’, made up of Santo André Crosse’s lattices, impresses the intellectual boldness, at that time, which demonstrated the inventive genius and the remarkable knowledge revealed by the military engineers in charge of the project, concerning the effectiveness and efficiency of the structural performance achieved with the ‘Pombalino building’. [2], [3] But the surprise didn't stop here. Those engineers had another powerful idea that revealed a deep understanding of the seismic behaviour and performance of the system they created and designed. They had the idea of filling the spaces between the vertical plumbs and the inclined and horizontal elements of the crosses of St. André on the vertical frontal lattice, with a lime mortar impregnated with
small ceramic and stone residues. For the time, this idea was surprising and incredible. Note that 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, but, despite this, the architects of the kingdom were able to create a remarkable construction system with anti-seismic characteristics. Those engineers knew very well that filling is a key that endows the vertical frontal lattice with extra capacity to dissipate the seismic energy. This form of filling was not carried out only in order to close the space because, at the time, the solution adopted to close space was different. This filling was designed by the military engineers, who created the Pombaline system, to provide the building with an additional seismic energy dissipation system, although at the time the humanity has not yet developed the concepts and theory of structural analysis and, obviously, the characterization and definition of the dynamic behaviour of a structure had not yet been achieved. The mortar filling, containing small ceramic and stone residues, cracked when the earthquake oscillations started and these cracks are in a large number. Since the seismic action is oscillatory, then, each crack performs a cyclic opening movement followed immediately by the crack closing. This continuous opening/closing of the mortar cracks facilitates the dissipation of the seismic energy transmitted by the earthquake. In each oscillation cycle, each crack dissipates a certain amount of energy transmitted by the earthquake. As the cracks are high in number, so the amount seismic energy dissipated by this mechanism for opening/closing reaches a high value, relieving the structural system, flattening the maximums stresses values, which increases the seismic resistance of the Pombaline building. This mechanism actually corresponds to a tool that provides the Pombaline structural system with a behaviour similar to that of the plastic phase of contemporary materials, steel and reinforced concrete. As we know, the plastic phase acts as a fuse that prevents the increase on stresses temporarily. Perceive in 1755 how this mechanism of the mortar acts in the Pombaline seismic response is a remarkable cognitive and conceptual feat. Amazing and surprisingly how these engineers in that time could innovated and achieve this solution that performs as a plastic phase on building response, when an earthquake occurs. [4]

In this article we present the conclusions of the calculations that prove the beneficial effect of the presence of the mortar to fill the spaces between the vertical plumbs and the horizontal and inclined bars of the vertical frontal lattice with their St. André crosses.

2. Structural study of the pombalino frontal’s design

2.1. Frontal design

The structural system of the pombaline building usually called pombaline cage contains in its constitution a vertical element in the entire height of the building, called Frontal that establishes the connection between the facade walls parallel to each other. The function of this vertical lattice frontal is very important, given that by establishing the connection between the facade walls parallel to each other, it forces them to move in the same direction and synchronously. The military engineers realized with the observation of the collapse of many buildings that in many buildings the facade walls remained upright and only the pavements and the roof collapsed. It was thus necessary to avoid the out-of-sync movement of the facade walls and from there they innovated and created the Pombaline frontal.

2.2. Characterization of seismic action in the frontal

The research methodology adopted to analyse 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 behaviour. 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 $F_b$, acting in parallel with the seismic action adopted and in the centre 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 $F_b = S_d(T_1)m \lambda$

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 ($a_g = \gamma a_{gr}$) 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 Pomhalino), we will have $a_{gr} = 1.7$ ms$^{-2}$. It was considered the importance factor corresponding to current buildings, importance II $\rightarrow \gamma_I = 1$. Thus, $a_g = 1.7$ ms$^{-2}$.

The calculation spectrum $S_d (T_1)$, 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:

$$S_d (T) = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_c}{T} \right]$$  \hspace{1cm} (EC8 3.2.2.5)

Where:
- $a_g$ – calculation value of the acceleration at the ground surface ($a_g = 1.7$ 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

$$T = C_t \cdot H^2$$  \hspace{1cm} (EC8 4.3.3.2.2)

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

2.3. 'Frontal panel’ geometric characterization

In this study, the frontal solution is adopted, consisting of a wooden structure, with panels of 1.0 x 1.0 m$^2$ locked by diagonals. This solution is usually presented in the characterization of the “frontal pombaline” [3]. For the overall dimensions of the ‘frontal wall’, a framework with 12 x 6 m$^2$ was admitted, characterizing a building of 4 floors including the ground floor. This structure consists of plumbs, beams, “travessanhos” and diagonals, as shown in the schematic drawing presented and which is considered to be repeated four times, one by each flat, in the analyzed model.

![Figure 1: Frontal panel schematic module](image-url)
In this paper we compare the solution of frontal wall structure without mortar infill and the solution of the framework braced by filling the panels with mortar infill. The first figure A reproduces the frontal model adopted in calculations. 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 infill of the orthogonal mesh of the lattice structures, consisting of diagonal locking posts and crossbars in each of the panels. In first model, there are no mortar infill (fig. B), the sections of all bars being identical to that of the standard structure of the ‘Pombalino building’. In the other model (fig C), identical spacing of 1.0 meter were used between uprights and crosspieces but the panels are filled with mortar. This last model corresponds to the standard structure of the ‘Pombalino building’, according to the ones made in several works of ‘Baixa Pombalina’ [3], [6].

2.4. 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:

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

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

2.4.1. Frontal wall wood structure \((c = 6m; h = 3m)\)

It was considered that the frontal wall structures in analysis consisted of profiles of pine wood totaling 1 m\(^3\) per floor.

\[ W_{fw} = 1 \text{ m}^3 \times 6 \text{kNm}^3 = 6 \text{kN} \]

2.4.2. Frontal wall mortar infill

It was considered that the frontal wall with mortar infill in analysis consists of triangular masonry panels with 0.15 m thick that fills the wooden structure panels of the frontal wall.

\[ W_{fm} = (6m \times 3m \times 0.15m - 1m^3) \times 20 \text{kNm}^{-3} = 34 \text{kN} \]

2.4.3. Flooring

Assuming an area of influence of 6 x 5m\(^2\) 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:

\[ W_{pav \, g+Q} = 6 \times 5 \times (0.5 + 3) = 105 \text{kN} \]

So, considering a), b) and c), the active vertical forces, the basal force \((F_b)\) is obtained.

\[ \Sigma W_i = 6kN + 34kN + 105kN = 145kN \]

\[ \therefore F_b = 4 \times 145kN = 580kN \]

From chapter 3.2 we obtain spectrum calculus \(S_d(T)\)

\[ S_d(T) = a_g \cdot S \cdot \frac{2.5}{\Omega} \cdot \left[ \frac{T_c}{T} \right] = 1.7 \times 1.8 \times \frac{2.5}{2.5} \times \left[ \frac{0.3}{0.32} \right] \approx 2.8 \]

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

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

\[ F_i = F_b \times \frac{h_i \times m_i}{\sum h_i \times m_i} \]

For the two structural models considered the results of the inertia forces \(F_i\) in the different floors are indicated in the following tables.

The forces of inertia

| Floor | \(h_i\) (m) | \(W_i\) (kN) | \(W_i \times h_i\) (kNm) | \(F_i\) (kN) |
|-------|-------------|-------------|-----------------------|-------------|
| 4     | 12          | 111         | 1332                  | 42.6        |
| 3     | 9           | 111         | 999                   | 32          |
| 2     | 6           | 111         | 666                   | 21.3        |
| 1     | 3           | 111         | 333                   | 10.86       |
| R/C   | 0           | -           | -                     | -           |
| Σ     | -           | 444         | 3830                  | 106.6       |
- Frontal wall wood structure without brickwork infill:
  The cutting force in the base $F_b$
  $$F_b = 0.24F_g = 0.24 \times 444 = 106.56 \text{kN}$$

- Frontal wall wood structure with brickwork infill:
  The cutting force in the base $F_b$
  $$F_b = 0.24F_g = 0.24 \times 580 = 139 \text{kN}$$

The forces of inertia

| Floor | $h_i$ (m) | $W_i$ (kN) | $W_i \times h_i$ (kNm) | $F_i$ (kN) |
|-------|-----------|------------|------------------------|------------|
| 4     | 12        | 145        | 1740                   | 55.6       |
| 3     | 9         | 145        | 1305                   | 41.7       |
| 2     | 6         | 145        | 870                    | 27.8       |
| 1     | 3         | 145        | 435                    | 13.9       |
| R/C   | 0         | -          | -                      | -          |
| $\Sigma$ |        | 580        | 4350                   | 139       |

2.5. Characteristics of materials

We consider for wood (forces considered // to fibers) the acceptable values:
- $\sigma_{\text{CMax}} = 25 \text{MPa}$; $\sigma_{\text{TMax}} = 25 \text{MPa}$; $\nu = 0.4$; $\gamma = 6 \text{kNm}^{-3}$; $E = 10 \text{GPa}$

For mortar infill the following characteristics were adopted:
- $\sigma_{\text{CMax}} = 5 \text{MPa}$; $\sigma_{\text{TMax}} = 0.5 \text{MPa}$; $\nu = 0.2$; $\gamma = 20 \text{kNm}^{-3}$; $E = 10 \text{GPa}$ ; $\alpha_{\text{attrito}} = 11^\circ$

3. Results

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

Based on the results obtained in the structural analysis of the two 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 $12 \times 6 \text{ m}^2$ with diagonals and reticulated mesh of $1.0 \times 1.0 \text{ m}^2$. In this model, maximum lateral and vertical elastic displacements are:
  $$U_x = 0.009 \text{ m}, U_z = -0.006 \text{ m};$$

- Frontal wall, with section $12 \times 6 \text{ m}^2$ with diagonals and reticulated mesh of $1.0 \times 1.0 \text{ m}^2$, considering panels with mortar infill. Masonry shear walls are modeled with appropriate material properties, previously described in 3.5, considering stiffness modifiers. The stiffness modifiers of membranes are used to simulate the behavior of cracked stage of mortar infill in the wood frame caused by seismic action. The stiffness adopted in these structural elements, to reflect a reduction in strength of the material, are reduced by membrane modifiers $f_{11}$, $f_{22}$ and $f_{12} = 0.25$ [6]. So, with this model, maximum lateral and vertical elastic displacements are reduced to:
  $$U_x = 0.006 \text{ m}, U_z = -0.003 \text{ m};$$

With regard to maximum lateral displacements at the top of the frontal wall, the elastic analysis shows a reduction of $U_x$ and $U_y$ in the order of $1/3$ and $1/2$ respectively.

3.2. Efforts and tensions in frontal frame elements
In the case of the bars with bigger efforts, considered in tables 1 and 2, it can be verified that the inclusion of mortar infill in frontal wall ensures significant reductions in the tensions of the constituent elements of the wood structural elements, as shown in table 3.

### Table 1. Calculations without mortar infill

| Elements  | Section (cm²) | Bar | \(N_{\text{max}}\) (kN) | \(V_{\text{max}}\) (kN) | \(M_{\text{max}}\) (kNm) | \(\sigma\) (MPa) | \(\tau\) (MPa) |
|-----------|---------------|-----|-------------------------|-------------------------|-------------------------|----------------|----------------|
| Beam      | 17x14         | 596 | 59,27                   | 11,4                    | -2,01                   | 5,5            | 0,72           |
| Plumb     | 14x10         | 627 | -138,8                  | 0,66                    | -0,41                   | -9,86          | 0,07           |
| «Travessanho» | 10x10     | 675 | 43,21                   | 0,21                    | -0,11                   | 4,97           | 0,03           |
| Diagonal  | 10X8          | 821 | -101,03                 | 0,87                    | -0,22                   | -14,24         | 0,36           |

### Table 2. Calculations with mortar infill considering stiffness modifiers

| Elements  | Section (cm²) | Bar | \(N_{\text{max}}\) (kN) | \(V_{\text{max}}\) (kN) | \(M_{\text{max}}\) (kNm) | \(\sigma\) (MPa) | \(\tau\) (MPa) |
|-----------|---------------|-----|-------------------------|-------------------------|-------------------------|----------------|----------------|
| Beam      | 17x14         | 596 | 21,07                   | 9,85                    | -1,75                   | 3,21           | 0,62           |
| Plumb     | 14x10         | 627 | -70,24                  | -0,41                   | -0,24                   | -5,75          | 0,05           |
| «Travessanho» | 10x10     | 675 | 13,69                   | 0,08                    | -0,04                   | 1,61           | 0,01           |
| Diagonal  | 10X8          | 821 | -69,01                  | 0,29                    | 0,10                    | -9,38          | 0,05           |

### Table 3. Reductions of the tensions in wood bars

| Elements        | \((1 - \sigma_2/\sigma_1) \times 100\) | \((1 - \tau_2/\tau_1) \times 100\) |
|-----------------|----------------------------------------|------------------------------------|
| Beam            | 42%                                    | 14%                                |
| Plumb           | 42%                                    | 29%                                |
| «Travessanho»   | 68%                                    | 67%                                |
| Diagonal        | 34%                                    | 86%                                |

### 4. Conclusion

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.

Through the linear elastic analysis of the "frontal panel", when subjected to the seismic action, characterized by EC8, it is possible to observe the exceptional relevance that the filling of the panels...
of the wood structure with masonry assumes in the reduction of stresses and maximum displacements in the constituent elements and in the overall structural system. These advantages are also evident even when considering a reduced stiffness of masonry elements to take into account the reduction of the resistance of this material when subjected to seismic actions. In the present study, the value of 0.25 is adopted. According to various codes, stiffness modifiers used to simulate the behavior of structure members in cracked stage should range between 0.25 and 0.30. Considering the infill, with regard to maximum lateral displacements at the top of the frontal wall, the elastic analysis shows a reduction of $U_x$ and $U_y$ in the order of $1/3$ and $1/2$ respectively. In the case of the bars with bigger efforts, it can be verified that the inclusion of mortar infill in frontal wall ensures significant reductions in the tensions of the constituent elements of the wood structural elements, as shown in table 3.

Equally and no less important is the conferred protection obtained by mortar infill with regard to fire resistance. In this context it is important to take into account that the construction system under analysis was developed to respond to the destruction that occurred in the city of Lisbon as a result of the earthquake of 1755 and the subsequent fires that broke out in the meantime.

5. References

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