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Characterization and Classification of Lisbon Old Masonry Buildings

A. Simões; R. Bento

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1. INTRODUCTION

Modern societies have been emphasizing the need of preserving the built heritage. Actually, the present policy is not only to preserve but also to make historic centres alive and attractive to its inhabitants and tourists (ICOMOS, 2003). 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.

The seismic activity in Portugal Mainland and Islands is marked by several earthquake episodes, many of them with a strong destructive potential. The 1755 Earthquake is considered by many authors as one of the largest earthquakes in history (SPES, nd) and is one of the references for the design seismic action in Portugal Mainland. Despite the present reduced activity, the seismic events are recurrent and, what happened in the past will certainly happen again in the future.

The seismic risk regards the probabilistic combination between the hazard, the exposure and the vulnerability (Sousa *et al*., 2010). The seismic hazard represents the propensity of the region for the occurrence of earthquakes, which is widely recognized as being high in Portugal Mainland (growing from the north to the south) and Azores Island. The exposure refers to the people and goods exposed to the event, depending therefore, on the population density and on the economic development of the region. The vulnerability can be defined as the capacity of buildings and structures to withstand the earthquake with minimal damage, in addition to the action of the population during and after the earthquake (Lopes *et al*., 2008).

The experience acquired to date leads to the conclusion that the difficulties and uncertainties related to seismic vulnerability appear to be, strangely, more important or severe, then those related to seismic hazard (Sousa *et al*., 2010). This justifies the urgent need of developing and implementing effective risk reduction strategies.

The existing building stock in Portugal results from more than eight centuries of history expansion. Excluding monumental buildings, such as churches, palaces and convents, residential buildings built making use of the available materials and traditional techniques of construction that changed very little through time primarily compose the building area. 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 possible to identify specific periods that pose important developments to the construction processes, beginning, for instance, with 1) the 1755 Earthquake and the after reconstruction of the city, 2) the massive reinforced concrete framing construction after 1940s and 3) the development of the modern design codes for buildings after 1960s.

Lisbon old masonry buildings were built long before the anti-seismic codes and earlier than the 1940s. 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, a short-term structural solution which precede the reinforced concrete buildings (Table 1).

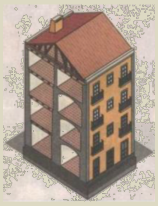
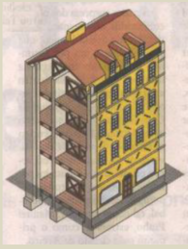
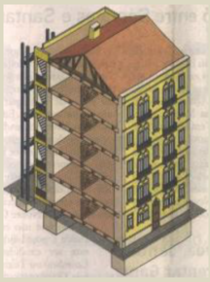
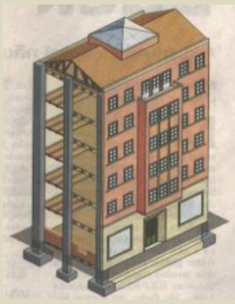
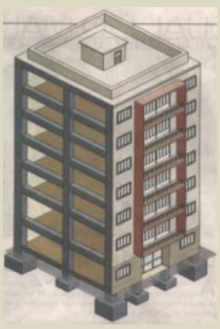
The assessment of the seismic vulnerability of old masonry buildings aims to identify the need, the type and the extent of a structural reinforcement. The goal is to gather the available information related with the period of construction, the materials of construction and structural system and the general state of conservation, for the qualitative assessment of their seismic behaviour. Nevertheless, the definition of the current structural capacity of old masonry buildings raises several difficulties.

An important number of old buildings have been subjected to several adaptations and structural modifications which may have compromised their overall performance. On the other hand, the physical and mechanical characterization of the structural materials (masonry and wood, mainly) is a difficult task considering the heterogeneity and stage of deterioration.

Assuming that the average expected life for a building is fifty years, old masonry buildings already fulfil the function for which they were designed. Given the cultural and economic value attached to the historic structures, interventions should be tailored to suit aesthetic and structural requirements of each building type, and provide sufficient reliability of performance in future earthquakes (Appleton, 2003).

This research work is developed in the scope of the FCT project entitled 'Seismic vulnerability analysis of old masonry buildings'. This document focuses in the Task 1 of the project (Characterization and classification of Lisbon old buildings) and is organized in two main sections. The first section - Construction Typologies - supports the global description of old masonry buildings. The second section - Vulnerability of Old Masonry Buildings – develops the characterization of the typical structural alterations and deteriorations and the potential consequences of the seismic action over the buildings. This section provides an initial idea to what might be the result of Task 3 of the project (Development of the capacity curves for each representative construction).

Table 1 - Evolution of masonry building typologies in Lisbon (adapted from Lopes *et al*., 2008 and Branco *et al*., 2011).

Before 1755	**Old masonry buildings**		Bearing masonry walls and timber floors
After 1755	**'Pombalino' buildings**		Reconstruction after 1755 Earthquake Bearing masonry walls and timber floors Interior wood structure with locking properties
1880-1940	**'Gaioleiro' buildings**		Higher buildings with bearing masonry walls and timber floors Back balconies made of steel beams and ceramic bricks Interior wood structure with weaker structural conception
1940-1960	**'Placa' buildings**		Bearing masonry walls and reinforced concrete slabs Reinforced framing elements start to appear on the ground floor
Before 1960	**First Period of Reinforced Concrete buildings**		Before the seismic codes ('Regulamento de Segurança das Construções contra os Sismos[1] (RSCCS)' published in 1958 and 'Regulamento de Solicitações em Edifícios e Pontes[2] (RSEP)' published in 1961)
Before 1985	**Second Period of Reinforced Concrete buildings**		Before the modern seismic codes ('Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes[3] (RSA)' published in 1983)
Nowadays	**Reinforced and Prestressed Concrete buildings**		

[1] In English: Safety Constructions against Earthquakes Regulation
[2] In English: Actions in Buildings and Bridges Regulation
[3] In English: Safety and Actions for Building and Bridges Structures

2. CONSTRUCTION TYPOLOGIES

2.1. OLD MASONRY BUILDINGS

Old masonry buildings were mainly composed by thick masonry walls and timber floors and roof, making use of techniques of construction supported on the traditional experience. "*The proportion of the structural elements have been refined by the perception of the structural behaviour in a trial and error process taking into consideration the static actions on the building*" (Logormasino, 2007), which were essentially reduced to the gravitational loads related with the weight of the materials.

In Lisbon County there is a range of geological formations, diverging from very soft soils, such as the alluvial formation, to soils with a high resistance, such as carbonated or calcareous-sandstone formations. Figure 1 gives a comparative look between the type of soil and the dominant location of each type of building in the city.

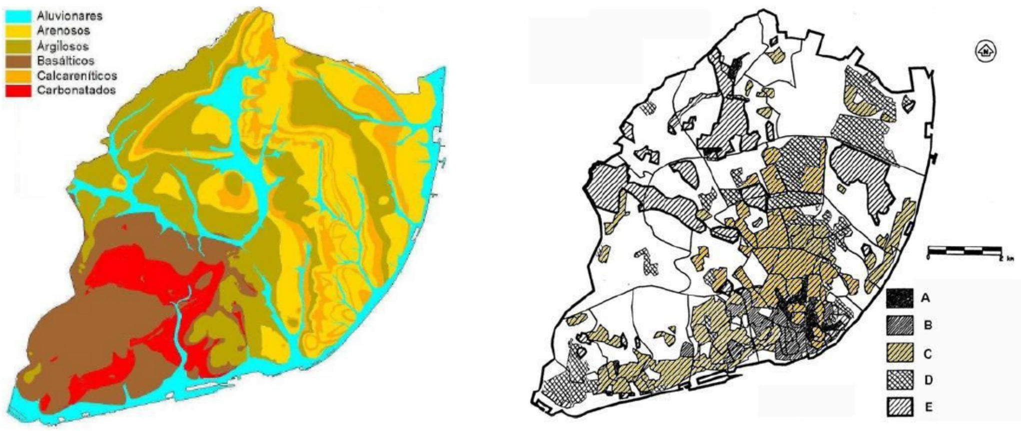
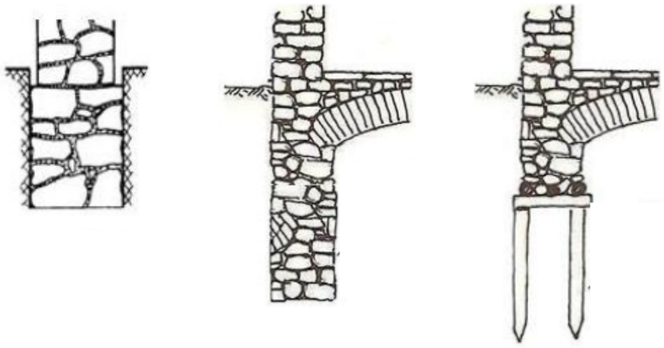


Figure 1 – Soil type in Lisbon County (CML, 2010) and dominant location of each type of building (Mendes-Victor *et al.*, 1993): A – Masonry Buildings (<1755); B – 'Pombalino' buildings (1755-1880); C – 'Gaioleiro' buildings (1880-1940); D – 'Placa' buildings (1940-1960); E – Reinforced Concrete buildings (>1960).

The foundation structure of old masonry buildings can be separated in three types: (i) direct foundation composed by the extension of the main masonry walls with the same thickness or slightly thicker (Figure 2.a); (ii) semi-direct foundation composed by rubble or brick masonry arches supported on masonry wells (Figure 2.b); and (iii) semi-direct foundation composed by timber piles crossing soft soils and reaching more resistant foundations (Figure 2.c). The opening of basements was also common allowing the construction of the foundation system at a lower level.

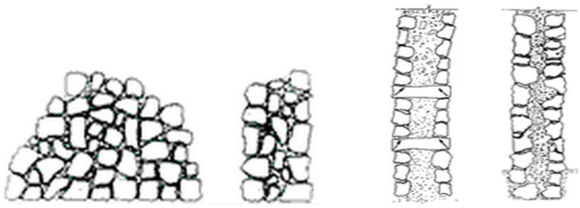


a) b) c)

Figure 2 – Foundation system (Carvalho, 2008): Direct foundation; b) Semi-direct foundation; c) Semi-direct foundation reinforced by timber piles.

Old masonry walls were generally composed by rubble limestone masonry with grit (red aggregate) or fragments of ceramic bricks bounded by mortar (Figure 3) or composed by a multi-leaf structure made of larger stones on the exterior and a rubble core (Figure 4). On the better quality constructions, the ground façade walls were made of regular limestone blocks, which were also used to reinforce the corner of the buildings along the height (Figure 5).

Figure 3 - Rubble stone masonry walls.



a) b)

Figure 4 – Composition of old masonry walls (Jones, 2002): a) Rubble stone masonry; b) Multi-leaf walls connected on the transverse direction by larger stones, iron ties or timber joists.

Figure 5 – Limestone blocks on the ground floor façade wall and on the corner of a 'pombalino' building.

The irregularity of masonry stones (composition and shape), the quality of the mortar and the workmanship significantly affect the structural performance of the construction. The bearing capacity of masonry walls is mostly guaranteed by the internal compressive stresses and by the friction generated between the elements (cohesion). As a result, old masonry walls have a low capacity to withstand shear and tensile stresses. The mortar was usually made with good quality sand, from the Leiria pine forest, and air lime in a ratio of approximately one part to the double (1:2, 2:5, 5:9) or even stronger (3:5) (Appleton, 2005).

These walls have a decreasing thickness from bottom to top, regarding the reduction of the loads on the lower levels, the rationalization of the materials and it also allowed a better support for the floor structure (Figure 6). Nevertheless, the construction of the exterior masonry walls suffered few variations over the centuries which lead to the progressive reduction of its thickness.

Figure 6 – Reduction of the wall thickness along the height of the building (Appleton, 2003).

Iron ties were commonly used to strength the connection between perpendicular masonry walls and parallel masonry walls at the floor level (Figure 7). The tie rods worked as passive reinforcements, only mobilized after changes on the structure balance, for example, connected with thermal variations, foundation settlements or horizontal forces due to the action of earthquakes.

Figure 7 – Iron ties in the connection between orthogonal masonry walls and between parallel masonry walls at the floors level (Lopes *et al.*, 2008).

Horizontal limestone lintels were often used to strengthen the openings on the walls (Figure 8.a); however, these masonry elements were limited by their natural size. The use of relieving arches allowed larger openings as the loads were transferred for the sides through compression stresses. These arches were made of regular masonry stones (Figure 8.b) or ceramic bricks (Figure 8.c).

a) b) c)

Figure 8 – Openings on masonry walls reinforced by (Appleton, 2003): a) Limestone lintel; b) Regular masonry stone relieving arch; b) Ceramic brick relieving arch.

According with the period of construction the solution for the interior structure diverges. The interior walls were placed on both directions of the building, providing support to the exterior masonry walls along the height of the building and the connection between the floors and roof structure (Appleton, 2003).

Reminding the conventional expressions, the ‘frontal’ walls were generally parallel to the main façade wall regarding the support of the floor beams, while the ‘tabique’ walls were considered for compartments division without other structural function.

On the older masonry buildings, ‘frontal’ walls were composed by an irregular timber structure filled by rubble masonry. With the construction of ‘pombalino’ buildings the conception of ‘frontal’ walls improved to a uniform and repetitive system.

Composed by vertical, horizontal and diagonal joists, these structural elements (frontal walls) were connected by assemblies and nailing fixation and filled afterwards by rubble masonry (Figure 9). By the end of the nineteenth century, the timber structure started to be simplified and the rubble masonry infill replaced by industrial masonry bricks (Figure 10).

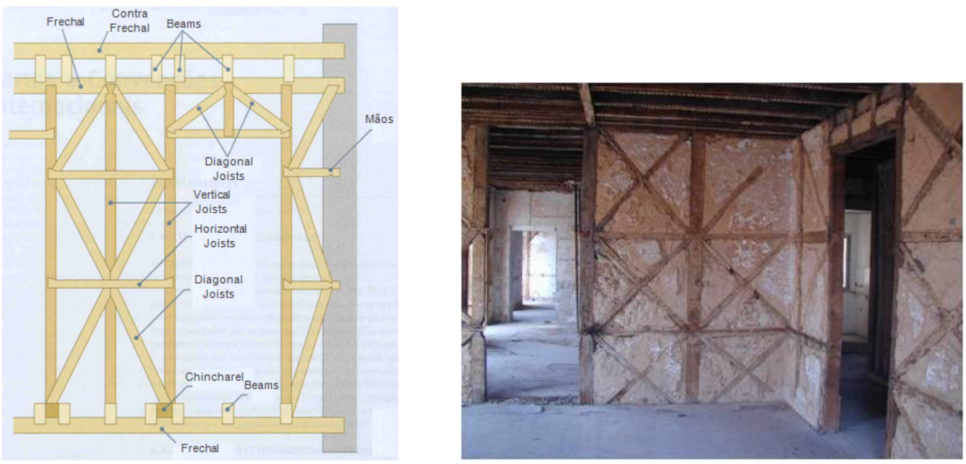


Figure 9 - ´Frontal’ walls characteristic of a ‘pombalino’ building (Appleton, 2003).

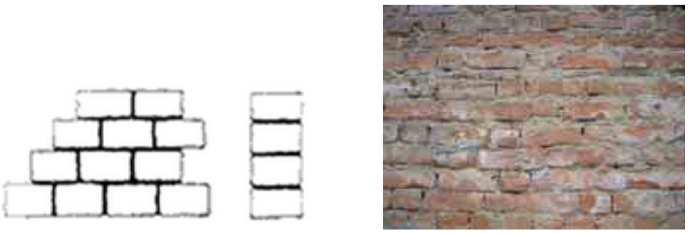


Figure 10 – Brick masonry walls characteristic of a ‘gaioleiro’ building (Jones, 2002 and Carvalho, 2008).

The most traditional solution for the interior division is made of timber laths nailed to vertical joists, filled afterwards by rubble masonry and mortar (Figure 11). These walls have a much deformable and light structure.

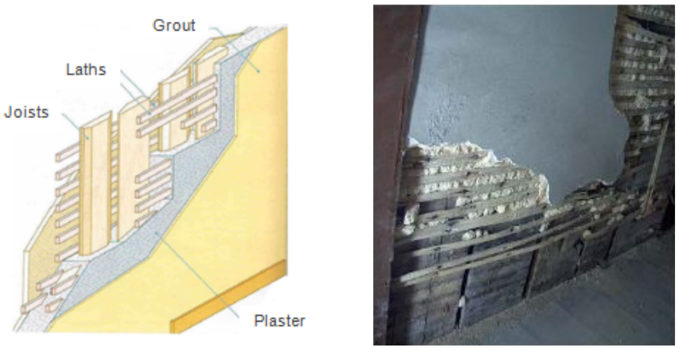
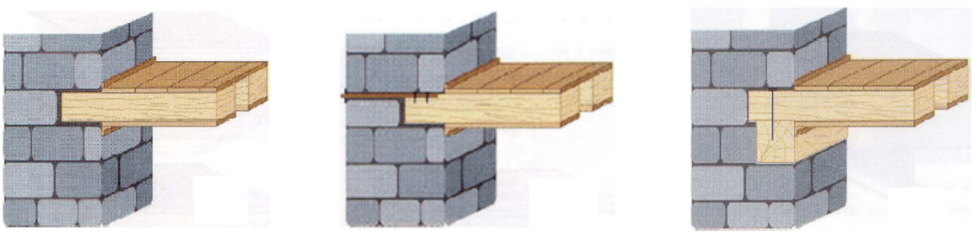


Figure 11 – ‘Tabique’ walls (Appleton, 2003 and Pena, 2008).

The construction of masonry arches and vaults on the ground floor was applied to the noblest buildings. This solution allowed larger spans for the commercial activity or warehouses and prevented the contact of the interior timber structures with rising humidity. The ground floors were based on a rock fill over which stands a floor coating made of stone.

The floors from the upper storeys were composed by wooden beams, usually disposed perpendicular to the façade walls, braced by smaller joists (named ‘tarugos’) preventing the transverse deformation of the beams. The floors were supported on the interior walls and embedded on the masonry walls (Figure 12). The height of the beams was usually less than 0.20 meters (m), restraining the spans of the rooms to a maximum of 4 m (Appleton, 2003).



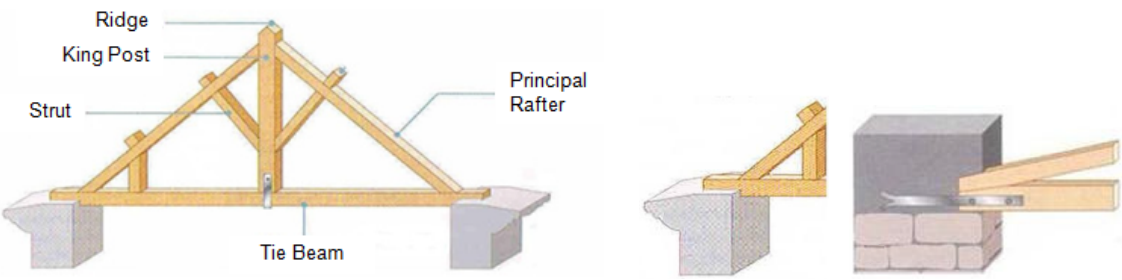
a) b) c)

Figure 12 – Connection between the floors and the masonry walls (Appleton, 2003): a) Beams simply embedded on the wall; b) Beams embedded and anchored to the wall by steel nails; c) Introduction of a regular stone or a timber beams on the wall to support the beams.

The buildings were then covered by pitched roofs, with a timber frame structure and ceramic tile coating. Depending on the location of the building and the type of use of the attic, the roofs include dormer or mansard windows (Figure 13).

Figure 13 – Dormer and mansard windows.

The roof structure is composed by a set of parallel trusses connected by purlins (main beams), common rafters and slats that support the roof tiles. On dual pitched roofs, purlins were usually parallel with the front façade walls and supported on the side walls of the building. The loads from the roof were transferred to the walls by the direct support (Figure 14.b) or through transition beams for a better distribution of the loads (Figure 14.c).



a) b) c)

Figure 14 – Timber roof structure (Appleton, 2003): a) Truss structure; b) and c) Example of connections to the masonry walls: b) Direct Support; c) Steel Reinforcement.

There is also an evolution on the staircase structure (Figure 15). On older buildings, the stairs only had one flight between floors and were located along the side walls. During the eighteenth century, the increasing ceiling height of the floors and the need of comfortable accessibilities brought the use of two-flight stairs separated by a half-landing or even three-flight stairs. The stairs were also moved to the centre supporting larger buildings with two houses per floor.

Stone masonry stairs were used on the ground floor level preventing humidity problems to the upper timber structure of the buildings. At the end of the nineteenth century, steel staircases were used on the back façade of the buildings for fire prevention. These structures were made by circular columns, I or T profile beams and grid steel plates on the stairs.

a) b) c)

Figure 15 – Different types of stairs (Andrade, 2011): a) Interior timber staircase; b) Ground floor masonry staircase; c) Steel staircase from a 'gaioleiro' building.

2.2. BUILDINGS BUILT BEFORE 1755

2.2.1. HISTORIC SURVEY

The buildings which remain after the 1755 Earthquake belong to a very heterogeneous group. Actually, is not possible to define a specific typology of buildings, as they emerged from several centuries of history without a proper urban planning.

Lisbon was founded in an indeterminate period. Originally known as Olissipo, the first populations were installed on the slope of the Castelo de São Jorge hill. In the second century (b.C.), the region was integrated in the Roman Empire and after, in fifth century (a.C.), the Muslim government began and lasted more than four hundred years. In 1147, Lisbon was conquered by D. Afonso Henriques and in 1225 was named the capital of the Portuguese Kingdom.

The city was never known for its magnitude, but it had a considerable importance because of its geographical location and port traffic. On the second half of the twelfth century, Lisbon had more than six thousand inhabitants and continued to growth outside the Moorish fence, initially to the northeast side, next to the Saint Vicente de Fora Monastery and to the south with Alfama district. By the end of fourteenth century, a new fence wall was completed by the name of King Fernando.

With the advent of maritime discoveries during the fifteenth century, Lisbon became an important trade centre enriched by the marketing products from the East. Inside the wall fence, Lisbon had the characteristics of a medieval village. The streets were interconnected in intricate outline with a high number of alleys and narrow streets. The construction of the Palace of Ribeira (1498-1503) by King Manuel, changed the political and economic centre of the city next to Tagus River (what is now Comércio Square). By the end of the sixteenth century, the city continued to growth outside the fence with the urbanization of Bairro Alto, in an orthogonal pattern of streets that continuous along the hillside to the river (what is now Cais do Sodré).

During Filipe's Spanish government (1580-1640), the urban expansion was insignificant. With the recovery of the national independence in 1640, the country experienced a period of great austerity imposed by various political and economic constrains. This situation had a strong influence on architecture, characterized by straight and sober lines.

The first half of the eighteenth century stands out for the resources that came from Brazil, supporting the economic growth and the construction of several historic buildings along the river to the west, between Ribeira de Alcântara and Santa Apolónia, and to the north with Prazeres, Santa Isabel, Rato, Santa Marta, Campo Santana, Graça and Senhora do Monte neighbourhoods (Figure 16).

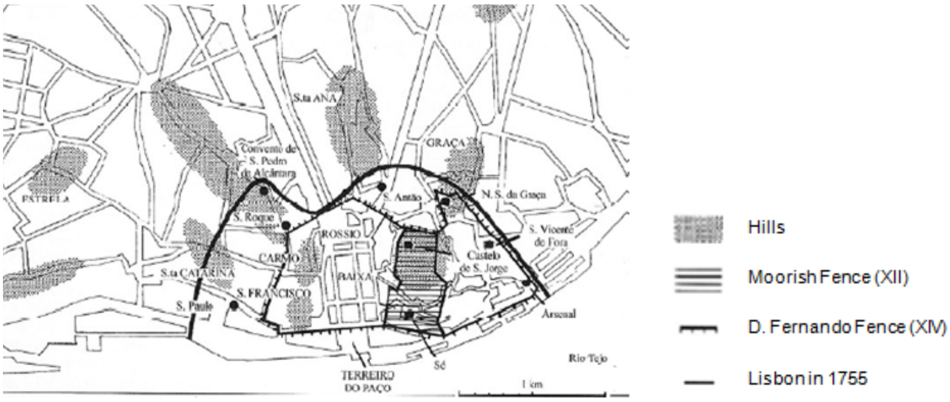


Figure 16 - Limits of Lisbon County before the 1755 Earthquake (adapted from Fernandes and Janeiro, 1991).

The knowledge of the ancient city before the 1755 Earthquake is mostly based on the interpretation of written and draw documents, since most of the buildings and streets no longer exist. Apparently, the urban area has not changed since the sixteenth century, mostly concentrated between Terreiro do Paço (what is now Comércio Square) and Rossio Square (Figure 17).

Figure 17 – General Plan of Lisbon on the seventeenth century, originally designed by Nunes Tinoco in 1650 (known has one of the oldest plans of the city – Museu da Cidade, 2007).

2.2.2. DESCRIPTION OF THE BUILDINGS

The buildings built before 1755 have to be independently assessed in its historical and geographical characteristics. According to Oliveira *et al*. (1985) and Santos *et al*. (1993) these buildings might be divided in two main groups: (i) noble and palatial buildings, with three to four floors made of planed masonry walls, at least on the building corners and other locking elements; and (ii) cramped masonry buildings weakly built, without planed locking elements, reduced ceiling height and few openings to the exterior.

At Castelo de São Jorge hill (Alfama, Mouraria and Castelo districts) and Bairro Alto, it is still possible to find original architectonic features of the second group of buildings. For instance, older buildings usually have a narrow front façade and pitched roofs to the side (Figure 18). Access to the upper floors is made next to the side masonry walls through one flight stairs with less than one meter width. These stairs are much leaned and uncomfortable (the risers could have 0.20 m), though only possible regarding the reduced ceiling height of the floors.

a) b)

Figure 18 – Reconstruction Model of the city in 1755[4] (Museu da Cidade, 2010): a) Nova dos Ferros Street; b) Rossio Square.

The perimeter walls were made of rubble masonry with few openings to the exterior. The interior walls were mainly composed by 'tabique' walls made of timber laths nailed to vertical joists, filled by rubble masonry and air lime mortar. Their structural role has to be individually assed, regarding the conservation of the materials and the connection between elements. There are some cases of interior walls made of a truss timber structure filled by rubble masonry. Despite the irregularity of the truss, it might have been the beginning of the 'frontal' walls characteristic of the 'pombalino' buildings (Figure 19).

The floors and roof structure were made of rounded joists and reused wood elements, explaining the diversity of the solutions. Some buildings have a rebounded ground floor, as the upper floors standout on the front façade wall increasing the interior space. These floors were supported by timber beams propped against the masonry wall working in cantilever (Figure 20). In order to reduce the loads involved, the exterior walls have a composition similar to the 'frontal' walls (Figure 19.b).

[4] Museu da Cidade developed a computational model of Lisbon in 1755 based on a large model produced between 1955 and 1959 by Ticiano Violante for the exhibition "Reconstruction of the city after the 1755 Earthquake" (Palácio das Galveias, 1955).

a) b)

Figure 19 – ‘Frontal’ wall (Lopes *et al*., 2008): a) Typical solution built before 1755; b) Solution from the eighteenth century distinguished by the regularity of the timber elements, although two diagonal joists are missing. In fact, figures a) and b) belong to the same building.

Figure 20 – Buildings from the XVI century with floor ledge and an interior view of the floor with the density of timber beams (Museu da Cidade, 2010 and Lopes *et al*., 2008).

Considering the time of construction and the multiple structural modifications suffered, these buildings are probably the ones that pose a higher seismic vulnerability between the typologies of old masonry buildings. Conversely, the rareness in number of the buildings built before 1755 and the specifics of its structure requires a case to case structural assessment.

The main difficulty is to clearly understand their structural behaviour, especially the type of connections between the elements. It is also complex the evaluation of the structural interaction between the buildings of different ages and typologies that coexist inside the compounds.

2.3. ‘POMBALINO’ BUILDINGS

2.3.1. HISTORIC SURVEY

Portugal was hit by a severe earthquake[5] followed by a tsunami that caused great damage in Lisbon on November 1st, 1755 (Figure 21). Consequence of the earthquake and destruction, several fires spread through the city lasting for days. This catastrophe was considered as one of the most deadly in history, leaving the city of Lisbon almost completely destroyed. It was estimated that about 75% of the buildings collapsed or became uninhabitable (LNEC, 2005).

a) b)

Figure 21 - Lisbon after the 1755 Earthquake: a) On the right the ruins from Patriarcal Church (what is now the City Hall) and on the left (back) the ruins from São Francisco Convent (what is now Belas Artes College) (Lopes *et al*., 2008); b) Damage caused on the Opera House or Ribeira Theatre. On the right, the score of a building façade wall (Museu da Cidade, 2007).

A complex reconstruction plan, led by Sebastião José de Carvalho e Melo after nominated as Marquês de Pombal, was immediately deliberated. The military engineer Manuel da Maia announced the first features of the program of improvements to the city taking into account the safety of the buildings, the cleanliness of the houses and streets (Museu da Cidade, 2007). The program was afterwards developed by the engineers Eugénio dos Santos Carvalho and Carlos Mardel and rigidly imposed during the whole reconstruction period.

One year was enough to design a new city centre with the purpose to withstand the effects of future earthquakes. Contrasting with the winding medieval city, the new downtown design required an absolute uniformity of the built area; placing the main streets of the city between Comércio Square (what was before Terreiro do Paço) and Rossio Square. The buildings were disposed in rectangular quarters with similar dimension and composition, followed by an orthogonal grid of streets. Figure 22 gives an overview of the ‘Pombalino’ Downtown Plan (dated of June 12th, 1758).

The main streets have 60 ‘palmos’ of width (about 13.2 m) with a 10 ‘palmos’ (2.2 m) walkway in each side (Appleton, 2008). Depending on the type of living, there were residential quarters measuring around 71 x 27 square meters (m^2) in plan and court quarters with 59 x 33 m^2 next to Comércio Square (economical centre), both with a narrow central yard (Silva, 2007).

[5] It was estimated that 1755 Earthquake had 8.5 to 9.0 magnitude in Richter scale (SPES, nd).

Figure 22 - 'Pombalino' Downtown Plan designed by Eugénio dos Santos Carvalho (adapted from Museu da Cidade, 2007).

The 'Pombalino' downtown, named after the Marquês de Pombal, border by Ouro Street and Madalena Street, is composed of approximately 53 blocks with an average of seven to eight buildings. The reconstruction plan was also extended to Alecrim Street and São Paulo Street (Chiado and Cais do Sodré districts) and São João da Praça Street on the opposite side of the downtown area (Castelo de São Jorge hill).

2.3.2. DESCRIPTION OF THE BUILDINGS

The design of the buildings arose from the inspection of the buildings which withstood the earthquake. The construction methods were based on normalization and rationalization principles, producing buildings of great uniformity in architectural and constructional terms, which allow a large scale reconstruction.

The composition of the façade walls and the height of the buildings were originally imposed by the plan. With no more than 5 stories (including the roof top), the ground level was occupied by shops and warehouses, with 3.5 to 3.70 m height, as the upper floors feature a decreasing ceiling height, which might be connected with the status of its inhabitants.

The façade walls were composed by large doors to the exterior on the ground floor; French windows with a stone balcony on the first floor; sash windows on the higher floors and dormer windows on the roof (Figure 23). Some buildings have also an extra floor between the ground floor and the residential floors, used as storage for the shops. These lower floors intended to ensure a constant high within the compound of buildings on sloped areas (Figure 24).

Despite the simplicity and repetitive of the exterior, there are a few variations on the exterior architectural elements according to the importance of the street where the buildings were located (Mascarenhas, 1996).

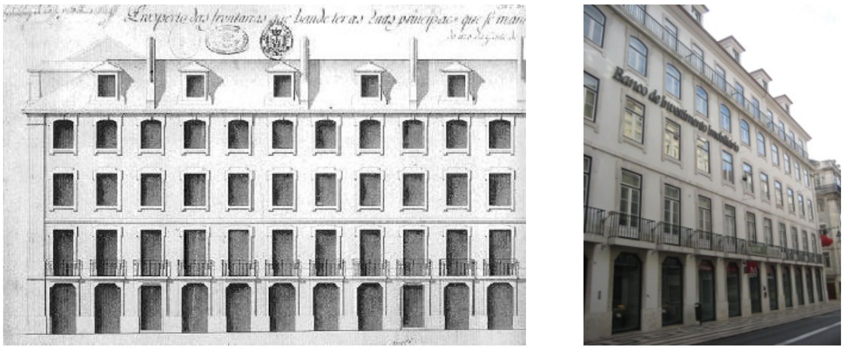


Figure 23 - The windows were aligned both vertically and horizontally in order to maintain the balance of forces (Mascarenhas, 1996).

Figure 24 – Lower floor above the ground shops.

The interior division of the buildings was mostly imposed by the structural elements, continuous on the elevation of the building. The houses were usually arranged in three alignments of rooms parallel to the façade walls (Figure 25).

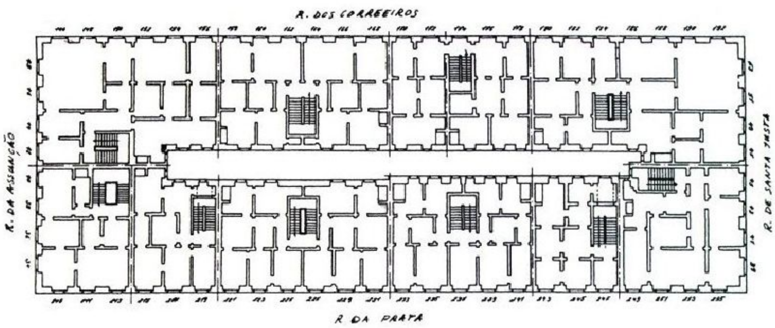


Figure 25 – Plan of the 1st floor of a 'pombalino' quarter (Mascarenhas, 2005).

The compartments next to the front façade were wider and lighter and reserved for social purpose. The back façade was meant for the kitchen and bedrooms. The inner rooms, without direct light or ventilation, were used for smaller bedrooms or circulation space. Access to the upper levels was made through two-flight stairs separated by a half-landing or even three-flight stairs. On the interior buildings of the quarter, the staircase has a central position introducing two houses in each storey. On buildings with only one house per storey, the staircases were placed next to the side walls.

2.3.3. DESCRIPTION OF THE STRUCTURE

The downtown soil is very soft and of alluvial nature, while the upper layers are composed by the rubble from the previous buildings. The 'pombalino' buildings were supported on a timber grid laid on short timber piles with no more than 0.15 m diameter and 1 to 6 m length (Figure 26), which hardly achieved the resistant soil. This support system intended to stiffen the alluvial soil and to create a working platform above the water level (Appleton, 2003). The general foundation system consisted in continuous masonry walls or caissons filled with rubble masonry covered by a group of arches (Figure 27).

Figure 26 – Timber pile from a 'pombalino' building: experience has demonstrated that these timber piles are closer to 1 m length rather than 6 m.

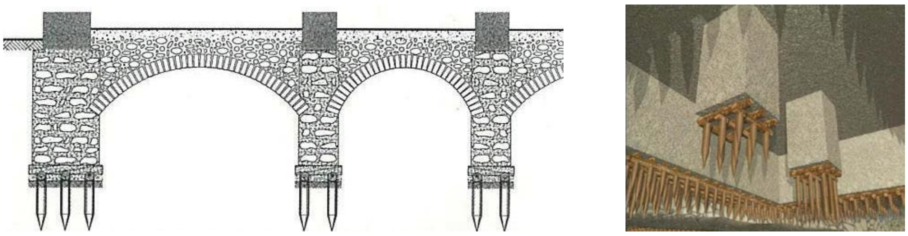
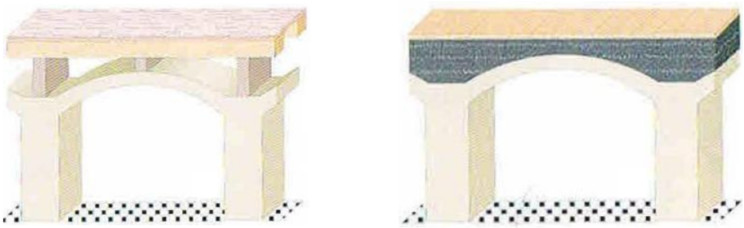


Figure 27 - Foundation system made of masonry arches and caissons supported in timber piles (Silva, 2007).

The façade walls were built in rubble stone masonry bounded by air lime and sand mortar with poor strength capacity. Typically these walls have 0.9 to 1.1 m thickness, decreasing approximately 0.05 m in each or every two floors. The façade openings were covered by relieving arches made of stone masonry or ceramic blocks. Under the windows, the façade walls were thinner, allowing an easy access to the exterior. Within the block, the buildings were built sharing the side walls. With 0.50 m to 0.75 m thickness, these masonry walls were higher than the roof tops preventing fire propagation between buildings.

The ground floor was entirely built in stone masonry, preventing the rising humidity from the ground soil or fire propagation from the warehouses. The ceiling was usually covered by barrel vaults and arches or crossed vaults made in stone or brick masonry (Figure 28). These structures were supported by thick walls or squared columns providing a large stiffness to the structure in its base. Above the masonry vaults and arches the flooring was supported on a timber frame structure (Figure 29.a) or debris infill (sand or loose soil) covered afterwards by a layer of mortar (Figure 29.b). On the better quality solutions, the ground façade walls were made of regular limestone blocks, which were also used to reinforce the corner of the buildings along the height (Figure 30).

Figure 28 – Example of arches barrel vaults and arches (Appleton, 2003) and crossed vaults.



a) b)

c) d)

Figure 29 – Representation of the floor structure: a) and c) Arch covered with timber frame structure (Appleton, 2003); b) Arch covered with rubble infill (Appleton, 2003); d) Cut over an arch with no infill.

Figure 30 – Ground floor façade wall made of regular limestone blocks.

On the upper floors, each building combined a three-dimensional timber structure called ‘gaiola’ (meaning cage) responsible for the bracing of the building which mostly is the answer to their good seismic behaviour (Lopes *et al.*, 2008). Vertical and horizontal elements were added to the façade walls, stiffening the masonry structure around the window openings (Figure 31). The interior structure was composed by timber-masonry walls, timber floors and roof (Figure 32), linked to the exterior walls by timber connecters, partially embedded on the masonry (0.25 to 0.30 m) and reinforced by metal straps.

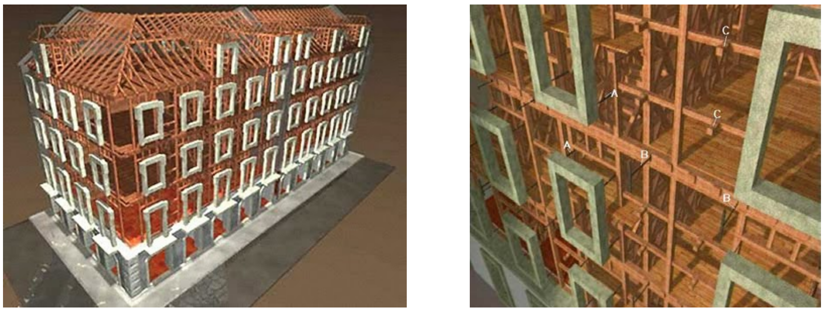


Figure 31 – Model of the timber frame on the façade walls (Silva, 2007).

a) b)

Figure 32 – ‘Pombalino’ cage (Appleton, 2008): a) Timber frame on the façade walls and relieving arch lintel above the window; and b) Interior timber-masonry walls (‘frontal’ and timber floors).

According to Farinha (1955), the ‘gaiola’ expression comes from the appearance of the buildings during the construction process, as the timber structure was built till the roof and then filled in by rubble masonry. This composite structure was then connected to the ground floor by steel ties bolted on the masonry arches and walls (LNEC, 2005).

‘Frontal’ walls were composed by vertical, horizontal and diagonal elements connected between assemblies and nailing fixation. The position of the joists is based on the empirical principle that is difficult to deform a triangle, without changing the length of its sides (Figure 33). In that case, ‘frontal’ walls, while requested by vertical and horizontal actions in the plan, respond by the axial strength of the timber elements and connections (Figure 33 and Figure 34).

The truss structure was then filled by rubble stone masonry restraining the deformation of the joist. These walls were placed in orthogonal directions sharing timber elements between, providing strength to the actions in any direction.

Figure 33 - Example of ‘frontal’ wall with (Appleton, 2008) and without masonry infill (Lopes, 2010). Despite the regularity that defines the ‘pombalino’ buildings, there is a great variability in shape and connection of the timber joists.

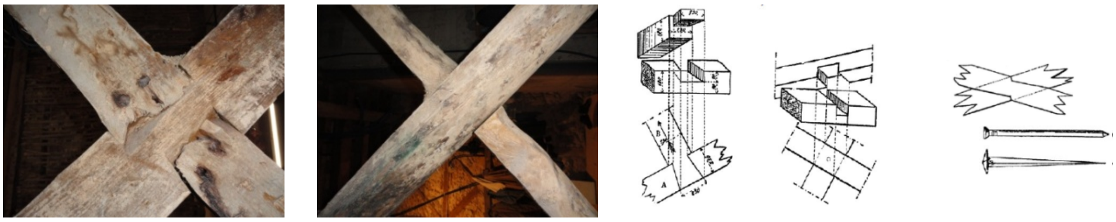


Figure 34 - Connection between diagonal elements: notch work, assembly and nailing fixation (Segurado, nd).

The ‘tabique’ walls have a lighter structure made of timber laths nailed to vertical joists, filled afterwards by masonry and air lime mortar (Figure 35). Both ‘frontal’ and ‘tabique’ walls were plastered on both sides, but can be easily distinguished by its thickness. Usually, ‘frontal’ walls were 0.15 to 0.20 m thick, while ‘tabique’ walls were slightly thinner, around 0.10 to 0.12 m, and mostly used for compartments division (Silva, 2007).

Figure 35 – Example of ‘tabique’ walls (Pena, 2008).

The floors were composed by wooden beams set perpendicularly to the façade walls and braced, on the perpendicular direction, by smaller joists (Figure 36.a). This standard defines the façade walls as the principal structural direction of the ‘pombalino’ buildings and the main orientation of the ‘frontal’ walls. The remaining interior walls, exception made to walls between houses and staircase, are made of ‘tabique’ walls.

The main beams were frequently spliced over the interior walls, though in better quality constructions those beams were continuous between perimeter walls. The floors were also connected to the timber frame disposed on the façade walls (Figure 32.a). On the corner buildings, the main beams are placed in different directions (Figure 37). The floors were after covered by soft pine boards perpendicular to the beams, while the ceilings were finished with wooden boards or wooden laths covered with plaster (Figure 36.c).

a) b) c)

Figure 36 – Floor structure: a) Main beams braced on the perpendicular direction (Appleton, 2008); b) Beams embedded on the masonry walls (Appleton, 2008); c) Ceiling finished with laths and plaster.

Figure 37 – Floor beams placed on both directions on corner building of a 'pombalino' quarter.

The staircase on the ground floor was made of masonry (like the other structural elements), while on the upper floors it was made of wood and supported on timber truss walls (Figure 38). The staircase also has an important contribution to the 'gaiola' structure, working as a resistant core throughout the building height, providing interior support to the floor beams.

The buildings were then covered by dual-pitched roofs, originally with dormer windows and after with mansard windows (Figure 39). The interior space was divided by a timber truss structure, support for the ceramic tiles and connected to the interior 'gaiola' structure.

Figure 38 - Timber staircase (Appleton, 2008 and Lopes *et al.*, 2008).



Figure 39 – Example of typical dormer windows and mansard roofs (Appleton, 2008).

2.4. 'GAIOLEIRO' BUILDINGS

2.4.1. HISTORIC SURVEY

During the first half of the nineteenth century there were few changes on the urban landscape as the city continued to grow accordingly with the 'pombalino' reconstruction plan. In 1864, a commission was nominated by the Ministry of Public Works to deal with a program of urban improvements and expansion of the city to the north upland.

The opening of Liberdade Avenue (inaugurated in 1886 - Figure 40.a) and Rainha Amélia Avenue (now called as Almirante Reis Avenue - Figure 40.b), in addition to developments on the east side, with the embankment of the seaport that originated the 24 de Julho Avenue, improved the connection of the city centre with the rural periphery (Figure 41).

In 1888, the engineer Ressano Garcia developed a new plan regarding the connection between Liberdade Avenue and Campo Grande through the opening of Picoas Avenue (what is now Fontes Pereira de Melo Avenue) and Ressano Garcia Avenue (called nowdays República Avenue - Figure 40.c).


a)


b)

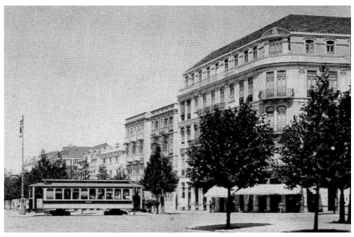

c)

Figure 40 – Contemporary pictures from the over (AFML): a) Liberdade Avenue in 1900; b) Almirante Reis Avenue in 1908; c) República Avenue.

The 'Gaioleiro' buildings are associated with the construction of Bairro de Camões occupying the hill on the east side of Santa Marta Street and 'Avenidas Novas' adjacent to Fontes Pereira de Melo Avenue and República Avenue. The buildings were aggregated in quarters with interior yards and surrounded by a grid of secondary streets, wider than the streets of the 'Pombalino' downtown.

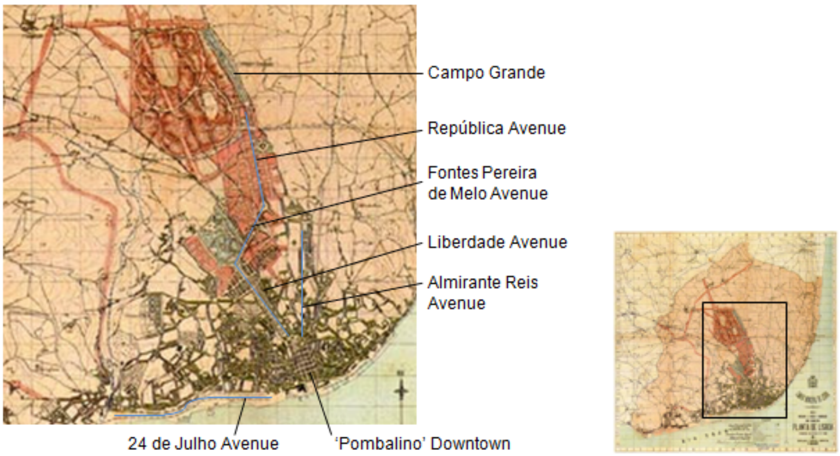


Figure 41 - Plan of Lisbon in 1903 (adapted from AFML).

2.4.2. DESCRIPTION OF THE BUILDINGS

The new areas of expansion included modest buildings intended to the middle class population (Figure 42) and singular buildings displaying the social status of their owners (Figure 43). During the nineteenth century, the memory of the 1755 Earthquake devastating effects was mostly forgotten, as well as the requirements of such a methodical construction system.

a) b) c)

Figure 42 – Rentable buildings (AFML): a) Building from 1895 in Liberdade Avenue; b) Building in Fontes Pereira de Melo Avenue; c) Building from 1913 in Liberdade Avenue.

a) b) c)

Figure 43 – Palatial buildings (AFML): a) Building from 1906 in Liberdade Avenue; b) Building in Fontes Pereira de Melo Avenue; c) Building in República Avenue.

The name ‘Gaioleiro’ (meaning cage in a depreciatory way) is related with the buildings built to be sold or to be rented by flats aiming to sustain the development of the city and the housing needs of an increasing population. The construction was carried out by private entities, and therefore the quality of the buildings is very variable.

The expansion program proposed by Ressano Garcia was very flexible comparing with the ‘pombalino’ reconstruction plan. There were no standards for buildings height or depth, neither for the architectural design of the façade walls (Figure 44). Within the quarter, there are buildings with five floors (including roof top), like the original ‘pombalino’ buildings, right next to buildings with seven floors, with generous ceiling height.

a) b) c)

Figure 44 – Comparison between the rationality of the ‘pombalino’ façade wall and the design of the ‘gaioleiro’ buildings (Appleton, 2005): a) Overview of Nova do Carvalho Street on ‘pombalino’ downtown; b) and c) Overview of 5 de Outubro Avenue on ‘Avenidas Novas’.

The front façades were often decorated with a collection of *Art Nouveau* details (flowers and organic shapes mostly), windows with laboured stone frames and with different shapes or positions within the floors (Figure 45). Three distinct levels can be identified on the front façade walls: the base masonry cover (often made of plaster instead of masonry), the middle part extensively adorned or covered by ceramic tiles, and the roof with dormer or mansard windows.

Figure 45 - Decorative details from the front façade wall.

The back façade walls are recognized by the steel balconies or galleries and service staircases to access the interior area of the block, which were actually imposed by the fire-fighters and influenced by the contemporary Iron Architecture (Figure 46).

Figure 46 – Steel service stairs and galleries on ‘gaioleiro’ buildings (Andrade, 2011).

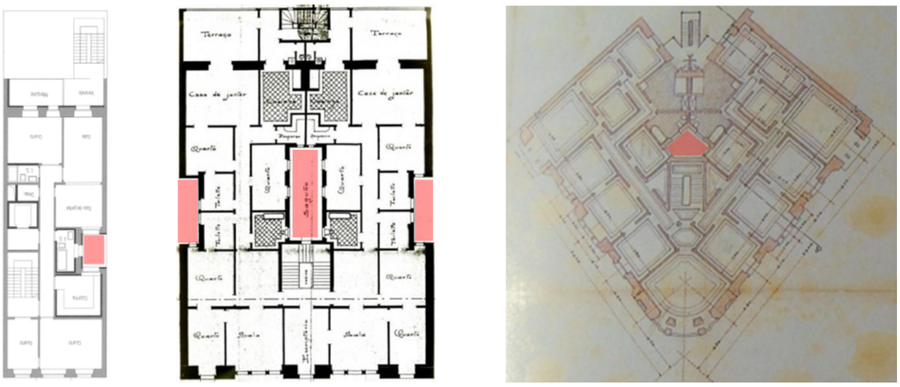
Generally, the quarters have a rectangular shape, though there are also trapezoidal shape quarters conditioned by the slope of the uptown land, originating corner buildings with irregular dimensions. The buildings are usually longer into the backyard and tighter into the façade walls, originally with two houses per storey or only one, resulting from the division of a larger fraction. According to Appleton (2005) the ‘Gaioleiro’ buildings might be divided into four different types (Figure 47 and Figure 48).

Type 1 – Small to medium size buildings with strait to medium front façade wall, lateral light-shaft, lateral stairs and one housing per floor;

Type 2 – Large size buildings, with large front façade wall, lateral light-shaft and one housing per floor;

Type 3 - Large size buildings, with large front façade wall, two lateral light-shafts and eventually one central light-shaft, central stairs and two housing per floor;

Type 4 - Large size buildings on the corner of the compound, with two or more light-shafts, central stairs and two or more housing per floor.



a) b) c)

Figure 47 – Plan of ‘gaioleiro’ buildings: a) Example of Type 1 (Andrade, 2011); b) Example of Type 3 (Branco, 2007); c) Example of Type 4 (Andrade, 2011).

Figure 48 – ‘Gaioleiro’ buildings with different size and shape.

The side walls, frequently shared by adjacent buildings, are interrupted by light-shafts, which provide natural light and ventilation to the interior rooms (Figure 49). Ventilated masonry boxes on the ground floor prevented the rising moisture from the soil and the rotten of the interior wooden structures. These elements can be identified outside by steel or masonry grids on the façade walls or on the entrance hall of the buildings with a first flight of masonry stairs that makes the connection to the interior timber staircase (Figure 50).

Figure 49 - Example of light-shafts (Appleton, 2005 and Andrade, 2011).

Figure 50 – First flight of masonry staircase gives access to an elevated ground floor with ventilated box underneath (Andrade, 2011 and Appleton, 2005).

2.4.3. DESCRIPTION OF THE STRUCTURE

The uptown soil was mostly composed by sandy-clay soils with low resistance rocks, thus the ‘Gaioleiro’ buildings were commonly supported on caissons and arches structure (foundation soil more than three meters deep) or continuous walls in limestone masonry solid grounded (Figure 51). The foundation system was usually larger below the façade walls, with 1.10 to 1.50 meters (m) thick, and thinner when below the gable and light-shaft walls, around 0.60 m to 0.70 m thick (Appleton, 2003).

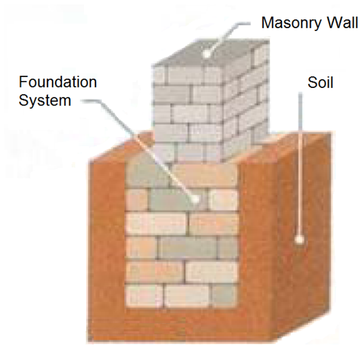


Figure 51 - Direct foundation system (Appleton, 2003).

The exterior walls were built in rubble stone masonry linked by air lime mortar and sand (Figure 52). The front façade walls are typically 0.60 to 0.80 m thick on the ground floor, with a decreasing thickness with the elevation of the building, resulting in rooftop walls with 0.30 to 0.40 m of thickness (Lopes *et al.*, 2008). The back façade walls were usually 0.50 to 0.60 m thick. Vertical and horizontal timber struts reinforced the masonry walls around door and window openings; although, experience has demonstrated that often these elements do not exist.

a) b) c)

Figure 52 – Exterior Masonry Walls: a) Rubble composition of the masonry side wall (Andrade, 2011); b) Connection between façade and side wall; c) Reduction of the thickness along the height.

By the end of the century, new industrial materials were introduced allowing different construction solutions. Therefore, the light-shaft and side walls, originally built in rubble stone masonry, started to be replaced by bricks masonry walls (Figure 53).

The wall thickness varies from 0.40 to 0.50 m in the case of the masonry walls or 0.30 to 0.15 m with brick masonry walls, but often constant in height (Appleton, 2005). Nevertheless, the light-shaft walls are usually thinner than the side walls.

a) b)

Figure 53 – Brick masonry walls (Andrade, 2011): a) Example of a light-shaft with rubble stone masonry wall (on the left) and solid brick masonry wall (on the right); b) Hollow brick masonry wall.

During the nineteenth century, the cage structure characteristic of the ‘pombalino’ buildings was progressively simplified. The diagonal elements from the ‘frontal’ walls started to be removed, conditioning the bracing of the timber structure. The rubble infill was then replaced by brick masonry, solid on the lower floors and hollow on the upper, or by ‘tabique’ walls, originally used on ‘pombalino’ buildings for compartments division (Figure 54).

Brick masonry walls were occasionally reinforced by an orthogonal grid of joist; horizontal elements at the floors level and at the middle height of the storeys and vertical elements around the door openings. The thickness of the brick masonry walls decreases along the height of the building by changing the position of the bricks (Figure 55). Nevertheless, this variation is also related with the transition between solid masonry bricks on the lower floors and hollow bricks on the upper floors or the replacement of the brick masonry walls by ‘tabique’ walls.

The ‘tabique’ walls were made of vertical timber strips with approximately 0.15 m large and horizontal timber square strips (0.02 to 0.03 m wide) filled on the breaks by rubble masonry, resulting in walls with 0.10 to 0.12 m thickness. There are some cases where these walls were placed perpendicular to the main beams or used as part of the staircase structure. In these cases, the ‘tabique’ walls were reinforced by inclined timber strips (15º with the vertical direction).

a) b)

Figure 54 – Interior structure characteristic of ‘gaioleiro’ buildings: a) Solid ceramic brick wall (Andrade, 2011); and b) ‘Tabique’ walls (Pena, 2008).

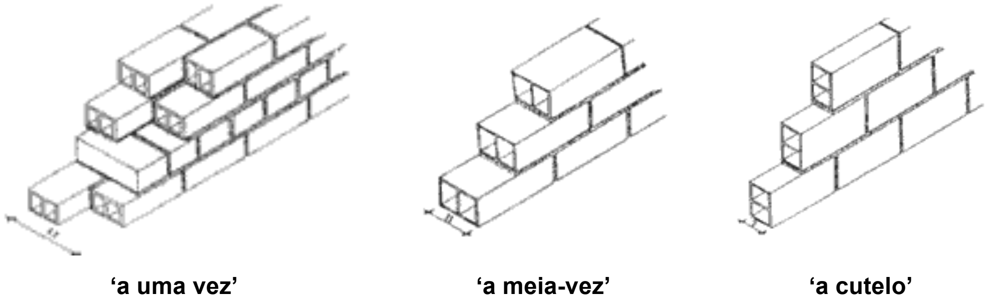


Figure 55 – As the traditional bricks had 0.22 x 0.11 x 0.07 m^3, the thickness (without plaster cover) of 'a uma vez' wall is equal to the length of the brick, 'a meia-vez' is equal to the height and 'a cutelo' is equal to the thickness of the brick (Jones, 2002).

The floors were made by wooden beams, with 0.07 to 0.08 m width and 0.16 to 0.22 m height disposed 0.35 to 0.40 m apart, supported on the façade walls and on the interior walls, and braced on the other direction by smaller beams (Figure 56.a). The beams edges were embedded on the masonry walls and occasionally fixed by steel nails.

The floors were made of soft pine strips with 0.15 m large arranged perpendicular to the main wooden beams or ceramic mosaic (in kitchens and bathrooms). The ceilings were finished by wooden strips or with square strips covered with plaster and interesting stucco details, frames and rosettes (Figure 56.b). The floor has an overall thickness of about 0.30 m.

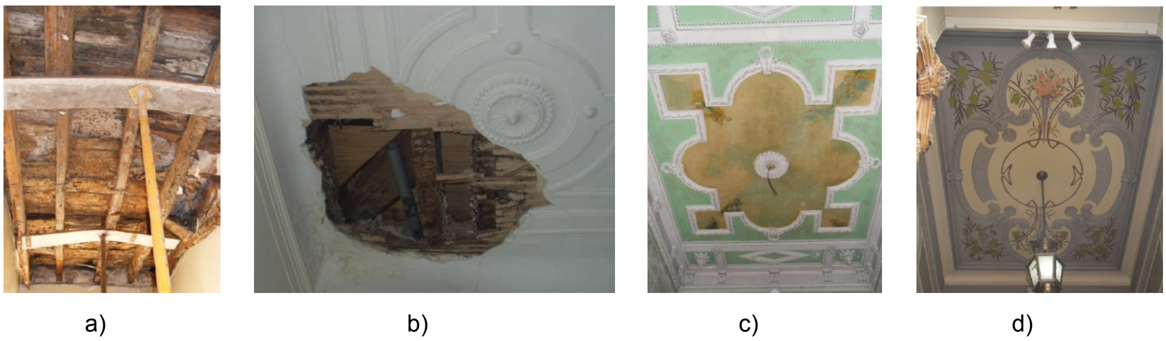


Figure 56 – Floor structure: a) Excessive deformation of a timber beams where apparently no locking joist were use; b) Ceiling finished with laths and plaster (Branco, 2007); c) and d) Examples of ceiling covered by stucco details and frames (Andrade, 2011).

The balconies and galleries from the back façade walls were built with iron beams in shape of I or T profile (around 0.20 m height) and brick masonry disposed in vaults, interconnected by air lime mortar or cement (Figure 57). The floors were restrained by the side and back masonry walls and supported on border beams and slender circular columns (Figure 58.a). Occasionally, there are diagonal tie rods connecting the border beams to the façade walls, supporting balconies which can reach 2.5 m depth.

The service staircases were made of circular steel columns, braced by I or T profile beams, which was the support to the stairs made of steel grid plates (Figure 58.b). The columns were founded on masonry sabots, while the stairs levels were embedded on the balconies' beams.

Figure 57 - Floors made of steel beams and ceramic bricks interconnected by mortar or cement (Appleton, 2005).

a) b) c)

Figure 58 – Back façade wall: a) Steel balconies supported by slender columns (Appleton, 2005); b) Steel service staircase; c) Interior backyard.

The steel floors were gradually adopted on the kitchen and bathrooms floors. The use of steel flooring became popular due to the supposed superior durability of iron in relation to wood, especially in areas in contact with water. However, due to the insufficient protection of these elements against corrosion, this solution turned out to be equally vulnerable.

The buildings were then covered by dual-pitched roofs with dormer windows or mansard roofs which allow higher ceilings and the occupation of the attic. The interior space was divided by a timber truss structure independent from the rest of the building, as the compartment walls were not connected with the interior building structure.

2.5. ‘PLACA’ BUILDINGS

2.5.1. HISTORIC SURVEY

The final period of ‘gaioleiro’ buildings construction is characterized by the complete abandonment of the wooden structures on the walls, the systemic use of brick masonry walls on the interior and by the use of composite floors (steel beams and ceramic bricks) on balconies, kitchens and bathrooms. Nevertheless, the majority of the buildings continue to be built with masonry exterior walls.

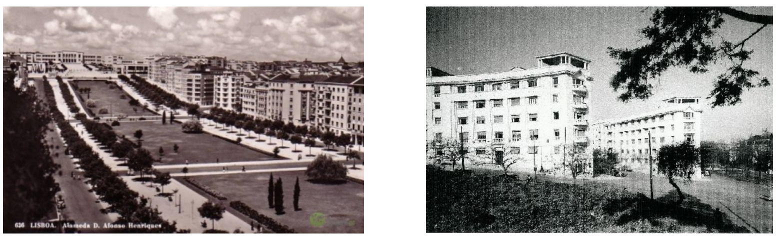
In 1930 the General Regulation of Urban Construction (Original Title: ‘Regulamento Geral da Construção Urbana (RGCU)’) recommended the addition of reinforced concrete beams at the floor level to ensure the locking of the exterior masonry walls, particularly on buildings with more than two stories (including basements) where an interior timber structure was not used.

The early 1930s were marked by the design of social housing neighbourhoods promoted by the Regime so-called ‘Estado Novo’. The political principles promoted the protection of the national values leading to the creation of its own architectural program known as ‘Português Suave’.

The traditional village environment (Figure 59) was recreated by the new neighbourhoods composed by single-family houses. This social policy result on the urbanization of different Lisbon areas: Bairro do Alvito (1937), Quinta do Jacinto (1937), Belém (1938), Camarão da Ajuda (1938), Quinta das Furnas (1938), Quinta da Calçada (1939), Alto da Boa Vista (1939-1940), Alto da Serafina (1940), Encarnação (1940), Madre Deus (1942), Campolide (1943).

Figure 59 – Program of Economic Houses (AFML): Alvito (1937), Alto da Serafina (1940) and Campolide (1943).

The construction of the Instituto Superior Técnico (completed in 1936), Casa da Moeda (1934) and Instituto Nacional de Estatística allow the consolidation of new areas of urban expansion, such as the Arco do Cego neighbourhood and D. Afonso Henriques Alley (Figure 60). In parallel, Eduardo VII Park was crossed by the opening of António Augusto de Aguiar Avenue and Ramalho Ortigão Avenue resulting in the construction of Bairro Azul (Figure 60), intended for wealthier inhabitants.



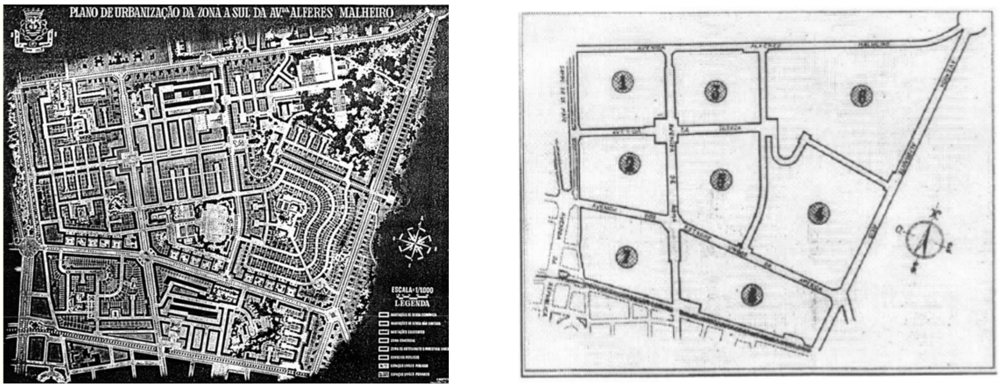
a) b)

Figure 60 – Urban expansion in the early 1930s (AFML): a) D. Afonso Henriques Alley; b) António Augusto de Aguiar Avenue with Marquês da Fronteira Avenue.

In 1938, a new urbanization plan was commissioned by engineer Duarte Pacheco, in the position of Lisbon Mayor and Minister of Public Construction (Original Title: ‘Plano Geral de Urbanização e Expansão de Lisboa’, only published in 1948). The project was designed by the

French architect Étienne de Gröer, involving the main lines of the city development. The program began with an enormous expropriation process which allowed the expansion to the north of the city.

Bairro de Alvalade developed by the architect Faria da Costa, was intended for 45,000 inhabitants (Figure 61). The district provided the integration of different social groups and activities organized in eight central residential compounds (or cells). The area was limited by Alferes Malheiro Avenue (what is now Brasil Avenue) on the north side, Aeroporto Avenue (what is now Gago Coutinho Avenue) on the east, Campo Grande on the west and by the railway line crossing Roma Avenue on the south, occupying an area of 230 ha (Figure 62).



a) b)

Figure 61 – Urbanization of Bairro de Alvalade (AMFL): a) Urbanization Plan of the south area of Alferes Malheiro Avenue developed by the architect Faria da Costa in 1945; and b) Division of the compound in eight cells.

Figure 62 - Overview on Bairro de Alvalade with São João de Brito Church on the left picture (AFML).

The studies for the urbanization of cells I to VII were developed between 1945 and 1957 by the architects Miguel Jacobetty, Fernando Silva, Dário Fernandes and Lima Franco in accordance with the architectural principles of the Estado Novo Regime. The studies for the urbanization of cell VIII on the south of Estados Unidos da América Avenue were developed by the architects Joaquim Ferreira and Orlando Azevedo between 1949 and 1952, approaching the contemporary modern architecture that launched the development of higher buildings built with reinforced concrete framing structures.

In 1947, a new program was developed for the construction of houses of limited income located between D. Afonso Henriques Alley and the railway line limit of Bairro de Alvalade with Bairro dos Actores and S. João de Deus (Figure 63).

Figure 63 – Overview on Guerra Junqueiro Avenue (S. João de Deus district) and Bairro dos Actores.

2.5.2. DESCRIPTION OF BUILDINGS

The social housing neighbourhoods promoted by the 'Estado Novo' Regime were mostly composed by single family houses with one to two stories in line with the rural areas. With the opening of António Augusto de Aguiar Avenue and Ramalho Ortigão Avenue starts the development of straight buildings characterized by the simplicity of the façade walls contrasting with the preceding 'gaioleiro' buildings.

The design of the front was dominated by large smooth surfaces covered in marble and by the complete absence of decorative details (Figure 64). Apart from the exterior modification, the buildings structure continues to be made of rubble stone masonry walls and timber floors. Nevertheless, the use of reinforced concrete elements (prefabricated) started to appear on the façade balconies and interior staircases.

Figure 64 – Building designed by the engineer Duarte Pacheco (1940) in an attempt of defining the Portuguese Architecture model for the rest of the country (Pereira and Buarque, 1995). The building is located at António Augusto de Aguiar Avenue. The floors are still made of timber beams.

Bairro Azul results from the opening of three parallel streets, adjacent to the intersection of Marquês da Fronteira Street and António Augusto de Aguiar Avenue. The buildings were disposed in extended quarters with interior backyards (Figure 65). The access to Ressano Garcia Avenue was marked by two outstanding buildings designed by the engineer Jacinto

Bettencourt (Figure 66), while the other buildings of the compound have four residential floors and the ground floor intended for shops.

a) b)

Figure 65 – Bairro Azul: a) Satellite view (Google Maps); b) Ressano Garcia Avenue.

a) b) c)

Figure 66 – Bairro Azul: a) António Augusto de Aguiar Avenue; b) António Augusto de Aguiar Avenue with Ressano Garcia Avenue; c) Ramalho Ortigão Avenue.

The design of the façade walls was very much identical and often marked by concrete balconies. In fact, the repetition and uniformity of the buildings reminds the composition of the 'pombalino' downtown. This architectural geometry was also implemented along with the urbanization of D. Afonso Henriques Alley, Bairro dos Actores, Arco do Cego, Londres Square and Roma Avenue (S. João de Deus district); although, in this areas the buildings were usually higher (Figure 67).

a) b) c)

Figure 67 – New neighbourhoods: a) Bairro dos Actores; b) and c) Roma Avenue.

Bairro de Alvalade began with the construction of the Economic Rent Houses located on cells I and II, financed by the 'Federação das Caixas de Previdência' (Figure 68). These cells were limited on the north by Brasil Avenue, on east by Rome Avenue, on the south by Estados Unidos da América Avenue and on the west by Campo Grande, occupying an area of about 47 hectares.

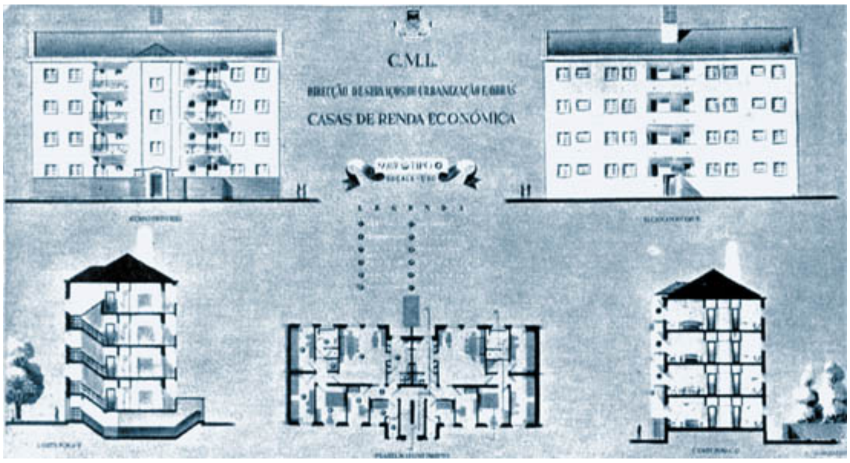


Figure 68 – Design of the Economic Rent Houses (Original Title: 'Casas de Renda Económica') designed by the architect Miguel Jacobetty in 1945 (Alegre, 1999). By the end of 1947, 225 buildings were almost completed.

The main idea was to start the urbanization of the district in a dynamic and active way in order to catch the private constructors for the urbanization of the remaining cells located on the south, closer to the urban area and therefore, valuing the land. The public intervention was after limited to the organization of the allotment, the selling of properties with the plan of the buildings attached and the coordination of works and public spaces.

The construction of cells I and II was developed between 1947 and 1950, and in the same year, began the construction of Houses of Economic Rent on cells V and VI. These buildings, with no more than four floors, were grouped in bands with interior backyards guaranteeing the natural light and ventilation (Figure 69). The exterior design of the buildings had few architectonic details in accordance with the picture of traditional village (Figure 70). The front façade wall had a symmetric design with balanced balconies aligned along the height of the building. The main entrance was located on the centre of the building accessing a two flight staircase separated by a half-landing, guaranteeing two houses per floor.

Figure 69 – The front façade walls were disposed along the grid of secondary streets or interior sidewalks (Coelho, 2006).

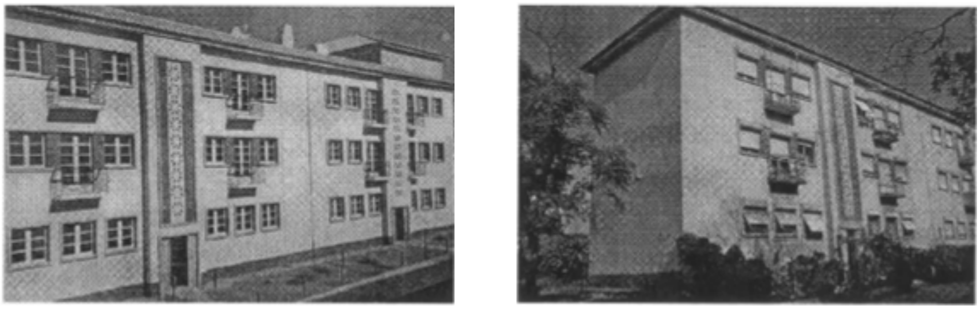


Figure 70 – Economic Rent Houses from cell I and II (Costa, 1997).

For the first time, after the Marquis of Pombal, the construction methods were based on normalization and rationalization principles, producing buildings of great uniformity in architectural and constructional terms. It is common the use of prefabricated elements, concrete balconies and staircases for example, or doors and windows framing, resulting in less expensive constructions.

Alongside, the urbanization of Bairro dos Actores and S. João de Deus was also in progress with the construction of Limited Rent Houses. These neighbourhoods were limited on the south by D. Afonso Henriques Alley, on west by Roma Avenue and on the north by the railway line.

These buildings have a characteristic shape in plan known as 'Rabo de Bacalhau' (Figure 71) originated by the expansion of the lateral light-shafts characteristic of the 'gaioleiro' buildings into the back yard of the compound of buildings. The size of the 'Rabo de Bacalhau' depends on the size of the building. This prominent shape of the building was intended for the service areas, including a secondary staircase to the building, while the living rooms were located in the main block of the building.

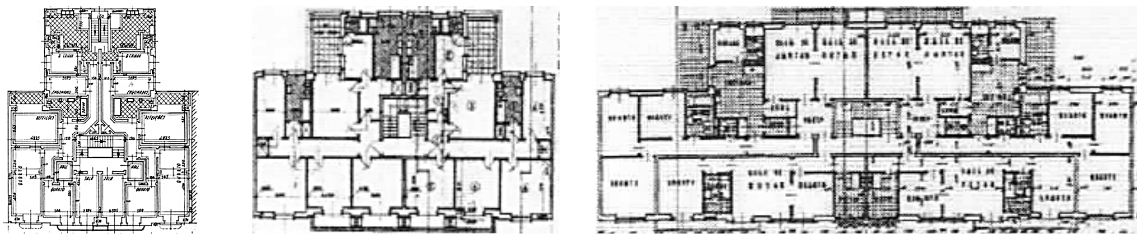


Figure 71 – Placa buildings with the characteristic shape in plan known as 'Rabo de Bacalhau' (Costa, 1997) (Nereu, 2001).

Cell III from Bairro de Alvalade limited on the north by Brasil Avenue, on east by Rio de Janeiro Avenue, on the south by Igreja Avenue and on the west by Roma Avenue is also characterized by the typology of 'Rabo de Bacalhau' buildings. This group was composed by mixed buildings, with commercial ground floors and housing on the upper floors, creating an elongated quarter in shape of U disposed along the Igreja Avenue with the open side to the north side (Figure 72).

Figure 72 – Overview on Igreja Avenue and Rio de Janeiro Avenue.

São Miguel compound was developed between 1949 and 1951 and corresponds to the best quality buildings of Bairro de Alvalade (Figure 73). The design of the buildings is similar to the design of the Economic Rent Houses; however, these buildings are slightly larger, allowing for more spacious rooms. The buildings from cell IV, limited on the north by D. Rodrigo da Cunha Avenue, on east by Gago Coutinho Avenue, on the south by Estados Unidos da América Avenue and on the west by Rio de Janeiro Avenue, represent the single family houses comparable to the design of social houses from Campolide and Encarnação districts.

Figure 73 – Bairro de São Miguel (Costa, 1997).

The urbanization of cell VIII, limited on the north by Estados Unidos da América Avenue on east by Gago Coutinho Avenue, on the south by the railway line and on the west by Roma Avenue, started to integrate some characteristics that set close to the modern European architecture. The traditional sense of urban design was effectively overturned after the First National Architecture Congress (1948), which renews the principles of modern urbanism set by the Charter of Athens (1933)[6].

After the 1950s masonry was no longer used on the construction of buildings in Lisbon, being limited to the construction of single family houses on the rest of the country. This period is also witness of the gradual increase of the buildings' height, which apparently was no longer constrained by the limitations imposed by the masonry structures.

[6] The Athens Charter results from the Fourth International Congress of Modern Architecture, held in Athens in October 1933. This document develops the principles for the so called Functional City that promotes the separation between the residential, leisure and work areas on the city and proposes the concept of garden city, where the buildings grow in height and are surrounded by gardens.

The earliest buildings in height built on Bairro de Alvalade appeared at the intersection between Estados Unidos da América Avenue and Gago Coutinho Avenue through the construction of two towers with ten storeys (Figure 74.a). It was the beginning of the reinforced concrete period as the main structural solution, perceptible by the construction of Bairro das Estacas (meaning piles) designed by Formosinho Sanchez and Rui d'Athouguia, which received several national and international awards (Figure 74).

a) b) c)

Figure 74 – Buildings from the VIII cell: a) Tower at the intersection between Estados Unidos da América Avenue and Gago Coutinho Avenue (Costa, 1997); b) and c) Overview of Estados Unidos da América Avenue (AFML).

2.5.3. DESCRIPTION OF THE STRUCTURE

The 'placa' buildings sign the abandonment of the traditional techniques and materials and the progressive adaptation to effective structural solutions of construction. This period is well acknowledged on the urbanization of Bairro de Alvalade (Figure 75).

Figure 75 – Transition from 'placa' buildings to reinforced concrete buildings with increasing number of storeys (AFML and Lopes *et al.*, 2008).

The Economic Rent Houses from cells I and II, and later in cells V and VI, were made with exterior rubble stone masonry walls and interior brick masonry walls (solid or hollow). These constructions represent the last examples of the use of timber floors made of pine beams (usually with 0.08 x 0.16 m^2 sections) and covered by pine boards. The roof structure was made of wood trusses covered by ceramic roof tiles.

Apparently, these buildings are not very different from the last 'gaioleiro' buildings; however, as recommended on the General Regulation of Urban Construction (RGCU, 1930), the timber floors started to be strengthened by peripheral concrete beams supported on the exterior masonry walls. Sousa *et al.* (2006) suggest that these concrete beams were reinforced by 4ϕ3/8'' longitudinal flat rods on the corners braced by ϕ3/16'' stirrups generally 0.20 cm spaced.

The reinforced concrete solutions, firstly introduced as ring beams at the floor level, were progressively replacing other structural elements. The presence of Portland cement and concrete starts to be noticed on the interior walls made of hollow concrete blocks, on the prefabricated concrete slabs from the balconies and staircases, and on other covering materials supporting a more economic construction.

In the late 1950s several works were carried out on the Economic Rents Houses under the direction of 'Federação das Caixas de Previdência' (Alegre, 1999). The contract refers to the replacement of the existing wooden floors by the combination of ceramic blocks supported by concrete prestressed beams (Figure 76).

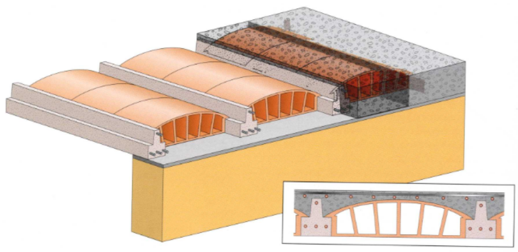


Figure 76 – Ceramic blocks supported by concrete prestressed beams (Lopes *et al*., 2008).

With the development of the Bairro de Alvalade, in parallel with the construction of Bairro dos Actores and S. João de Deus, the back balconies and the service rooms of the houses (kitchens and bathrooms) started to be built with slender concrete slabs (70 to 100 mm thickness) supported on the masonry walls.

After, the solution was extended to the whole floor supporting the name 'placa' (meaning concrete slab) given to this typology of buildings. These concrete slabs were barely reinforced by steel rods, generally with only one layer of reinforcement for positive moments, and there is also no guarantee on the continuity between spans (Alegre, 1999).

The reinforced concrete frame structure started to appear at the ground floor of the buildings meant for commercial occupation. The need of larger spans on the front wall and open spaces was overcome by the use of reinforced concrete beams supported on reinforced concrete columns. At this level the exterior walls were also built making use of hollow concrete blocks (Figure 77). Nevertheless, the structural differences between the ground floor and the upper floors results in important stiffness variations on the buildings, leading to significant structural irregularities and the interruption of the interior walls on the first floor.

Figure 77 – Front and back façade walls of a ‘placa’ building in Guerra Junqueiro Avenue (Lopes *et al.*, 2008). The concrete elements on the façade walls allow larger spans on the ground floor.

The concrete frame structure was then extended to the exterior structure, first on the corners of the building making the connection between perpendicular rubble masonry walls and on the back façade wall structure. For instance, on ‘Rabo de Bacalhau’ typology (Figure 71) the prominent shape of the building is made with a reinforced concrete frame structure and concrete slabs. There are other cases where the side masonry walls were replaced by unreinforced concrete blocks with or without reinforced concrete beams and columns (Lopes *et al.*, 2008).

These concrete columns were reinforced by 4ϕ5/16’’ longitudinal flat rods on the corners braced by ϕ3/16’’ stirrups generally 0.25 cm spaced (Sousa *et al.*, 2006). Connected with this transition, there is an evident absence of specific design features in terms of the amount and detailing of the reinforcement to ensure the structural safety and ductility of the system. Moreover, the concrete used on the buildings has a low to moderate resistance (C20/25 on the best cases) and was slightly compact. The reinforced concrete structures are slender and lightly reinforced, revealing problems of excessive deformability and structural weaknesses in the face of the seismic action.

The generalization of reinforced concrete structures has imposed a revolution on the construction habits, on the structural solutions and on the architecture of the buildings. It establishes a clear separation between the old buildings made of masonry and wood structures and the modern buildings characterized by the dominant use of concrete.

3. VULNERABILITY OF OLD MASONRY BUILDINGS

3.1. GENERALITIES

The seismic risk results from the product between hazard, exposure and vulnerability. When one of the parameters is low, the seismic risk is also low. Usually it is not possible to reduce the seismic hazard as it is a natural condition. As to the exposure there are also limitations, since it implies restricting the settlement of populations in areas of higher seismic risk. The vulnerability is related with the seismic resistance of the buildings, i.e., depends on the structural solution, including design, materials and constructive techniques; parameters that change with the region (available materials) and period of construction. The vulnerability results from the human action, and therefore it is possible to operate some changes in this field.

The period of construction is in fact an indicator of the seismic resistance of the buildings. For instance, the first anti-seismic guideline in Portugal appeared with the 'pombalino' reconstruction after the 1755 earthquake and was rigidly imposed during the eighteenth century. Although, on the nineteenth century, these construction methods were gradually abandoned resulting on the design of buildings with inferior constructive quality. On the other hand, the introduction of reinforced concrete solutions between the 1930-1950 decades represented an improvement to the resistance of the buildings.

Nevertheless, the first written code developed for the design of seismic-resistant structures was only published in 1958 ('Regulamento de Segurança das Construções contra os Sismos[7] (RSCCS)'), after the Symposium about the Seismic Action (Ordem dos Engenheiros, 1955) with the purpose of reminding the bicentenary of the 1755 Earthquake and discuss the effects of earthquakes on structures and ways to mitigate those effects. The regulation was then replaced in 1961 ('Regulamento de Solicitações em Edifícios e Pontes[8] (RSEP)') conserving the main principles from the RSCCS, however the seismic actions was now effectively included on the safety verification of structures.

In the beginning of the 1960s the masonry walls were barely used as structural elements on the construction of buildings in Lisbon. It was the beginning of the reinforced concrete as the main structural solution. The first period of construction was characterized by the use of slender frame structures with several irregular dispositions on the buildings. Only after the publication of RSEP (1961) and with the regulation of reinforced concrete structures ('Regulamento de

[7] In English: Safety Constructions against Earthquakes Regulation
[8] In English: Actions in Buildings and Bridges Regulation

Estruturas de Betão Armado[9] (REBA)', 1967) the seismic action started to be considered on the safety verification of reinforced concrete structures.

The effective compliance of the technical regulations is then reflected on the average improvement of the seismic resistance of the building structures, emphasized by the publication in 1983 of new regulations ('Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes[10] (RSA)' and 'Regulamento de Estruturas de Betão Armado e Pré-Esforçado[11] (REBAP)'), which since then have not been updated. Nowadays, Eurocode 8 (EC8, CEN, 2004) set the guidelines for the design of structures in seismic areas.

In conclusion, old masonry buildings were built long before the modern seismic-resistant regulations, although this does not necessary imply that these buildings are vulnerable. Assuming that the average expected life for a building is fifty years, old masonry buildings already fulfil the function for which they were designed. The assessment of the seismic vulnerability of old masonry buildings aim to identify the need, the type and the extent of a structural reinforcement.

The analysis can be performed with a specific building or with a prototype representative of a characteristic group of buildings (Candeias, 2000). On the first case, the evaluation comprehends the survey of the building including the structural modifications performed during its lifetime and causes of degradation, followed by the modelling of the structural and non-structural elements of the building.

The current codes are mostly prepared to the design of new buildings, and might be too demanding when applied for existing buildings. In some cases, it seems acceptable to ensure a lower security level for the older buildings, if it represents an improvement from its current condition (Lopes *et al*., 2008). Though, the European Codes do not clearly state this possibility, as the design seismic action is defined for the lifetime of current buildings.

On the second case, the assessment comprehends the definition of classes of buildings with common characteristics (building typologies). Each typology is connected to a prototype that represents the average structural behaviour of the buildings. Since the prototype characterizes a type of construction, this second approach is conceptually equivalent to the individual approach (Candeias, 2008).

Depending on the objectives of the analysis and on the available information, the assessment of the seismic vulnerability can be divided into the methods of calculated vulnerability and the methods of observed or subjective vulnerability (Sousa, 2006). The first group of methods includes the assessment of the seismic response and damages on the structure through the development of vulnerability curves function of a given seismic action (this is the method within

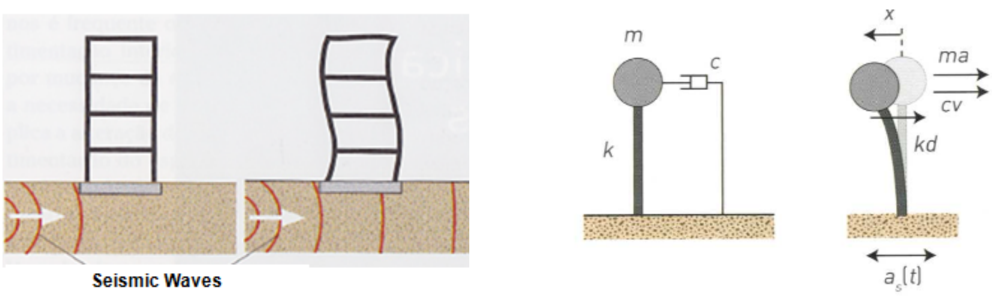
[9] In English: Reinforced Concrete Structures Regulation
[10] In English: Safety and Actions for Building and Bridges Structures
[11] In English: Reinforced Concrete and Prestressed Structures Regulation

the scope of the research project SEVERES). The second group of methods is a qualitative analysis based on the damage reported on the buildings subjected to past earthquakes. The procedure comprehends the analysis of the level and extent of damages on the buildings in connection and the association of its current state to a vulnerability class.

3.2. EFFECTS OF THE SEISMIC ACTION

As the overall subject is the seismic vulnerability of old masonry buildings, it is first necessary to understand and characterize the phenomenon and after the main effects of the seismic action over the buildings.

The majority of the earthquakes results from the accumulation of internal tensions on Earth's crust leading to the rupture of geological faults. The event origins the sudden release of energy generating seismic waves that spread in all directions at high speed. The seismic waves instigate various disorders, including ground vibrations that are transmitted to the foundation of the buildings and other equipments (bridges, networks or geological structures) causing their deformation (Figure 78).



a) b)

Figure 78 – Effects of the seismic action (Lopes *et al.*, 2008): a) Seismic excitation on an ordinary structure; b) Single degree of freedom structure subjected to a horizontal acceleration at the base (Lopes *et al.*, 2008).

The dynamic behaviour of the structure can be easily analysed through an oscillatory system. Each vibration mode represents a deformed configuration of the structure with a specific vibration period or frequency[12] (Lopes *et al.*, 2008). The first vibration mode of the structure, also called the fundamental mode, corresponds to the highest vibration period of the structure, and consequently to the lower vibration frequency. In the case of a single degree of freedom structure with mass (M) and stiffness (K), the vibration frequency is determined according to equation (3.1):

[12] The vibration period (T) represents the time required for a complete vibration (oscillation), measured in seconds (s). The vibration frequency (f) represents the number of oscillations (cycles) performed during a second, measured in cycles/second or Hertz (Hz). The vibration period and the vibration frequency are inversely proportional.

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \tag{3.1}$$

The capacity of a structure to withstand an earthquake is strongly related with its resistance to the inertia forces generated in the mass of the structure itself[13]. In addition to the inertia forces (F_i), there are other forces that ensure the dynamic equilibrium of the system, including: damping forces (F_d), elastic recovery forces (F_r) and external forces (F_{ext}), according to the following equation (3.2):

$$F_i + F_d + F_r = Ma + C(v - v_s) + K(d - d_s) = F_{ext} \tag{3.2}$$

The damping forces (F_d) are responsible for the progressive reduction of the amplitude of the movement through the viscous behaviour of the materials (energy dissipation), and therefore are orientated against the movement. These forces are proportional to the viscous damping coefficient (C) and to the difference between the structure and the ground velocity ($v - v_s$).The elastic recovery forces (F_r) represents the ability of the structure to return to its original position and are proportional to the stiffness of the system (K) and the difference between the structure and the ground displacement ($d - d_s$).

"*The random nature of the seismic events and the limited resources available to counter their effects are such as to make the attainment of these goals only partially possible and only measurable in probabilistic terms*" (EC8-1, CEN, 2004). The seismic action at a given point is commonly represented by an elastic ground acceleration response spectrum, based on the range of ground accelerations recorded on the region. The seismic action is described by two horizontal and orthogonal components, assumed as being independent, and one vertical component, insignificant for the majority of the structures but important for old masonry buildings as following referred (Lopes *et al.*, 2008).

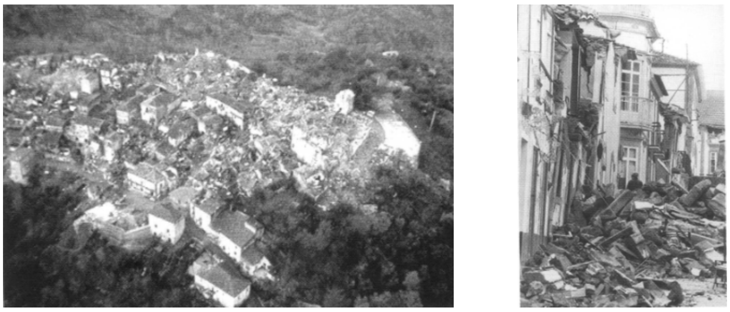
The Portuguese National Annex of Eurocode 8 (EC8-1, CEN, 2005) defines two types of seismic action for each location: Type 1 – representative of the interplate earthquakes and Type 2 – representative of intraplate earthquakes. Within each type of seismic action, the response spectrum considers five types of foundation soil, depending on their local stiffness, and different damping values, depending on the structural typology.

With the definition of the design response spectrum (typically plotted against structural vibration period (T)) and knowing the fundamental frequency (f) and the viscous damping ratio of the

[13] The ground shaking is measured by the ground acceleration (proportional to the speed variation) which is directly related to the forces that the objects are subjected. According to the Fundamental Law of Dynamics (2[sd] Law of Newton), the force (F) is directly proportional to the mass (M) and to the acceleration (a) of a body ($F = Ma$).

structure (ζ) it is possible to determine the maximum response of the structure to the seismic action in question.

The severity of the local effects of an earthquake depends in one hand, on the earthquake magnitude, the type of rupture and distance to the fault (hypocentre), and on the other hand, on the local geological and geomorphologic conditions (Figure 79).



a) b)

Figure 79 – Effects of the seismic action (Lopes *et al*., 2008): a) Village in Italy where almost any building escape undamaged. It is referred that the foundation soils had a soft nature resulting on the amplification of the seismic action on the area; b) Widespread collapse of a street in Angra do Heroísmo, Azores, after the 1980 Earthquake.

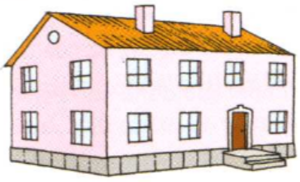
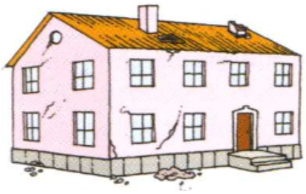
The Moment Magnitude Scale (formerly Richter Scale) measures the energy released with the earthquake, whereas the Modified Mercalli Scale (intensity II–XII) quantifies the effects of the earthquake based on the reported and observed structural damages. The recent European Macroseismic Scale (EMS-98) quantifies in more detail the damages connected to each type of buildings. Table 2 reproduces the classification of the damage on masonry buildings to be used on the calculation of the EMS-98 intensity.

The seismic activity in Portugal Mainland and Islands is marked by several episodes of greater or lesser relevance. Despite the present reduced activity, the seismic events are recurrent and what happened in the past will certainly happen again in the future. Taking into account the period of construction of the old masonry buildings (excluding the group of buildings built before the 1755 Earthquake), the most important earthquakes in Portugal Mainland that these buildings had to withstand were the Benavente Earthquake in 1909 (district of Santarém) and the Lisbon Earthquake in 1969.

With the Benavente Earthquake[14], the most affected buildings structures were composed by single family houses made of clay or abode walls (Sousa, 2006), and therefore few information is available regarding the damage on the stone masonry buildings.

[14] The Benavente Earthquake of April 23, 1909 reached a magnitude less than 7.0 and was the largest intraplate earthquake of the twentieth century in Portugal Mainland. Originated in the Tejo River Valley fault, the earthquake had its epicentre between Samora Correia and Benavente. It was felt with a maximum intensity of grade IV to X (Modified Mercalli Scale) in Benavente, Samora Correia and Santo Estevão (districts of Santarém) and with maximum intensity of VI in Lisbon (SPES, nd).

Table 2 – Classification of damage on masonry buildings to be used in the calculation of EMS-98 intensities (adapted from Lopes *et al.*, 2008).

	Level 1 – Without damage or light damage (without structural damage, light non-structural damage)
	Level 2 – Moderate damage (light structural damage, moderate non-structural damage)
	Level 3 – Substantial to severe damage (moderate structural damage, severe non-structural damage)
	Level 4 – Very severe damage (severe structural damage, very severe non-structural damage)
	Level 5 – Collapse (very severe structural damage)

Conversely, with the Lisbon Earthquake[15] there is a record of the main damages occurred in the old masonry buildings, approximately with more than 40 years old (Candeias, 2008). The most relevant aspects might be summarized to "*(...) there is considerable damage (...). Such damage should be attributed in large part to the poor quality of masonry, the precarious state of conservation and the lack of locking between the resistant walls and the compartment walls. The connection between orthogonal walls is often made by the simple support without the interconnection of the elements, from which result the cracks on the wedges and the consequent bend of the resistant walls.*" (Candeias, 2008). This simple description identifies the main vulnerabilities of the old masonry buildings that will be developed on the following sub-sections.

[15] The Lisbon Earthquake of February 28, 1969 reached a magnitude between 6.5 and 7.5 and was the originated near the Gorrige Bank (interplate earthquake), located approximately 200 km southwest of São Vicente Cape (Algarve). Although strongly felt in Lisbon and in the south of Portugal, the earthquake caused few damage (SPES, nd).

3.3. EARTHQUAKE RESISTANT BUILDINGS

The Eurocode 8 (EC8-1, CEN, 2004) defines a set of basic principles for the conception of earthquake resistant buildings. Although defined for new structures, some of these principles may be reviewed on the context of existing structures, in order to ensure responsible future interventions. The main principles are:

(i) Structural Simplicity

The structural simplicity is characterized by the existence of clear and direct paths for the transmission of the seismic forces that allow a reliable prediction of the seismic behaviour of the structure.

(ii) Uniformity (both in plan and elevation)

"*Uniformity in plan is characterized by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building*" (EC8-1, CEN, 2004). The best plan configurations are the convex or compact shapes, for example rectangles with similar side length.

"*Uniformity in the development of the structure along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse*" (EC8-1, CEN, 2004).

Since the lower floors are commonly subjected to higher axial loads due to the weight of the upper floors and higher internal stresses due to the seismic action, it is reasonable to accept the gradual dimensional variation of the elements. For instance, the thickness variation of exterior masonry walls or the variation on the type of interior walls is common on the old masonry buildings.

Conversely, the structural discontinuity on the elevation of the building has serious consequences to the balance of the structure, since the natural pattern of the loads to the foundation is interrupted. The removal of structural elements requires the transfer of the loads to the adjacent structural elements overloading them.

Regarding that the ground floor is the most demanding level of the buildings, it is much negative to decrease its stiffness or resistance from the top to the bottom. Nevertheless, there are several cases related with the removal of structural elements on base of the building, for instance for the opening of larger windows. In this circumstances, the deformation due to the horizontal loads tend to be concentrated on the soft storey, generating large interstorey drifts and important second order effects on the supporting elements. These irregularities usually lead to the rupture of the elements and can result on the global collapse of the building.

(iii) Symmetry

"*A close relationship between the distribution of masses and the distribution of resistance and stiffness eliminates large eccentricities between mass and stiffness*" (EC8-1, CEN, 2004), minimizing the tendency for the rotation of the floors in their plan. The symmetrical layout of structural elements is appropriate to achieve uniformity.

(iv) Redundancy

Redundancy is related with the number of connections beyond the necessary to guarantee the support of the loads involved. "*The evenly distribution of structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure*" (EC8-1, CEN, 2004).

(v) Bi-directional resistance and stiffness

The seismic action is a bi-directional event and therefore "*the building structure shall be able to resist horizontal actions in any direction. The structural elements should be arranged in an orthogonal in plan structural pattern, ensuring similar resistance and stiffness in both main directions*" (EC8-1, CEN, 2004). The building stiffness, "*while attempting to minimise the effects of the seismic action should also limit the development of excessive displacements that might lead either instabilities due to second order effects or excessive damages*" (EC8-1, CEN, 2004).

(vi) Torsional resistance and stiffness

"*Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. Arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages*" (EC8-1, CEN, 2004).

(vii) Diaphragmatic behaviour at storey level

"*In buildings, floors (including the roof) (...) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action*" (EC8-1, CEN, 2004).

This rigid diaphragm behaviour is extremely favourable in the resistance to horizontal inertia forces as it takes the complete advantage of the strength and stiffness of the elements with greater stiffness, which resist to the horizontal inertia forces generated in the vicinity and to the generality of the inertia forces generated on the floor (Lopes *et al*., 2008). This behaviour also contributes to a typified dynamic behaviour of the structure (masses vibrate together), easier to characterize and predict, leading to a more reliable analysis models.

It is important to emphasize that for most of the old masonry building the floor is in wood, thus it is not act as rigid in its plan.

(viii) Adequate connection between the elements

The correct connection between structural elements is a basic principle of a good conception. For example, the connection between orthogonal walls generates three-dimensional elements with greater strength and rigidity than the result of each isolated wall. The joined operation of the structural elements is essential for the good seismic performance of the buildings, preventing the overturning of the façade walls, which is the most common failure of old masonry buildings.

(ix) Adequate foundation

"*The design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation*" (EC8-1, CEN, 2004). The inadequate type of foundation or its undersize can lead to excessive deformations of the soil foundation, causing vertical movements of the foundation system due to the permanent loads on the structure (differential settlements). In the case of an earthquake, the problem may be aggravated as the action can affect the mechanical properties of the soil and its support capacity, inducing liquefaction or settlements.

The soil liquefaction results from the modification of the soil properties during an earthquake. In the presence of water, the soils become fluid losing the ability to support loads. The uniform liquefaction of the foundation soil results on the settlement of the building. The non-uniform liquefaction results on the rotation of the building. In order to avoid this problem, the foundation system should be extended to ensure the transmission of loads to the bedrock (for example through piles).

3.4. SEISMIC BEHAVIOUR OF OLD MASONRY BUILDINGS

The seismic resistance of a structure depends on the elements capacity of discharging the inertia forces, imposed by the dynamic actions, directly to the foundation system without damaging the building (Lopes *et al*., 2008). This capacity relies on the quality of materials and construction techniques and depends on the position of the walls on the plan of the building.

Differences between the resulting mass and structural stiffness generate torsional motions which tend to stress the structural elements in a non-uniform way. The movement of the structure and the resultant damages decrease the stiffness of the connections between the exterior walls and the interior structure (walls, floors and roof) and also the connections between the interior structure, which are very sensitive to rotation and settlement movements (Lopes *et al*., 2008). Additionally, the inadequate type of foundation or its undersize can lead to excessive deformations of the soil foundation, causing vertical movements of the foundation system due to the permanent loads on the structure (differential settlements).

In the case of the old masonry buildings all the components are important for the proper behaviour of the structure. Masonry has good behaviour when in compression, but a low capacity to withstand shear and tensile stresses. These stresses are sustained on the masonry walls only by the gravity loads (weight of the structure) and by the interaction between masonry stones and mortar joints (friction forces), which with time, frequently lose their binding characteristics[16].

As a result, masonry walls have a good performance in its plane, supporting the vertical axial loads which only introduce compressive stresses on the element. When the walls are subjected to horizontal inertia forces generated by the seismic action, the vertical loads tend to bend the walls, increasing the tensile and shear stresses on the element, stresses that the masonry is practically unable to resist (Figure 80). Moreover, the foundation structure was only designed taking into consideration the vertical static actions on the building and is practically unable to withstand the moments generated by the seismic action.

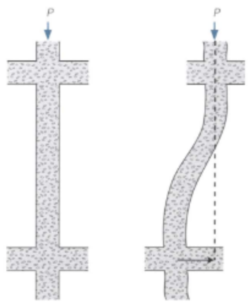


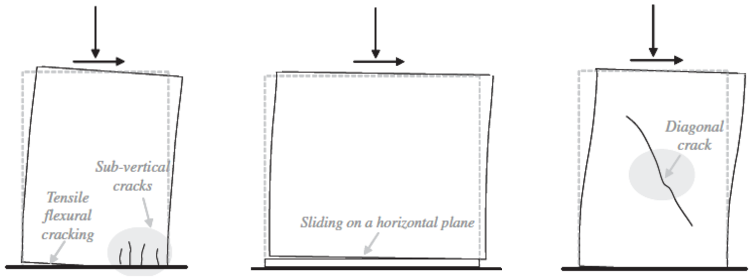
Figure 80 – Horizontal displacements on the structure have to be controlled; otherwise they can lead to the collapse of the element due to the second order effects (Lopes *et al.*, 2008).

The building stiffness, "*while attempting to minimise the effects of the seismic action should also limit the development of excessive displacements that might lead either instabilities due to second order effects or excessive damages*" (EC8-1, CEN, 2004). The connection between the walls, floors and roof structure represent the constraints that limit the deformation and damage on the building.

The assessment of the seismic vulnerability of masonry buildings can be based on the level of structural damage, including the type of failure (flexural or shear behaviour) and the position on the structure. This analysis allows the identification of the possible collapse mechanisms of the buildings related to in-plane behaviour of the walls (Figure 81) and the out-of-plane behaviour.

In their plan, the flexural behaviour of masonry walls occurs when the horizontal actions are not balanced by the vertical compressive actions, producing tensile flexural cracking on the masonry (perpendicular to the main direction of tensile stresses). As a result, the walls practically behave as a rigid body rotating - Rocking Failure Mode (Figure 81.a).

[16] Sometimes, the action of our fingers is enough to reduce the air lime mortar to powder.



a) b) c)

Figure 81 – Typical failure modes of masonry walls in their plan (Calderini *et al*., 2009): a) Rocking; b) Sliding Shear; c) Diagonal Cracking.

The consequent reduction of the contact surface between the elements results on the increment of the compressive stresses. In this case, the wall is characterized by a widespread damage pattern with sub-vertical cracks orientated towards the more compressed corners, which may end up crushing. On the other hand, the development of tensile flexural cracking is attained with sliding – Sliding Shear Failure (Figure 81.b). In both cases, the ultimate state is achieved by failure at the compressed corners.

The shear behaviour of masonry walls may also occur with the formation of a diagonal crack (Figure 81.c). The crack (perpendicular to the main direction of tensile stresses) crosses the element through the mortar joints (assuming the shape of a 'stair-stepped' path in the case of regular masonry pattern) or through the blocks (Calderini *et al*., 2009). The general cracking of the wall reduces the overall stiffness and may also lead to the collapse of the structure.

Old masonry buildings are also vulnerable when exposed to vertical accelerations generated by the seismic action (Lopes *et al*., 2008). The resulting vertical inertia forces modify the balance on the vertical support elements by adding or subtracting the vertical axial loads, depending on the direction of the ground acceleration.

When the vertical inertia forces (from bottom to top) reach significant values, the gravity forces (from top to bottom) strongly decrease, also decreasing the horizontal friction forces generated between the masonry stones and binding mortar. In this sense, the resistance to horizontal forces is significantly reduced and, at the end, it may lead to the collapse of the structure.

From past seismic damage on masonry walls, as well as the results from laboratory experimental tests, it became clear that the occurrence of different failure modes depends on (Calderini *et al*., 2009): the geometry of the wall, the mechanical characteristics of the masonry constituents (stones, mortar and interfaces), the boundary conditions and the acting axial load.

Nonetheless, it might not be enough to classify the type of collapse mechanism, because different failure mechanisms may present similar damages (Cardoso, 2002). For example, the tensile or compressive failure of masonry walls may derive from the out-of-plane or in-plane failure modes, and so it is worth to analyse the tangential stress distribution to clarify which situation prevails (Figure 82).

The resistance of the masonry walls is mostly dependent on the boundary conditions, particularly on the connection between the masonry walls and between the masonry walls and the interior timber structure (Figure 83). The joined operation of the structural elements is essential for the good seismic performance of the buildings, preventing the overturning of the façade walls, which is the most common collapse mechanism of old masonry buildings (Figure 84).

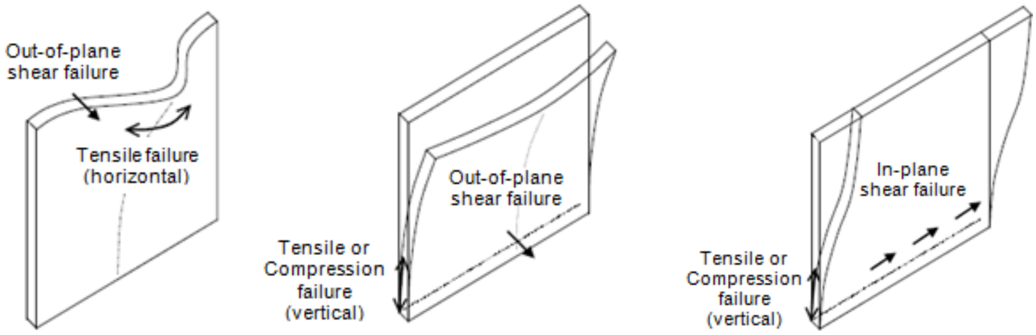


Figure 82 – Out-of-plane and in-plane failure modes (adapted from Cardoso, 2002).

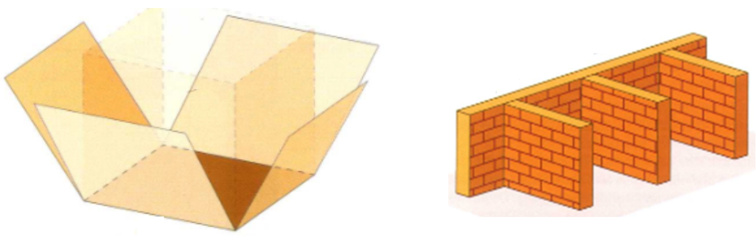


Figure 83 - The lack of connection between orthogonal walls result with the rotation of the walls around the base (Lopes *et al*., 2008).

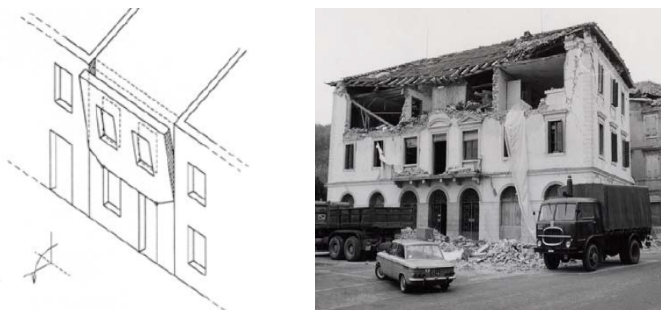
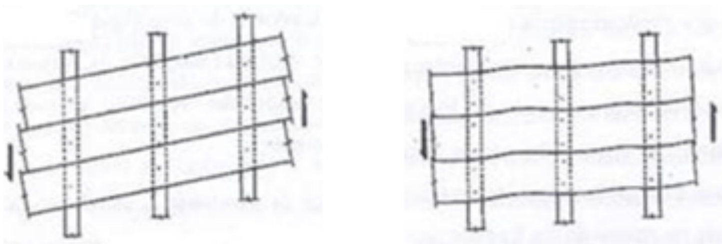


Figure 84 - Overturning of the façade wall (Logormasino, 2007).

The connections between masonry walls are very vulnerable due to the difficult interlocking the masonry units, especially when the buildings share the side masonry walls. On the best quality constructions, the connection between exterior masonry walls was reinforced by regular stone blocks (wedges) or iron ties crossing the connection, and in this cases it is expected a better behaviour of the compound.

The partition walls were placed on both directions providing support to the floors and transversal locking to the exterior masonry walls. Even if it is possible to define which walls are effectively important for the support system, the remaining walls together with the floors also have a structural role (permanent deformation of the structure).

The floors and roof structure also play an important role collecting and transmitting the inertia forces to the vertical structural systems (as aforesaid referred). Timber floors are usually very deformable regarding the weak connections to the masonry walls[17] and the lack of nailing fixation (Figure 85).



a) b)

Figure 85 – Possible deformation of the timber floors (Carvalho and Oliveira, 1997); a) Timber boards and floor beams connected by one nail; b) Timber boards and floor beams connected by two nails.

On flexible floors, the vertical elements support the horizontal inertia forces, approximately in the same percentage as the vertical loads, leading to a major weakness of this type of buildings. Nevertheless, the wooden floors are light structures, and therefore generate lower horizontal inertia forces. Consequently, the replacement of the wooden floors by reinforced concrete slabs or composite floors might be a major problem to the seismic performance of the buildings.

'Placa' buildings generalized the use of reinforced concrete slabs. These slabs act as rigid diaphragms on the horizontal plan, ensuring that the vertical support system works together resisting the horizontal seismic action (see section 3.3.(vii) Diaphragmatic behaviour at storey level).

Figure 86 shows the sequence of the out-of-plane failure of a masonry façade wall. In the case of 'pombalino' buildings, the presence of a well preserved tri-dimensional timber structure may restrain the overturning of the façade walls (group effect), if the connections between different structural elements are adequate.

However, in the case of 'gaioleiro' buildings, the poor connection between the interior walls and floors and between the interior structure and the exterior masonry walls predicts a weaker out-of-plane performance of the masonry walls.

[17] Figure 12 (section 2) suggests the way the connection between the timber floors and the masonry walls was made, and most of the times the timber beams were simply embedded on the masonry.

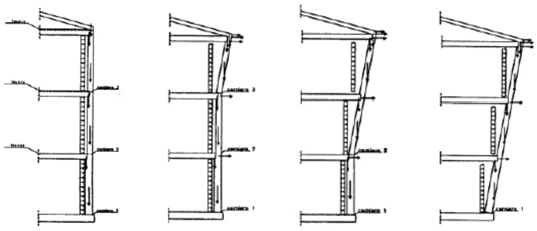


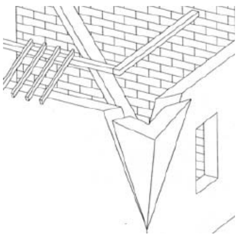
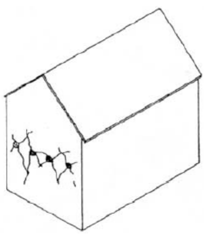
Figure 86 – Sequence of the out-of-plane failure of the façade wall (Cardoso, 2002).

The presence of openings in the walls influences the way the horizontal forces from the upper floors are transferred to the base of the floor[18]. The openings decrease the resistance and stiffness of the panels and affect the in-plane cracking pattern on the façade walls, depending on the number, size and position of the openings. Figure 87 shows a series of damage on masonry buildings resultant, illustrating the effect of an earthquake.

Figure 87 - Damage on a masonry wall around the openings (Logormasino, 2007).

The cracking pattern is characterized by large crossed diagonal cracks on the piers or on the spandrels, consequence of the bi-directional ground acceleration. It is also possible to conclude that the openings should be symmetrically distributed on the panel, should have similar dimensions and should be aligned on the vertical and horizontal direction. On the other hand, the openings should be uniform between the walls of the building, preventing additional stiffness variations. Located failures of the structure might also have serious consequences to the integrity of the building, particularly on the connections between the roof structure to the masonry walls and between the floors and the masonry walls (Figure 88).

a) b) c)

Figure 88 – Located failures (Logormasino, 2007): a) Connections between the roof structure and the masonry walls; b) and c) Connections between the floors and the masonry walls.

[18] Masonry panels may be discretized in piers, which are the principal vertical resistant elements for both dead and seismic loads, and spandrels, which are secondary horizontal elements, coupling piers in the case of seismic loads (Calderini *et al.*, 2009).

The assessment of the seismic vulnerability on historic city centres should consider the structural interaction between buildings as they were frequently built in compounds side by side or slightly apart. The sequence of construction within the quarter is important to understand the actual structural interaction. The former constructions were built as independent units as the following occupied the remaining spaces, sharing the side-walls with the existing buildings (Figure 89). It is expected that the connection between the existing masonry walls and the new façade walls is weaker due to the difficulty interconnection of the masonry stones.

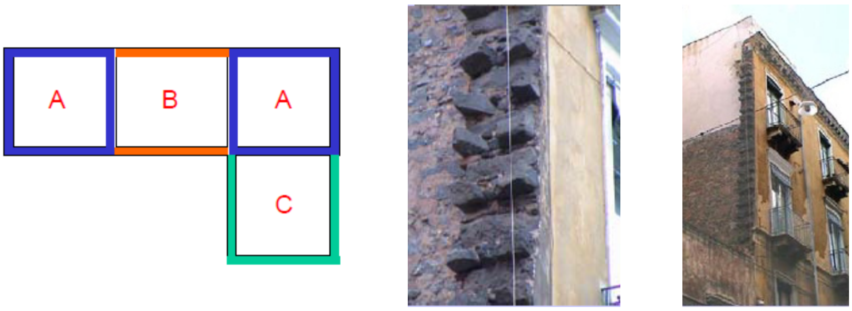
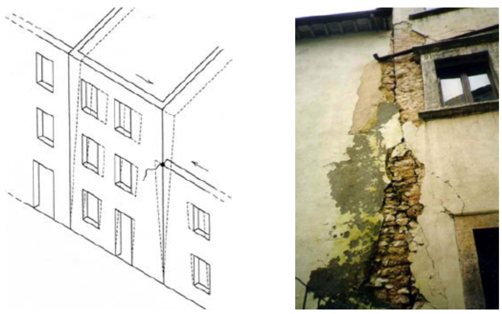


Figure 89 – Example of the construction sequence of a compound of buildings (Logormasino, 2007).

The structural interaction between buildings and the coexistence of different constructive systems might influence their global performance in a positive or negative way, depending on: (i) the position of the building inside the compound; (ii) the structural irregularities (in plan and in elevation); (iii) the pounding effect between buildings.

During a seismic event the horizontal displacements generated on the more flexible structures are constrained by the structures with superior stiffness within the compound. For low vibration amplitudes, the forces on the surface of the gable walls are enough to equalize the displacements between the buildings; however, for higher amplitudes the buildings with different constructive systems tend to vibrate in a different way resulting on the separation and collision between buildings (Figure 90).



a) b)

Figure 90 – Pounding effect between adjacent buildings (Logormasino, 2007): a) Representation of the shock effect between buildings arranged in band; b) Crush between masonry walls.

Buildings placed at the ends of bands or blocks of buildings are in a more vulnerable position because there is no adjacent building to limit the consequent horizontal displacements (Figure

91). Moreover, all the properties inside the compound should be occupied avoiding more buildings on a side position. Conversely, adjacent buildings with different heights are in a much more vulnerable position due to the pounding effect at the top level of the lower building (Figure 92). The damage will mainly occur on the weakest building in the impact area. Nevertheless, in this circumstance, the side-walls became unsupported, and therefore are more susceptible to overturning mechanisms (Figure 92.c).

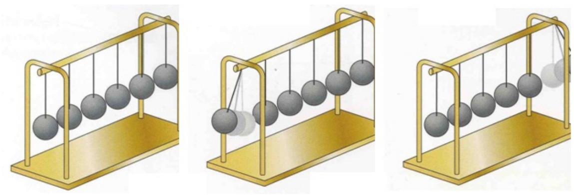


Figure 91 – Shock between buildings arranged in band and the effect over the buildings placed at the end of the band (Lopes *et al.*, 2008).

a) b) c)

Figure 92 - Interaction between buildings with different height: a) Pounding between buildings (Lopes *et al.*, 2008); b) Damage on the lower building caused by the compression applied by the higher buildings. (Lopes *et al.*, 2008); c) Overturning of the side wall consequence of the different height of the buildings inside the compound (Logormasino, 2007).

Differences between the floor level, usually caused by the construction on slopes or different ceiling heights, can also result in severe consequences. The impact of the floor on the gable wall significantly increases the stresses on the area, which may affect the stability of the element (Figure 93).

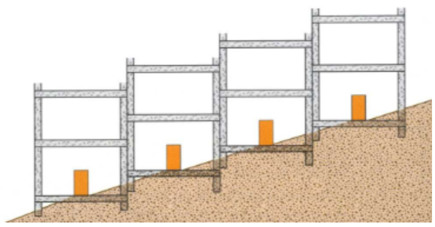


Figure 93 – Buildings with unlevelled floors (Lopes *et al.*, 2008).

The construction of corner buildings on compounds, end buildings on continuous bands or buildings implanted in slopes cannot be avoided. However, the negative effects of these external conditions may be balanced through the increment of the inertia forces that these buildings have to withstand, for example (Lopes *et al.*, 2008).

The assessment of the seismic vulnerability of old masonry buildings may be based on their chronologic evolution, including the survey of the structural modifications and causes of degradation. As the evolution between types and techniques of construction was progressive, each building as to be evaluated as a whole in order to determine which elements have a structural role.

3.4.1. BUILDINGS BUILT BEFORE 1755

The buildings built before 1755 have to be independently assessed in its historical and geographical context. The buildings that still remain were very much changed in result of several interventions performed after the 1755 earthquake and throughout their lifetime.

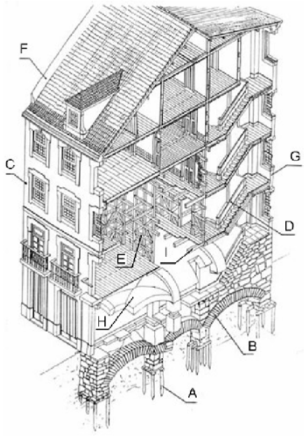
These buildings are probably the ones that pose a higher vulnerability within the typologies of old masonry buildings. Conversely, the reduced number of the buildings and the specifics of the structure require a case-to-case assessment (Figure 94).

Figure 94 – Buildings built before 1755 (Valério, 2011).

3.4.2. ‘POMBALINO’ BUILDINGS

The ‘pombalino’ construction represents the first time in history that a city was entirely built making use of solutions designed to withstand future earthquakes. According to Mascarenhas (1996), the structural regularity of the ‘pombalino’ buildings provided a similar performance of the construction within the compound, which besides reinforcing the group effect also gave them superior structural stability (Figure 95).

The connections between the interior walls, the floors and roof structure to the masonry exterior walls, continuous along the height of the buildings, restrain the deformations and stresses on walls (Lopes *et al.*, 2008). The timber-masonry cage structure on the upper floors results on the buildings strength and energy dissipation capacity, essential to support the seismic actions in any direction. Nevertheless, the timber elements are just notched and barely reinforced by nails, highlighting the fragility of the connections.



A – Wood Piles

B – Vaults of block ceramic masonry and stone arches (ground floor)

C – Masonry front façade wall

D – Wood floors and stairs

E – ‘Frontal’ wall

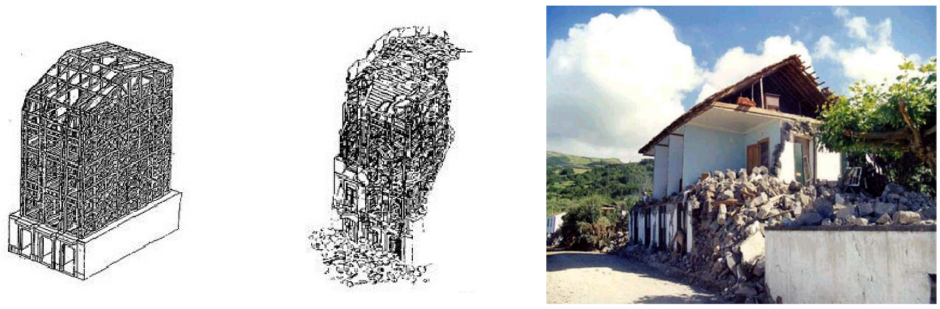
F – Masonry side wall

G – Masonry back façade wall

Figure 95 - Structural details of a 'pombalino' building (adapted from Mascarenhas, 1996).

It was believed by the original designers that during a seismic scenario the external walls would release from the interior structure avoiding the complete collapse of the building (Figure 96). This conception may be efficient for buildings with one or two floor buildings, as shown in Figure 96.c; however, there are some uncertainties when applied to buildings with five storeys, as the overturning of façades probably will bring down other parts of the building (Mascarenhas, 1996).

Research done in the last twenty years has shown that the cage structure still ensures an effective seismic resistance, even in those buildings weakened by several structural interventions.



a) b) c)

Figure 96 – ‘Pombalino’ structural design and possible collapse mechanism (Mascarenhas, 1996): a) Tri-dimensional timber structure; b) After a seismic scenario the masonry walls would fall without putting at risk the interior timber structure; and c) Building whose exterior walls have fallen out-of-plane in the 1998 Azores earthquake without complete collapse (Cardoso, 2002).

The ‘pombalino’ reconstruction was extended for almost one century; hence structural variations are expected within the uniformity and regularity of the original designs. For instance, it was reported the discontinuity of the timber structure from the ‘frontal’ walls between floors.

From Figure 97 it is possible to realize that, in one hand, there is no connection between the vertical joists that would ensure the direct transition of loads and, on the other hand, the configuration of the truss structure changes between the floors. Moreover, above the doors' frame, the timber structure is only composed by vertical joists, interrupting once more the truss structure.

Figure 97 – Structural variations of the 'frontal' walls on the building.

Other insufficiencies can be found related with the original design of the buildings. While the ground floor is entirely made in masonry, the upper floors are formed by a much more flexible timber-masonry structure. This stiffness variation in height can actually be prejudicial to the global seismic behaviour of the structure.

The stone or brick masonry vaults and arches that compose the ceiling from the ground floors are very vulnerable to the movements on the support system (different settlements, for instance), which may also be amplified due to the vertical component of the seismic action. Conversely, the possible reduction of the gravity loads, caused by the subtracting vertical inertia forces, affect the lateral friction connection between the masonry stones or bricks, that tend to slide along the joints causing the collapse of the arc structure.

3.4.3. 'GAIOLEIRO' BUILDINGS

'Gaioleiro' buildings have been a major concern justified by the structural ambiguity that characterises these buildings. The connections between walls and between walls and floors are probably one of the main weaknesses of these buildings when subjected to seismic actions. Actually, there is the record of the collapse of several buildings as shown in Figure 98, though the collapse may also be related to their poor state of conservation.

Brick masonry walls, solid on the lower floors and hollow on the upper, and 'tabique' (with low bearing capacity) mostly compose the interior walls of this type of buildings. Firstly used for the division of the rooms, the 'tabique' walls assume a structural role on the 'Gaioleiro' building as they were copiously used on the upper floors of the buildings. This fact points another inadequacy of these buildings related with the variation of the type of interior walls used along the elevation of the building. As a result, the masonry walls are not continuously laterally supported by the interior structure and are therefore prone to out-of-plane failure.


Figure 98 – Collapse of 'gaioleiro' buildings (Lopes *et al*., 2008).

Other structural limitations are related with the increasing number of storeys and high ceiling heights. The spans also become more generous, leading to increasing deformations and consequent degradation of ceilings. The weak connections to the masonry walls and the lack of nailing fixation between the beams and the floor boards result on the low horizontal stiffness of the floors.

This feature is a major disadvantage as it influences the transmission of horizontal seismic actions (as inertial forces) to the resistant structure. Moreover, these buildings have a rectangular shape in plan, generating structures with disproportional dimensions and distinct behaviour when subjected to directional dynamic actions. As referred before, "*uniformity in plan is characterized by an even distribution of the structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building*" (EC8-1, CEN, 2004).

3.4.4. 'PLACA' BUILDINGS

The 'placa' typology of buildings has specific features that should be emphasized, particularly due to the combination of different structural materials and construction solutions. Whereas in old masonry buildings the gravity loads were mostly concentrated in the walls, on the 'placa' buildings, the self-weight of the concrete slabs becomes an important load to the structure. On the other hand, the reinforced concrete slabs act as horizontal diaphragms collecting and transmitting the inertia forces to the vertical structural system, gathering the whole structure in response to the horizontal seismic action.

Associated with this transition period, there is still the absence of specific design guidelines and problems regarding the durability of the concrete, so far seen as an eternal material. The concrete slabs were barely reinforced by steel rods, generally with only one layer of reinforcement for positive moments, and there is also no guarantee on the continuity between spans (Alegre, 1999). Moreover, the concrete used on the buildings has a low to moderate resistance class (C20/25 on the best cases) and was slight compact.

Conversely, the main weakness of these buildings is the insufficient number of vertical structural elements (Sousa *et al*., 2006). Firstly, vertical reinforced concrete elements were introduced as columns on the corners of the back façade walls (often with 4ϕ5/16'' on the corners and ϕ3/16'' stirrups generally 0.25 cm spaced), and only after extended to the front façade walls.

On the ground level, reinforced concrete beams were disposed guaranteeing larger spans on the building entrance and shops, while on the upper floors the interior structure was made by brick masonry walls and 'tabique' walls. The structural transition between the ground floor and the upper floors infers important stiffness variations, which can actually be prejudicial to the global behaviour of the structure. Moreover, the framed structure from the ground floor was slender because the seismic action is not taken into account on the design.

Is to be noted the existence of important structural weaknesses in the face of the seismic action, problems of excessive deformability of slender slabs, which were lightly reinforced and pathologies associated with steel corrosion and depth concrete carbonation.

3.5. STRUCTURAL MODIFICATIONS

Ancient buildings have experienced several structural interventions mostly motivated by different habits and needs of occupation. Some of the modifications ended diluting the original design of the buildings and compromising the structural safety of the constructions.

Occasionally these changes were made only to ensure the resistance to vertical actions, without taking into account the consequences on the resistance to horizontal actions. Below are a few examples of typical posterior interventions and their consequences on the seismic behaviour of the structures.

3.5.1. INCREASING NUMBER OF STORIES

According to Mascarenhas (1996), the structural regularity of the 'pombalino' buildings provided a similar performance of the construction within the block, which besides reinforcing the composition effect also gave them superior structural stability. Nowadays, several 'pombalino' buildings have more stories than the original buildings (five storey height including the attic) added at the end of the nineteenth century in response to an increasing population (Figure 99).

The new floors were built without a proper structural or foundation safety verification, or if there was it only considered the reinforcement of the structure to the increment of the vertical loads.

The direct consequence is the increase of the weight of the structure at the top of the building resulting on the increasing inertia horizontal forces and displacements during the seismic action. In fact, special attention must be provided to the external walls, as the added floors may not have been properly connected to them through interior perpendicular walls, which can lead to the collapse of the walls (Lopes *et al*., 2008).

Figure 99 – The ‘pombalino’ buildings were supposed to have five stories height, though a brief view of the ‘pombalino’ downtown gives a different perception.

On the conception of the ‘gaioleiro’ compounds of buildings, there were no restrictions with the buildings height generating quarters of buildings with several structural variations and an unpredictable global dynamic behaviour (Figure 100). This type of interventions brings important structural irregularities within the group behaviour related with the increment of the plan eccentricity between mass and stiffness.

a) b) c)

Figure 100 - 'Gaioleiro' buildings with different heights in its original conception: a) Rainha Amélia Avenue (now Almirante Reis Avenue) in 1908 (AFL); b) Tomás Ribeiro Street; c) São Sebastião da Pedreira Street.

Conversely, the introduction of basements induces important changes on the ground water level affecting the stability of foundation system. In fact, ‘pombalino’ buildings are supported on a timber grid laid on short timber piles, vulnerable materials when exposed to several variations of the water level.

3.5.2. ELIMINATION OF STRUCTURAL ELEMENTS

The most common modifications are related with the removal of structural walls regarding the enlargement of the existing rooms or the adaption to a different use (Figure 101). The discontinuous alignment of the walls might result on the excessive deformation of the overlying walls and floor that lose their original support system.

It is common the introduction of steel or reinforced concrete beams (pre-fabricated) in the place of the removed walls, supported on columns (steel or reinforced concrete) or simply supported on the adjacent walls (interior or exterior). Depending on the orientation of the wall, the actual bracing to the masonry façade walls might be compromised.

Figure 101 - Removal of interior walls which were replaced by steel beams and columns.

These adjustments introduce important weaknesses and stiffness variations on the building. As stated in the EC8-1 (CEN, 2004), "*the uniform development of the structure along the height of the building tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.*".

The elimination of a structural element (wall, column, beam, and floor) results on the overloading of the adjacent structural elements, the reorganization of the load pattern and a new structural balance for the structure. Conversely, the presence of 'frontal' walls on the 'pombalino' buildings is an important source of structural seismic resistance, which is simply excluded for the need of larger rooms.

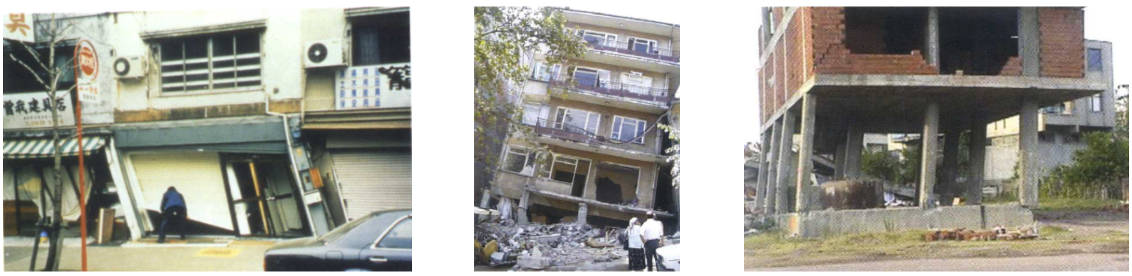
The existence of supply networks is recent in ancient buildings and sometimes these networks are introduced without any structural concerns. From time to time, the insertion of pipes ends up with the slash of the timber joists of the 'frontal' walls (Figure 102). As referred before, these walls are important for the support of the horizontal inertia forces, contribute to the energy dissipation capacity of the system and are fundamental to brace the masonry façade walls.

Figure 102 – Cut of a timber joist for the introduction of a pipe, which might compromise the purpose of the ‘pombalino’ cage (Lopes *et al.*, 2008).

The commercial occupation of the ground floor usually takes advantage of enlarged façade openings (Figure 103) through the cut of the vertical façade elements (piers). These actions generate artificial soft-stories which result on the support of a rigid element (upper floors) over weaker and flexible elements.

Figure 103 - Example of the interruption of the vertical façade elements.

Based on the effects of past seismic actions, it is possible to conclude that this type of structural irregularity is highly destructive (Figure 104). Thus, it is alarming the number of the actual cases on the ‘pombalino’ downtown area, easily detected by the contrast with the existing façade elements on the upper floors (Lopes *et al.*, 2008). There are even cases where different buildings were connected through openings on the side walls, a major weakness to buildings structural support.



a) b) c)

Figure 104 – Effect of the seismic action over soft-stories on the ground floor (Lopes *et al.*, 2008): a) Timber building damaged after the 1989 Earthquake in Kobe, Japan; b) Reinforced concrete building damaged after an earthquake; c) Reinforced concrete building damaged during the construction process.

3.5.3. REPLACEMENT OF FLOORS

The final period of 'gaioleiro' buildings construction is characterized by the progressive replacement of the timber floors by steel beams interconnected by ceramic bricks on the wet areas (kitchens and bathrooms). Other solutions involved the use of the timber floors as support to a layer of concrete, resulting on the degradation of the wood.

With the 'placa' buildings the use of reinforced concrete slabs is generalized to whole floor. In the meantime, several timber floors started to be replaced by reinforced concrete slabs or, more recently, by composite slabs combining steel plates and reinforced concrete. These new floors have rigid diaphragm behaviour in contrast with the flexibility of the timber-masonry structure, changing the distribution of internal forces and the balance of the structure. In addition, the weight of structure increases, affecting significantly the dynamic performance of the building.

As previously referred, the introduction of steel or reinforced concrete elements result in important changes on the performance of the buildings. Firstly, because these elements are usually replacing or strengthening original elements, introducing variations on the weight and on the stiffness of structure and secondly, these materials have a different mechanical behaviour when in comparison with traditional materials which might lead to several incompatibilities.

On 'gaioleiro' buildings it is common the transformation of the steel balconies of the back façade wall into galleries (Figure 105) or the replacement of the floor by reinforced concrete slabs (forced by the oxidation of the metal structure), contrasting with the original slender structures. There are other cases, where the steel staircases were also replaced by reinforced concrete solutions.

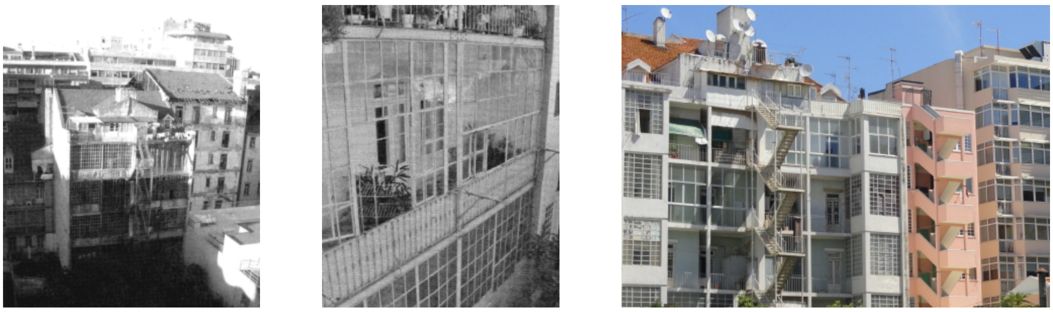


Figure 105 – Transformation of the back balconies into galleries (Appleton, 2005) and service staircase in reinforced concrete.

According to Alegre (1999) between the late 1950s and the beginning of 1970s several works were performed on Bairro de Alvalade under the responsibility of 'Federação das Caixas de Previdência' (FCP, the organization that financed part of the social project). The contract refers to the replacement of the existing wooden floors for the combination of ceramic blocks supported by concrete prestressed beams, which affected, in some cases, the resistant structure of the building as a result of the increasing loads involved.

3.5.4. REPLACEMENT OF BUILDINGS

There are several cases where the old masonry buildings were just demolished and replaced by new reinforced concrete structure, coexisting in the same compound buildings with different structural conceptions. A new structure to be built on the quarter should benefit the overall behaviour of the compound and avoid further vulnerabilities to the existing structures.

Nevertheless, these new buildings are usually much higher than the old masonry ones (Figure 106). In the best cases, the façade walls are preserved and just the interior is demolished; however, there are also cases where new storeys are introduced above the existing façade wall.

Figure 106 – New reinforced concrete buildings within the old masonry compounds of buildings.

As stated before, old masonry buildings were frequently built sharing the side walls. However, when a building is demolished the remaining side walls should belong to the existing buildings of the compound. Solutions like the one shown on Figure 107.c must be avoided.

a) b) c)

Figure 107 – Structural modifications: a) Preservation of the façade wall; b) Introduction of new storeys; c) Partial demolition of the side masonry wall to build the new reinforced concrete structure.

In general, the most frequent structural modifications are introduced combined with each other in the same building and in several buildings from the same block, changing its characteristic features and structural regularity. Certainly, some of these interventions compromised the building's structural stability, putting into question its reliability to withstand an earthquake.

3.6. PATHOLOGIES AND DEGRADATION

Ancient buildings have long exceeded their expected lifetime. The age of the buildings combined with the lack of proper maintenance actions affect the durability and the resistance of the materials and, consequently the structural safety of the building and its inhabitants. In addition to the progressive abandonment of the city centre, these factors instigated the present degradation of the old masonry buildings. There are more than 40,000 unoccupied buildings in Lisbon, representing 14% of the existing buildings. In 2001, it was estimated that 61% of the existing building stock need repair works, as 5% were in deep degradation (EPUL, 2002).

The majority of the pathologies occur on the envelope of the building (walls and roof), and might be divided into structural and non-structural anomalies. The structural anomalies affect either the masonry walls or the coating of the walls (plaster). The non-structural anomalies only affect the coating of the masonry walls (Figure 108 and Figure 109); however, these can eventually end up affecting the masonry walls and thus, the structural safety (Branco *et al*., 2011).

Figure 108 - Degradation of the façade wall of old masonry buildings.

a) b) c) d)

Figure 109 – Coating degradation (Branco *et al*., 2011): a) Paint peeling due to water infiltration; b) Plaster peeling due to raising water (capillarity); c) Efflorescence and cryptoflorescence; d) Parasite vegetation.

For instance, the degradation of the plaster from the exterior walls leaves the masonry unprotected and subjected to the action of the water, menacing with the binding properties of the mortar. Conversely, the degradation of the window frames and failures on the roof coating allow the infiltration of the water promoting the degradation of the interior timber structures.

Certain conditions become the cause of other pathologies, and therefore it is important to appreciate the pathological phenomenon as a whole in order to identify the starting point. Physical, mechanical, chemical or biological are the main source of pathologies; however, the cyclical atmospheric conditions (temperature and humidity variation) or the aggressive environment (air pollution, traffic vibration or earthquakes) also has an important contribution to the conservation of the building.

For example, the damage or collapse of buildings affected by earthquakes might not be currently visible. After the seismic action, the buildings may be affected by permanent damages that reduce the overall strength and increase the progress of deterioration. If the permanent damages are not corrected, the building may eventually collapse years after the earthquake.

According to Lopes *et al*. (2008), the disconnection between walls, the deformation and the cracks resulting from past earthquakes combined with the lack of a proper maintenance may be the main cause of the decay and consequent collapse of old masonry buildings. Moreover, the settlement of foundation system, the temperature or the humidity variation, introduce over the time additional stresses on the structural and non-structural elements that might not be perceptible.

Cracking, crush and disintegration of the masonry elements are usually related with the material low tensile strength and cohesion, structural modifications linked with the removal of interior walls without a proper support system or related with differential settlements of the foundation system. These factors can generate different displacements inside the wall, leading to the opening of diagonal cracks pointing to the settlement direction (Figure 110); or different displacements between walls leading to vertical cracks on the connection between perpendicular walls.

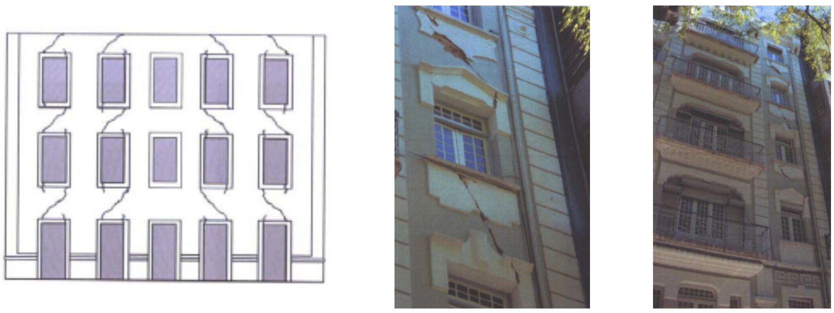


Figure 110 – Diagonal cracking due to differential settlements (Branco *et al*., 2011).

The inclination of the timber floors or lintels above the doors and windows also gives the indication of the settlement direction. Though, the opening of new cracks on the walls suggests that the settlements are still happening and, therefore strengthening measures should be taken on the structure and foundations. The presence of diagonal cracks in both directions (cross cracks at 45 degrees) on the masonry walls are usually connected with the effect of a seismic action, resulting from bi-directional ground acceleration (see Figure 87).

Located crush mostly happen due to excessive concentrated loads, for instance due to the introduction of steel or concrete beams (replacing interior structural walls) without a proper connection and reinforcement of the support system. As to the disintegration, the main causes are related with cyclical actions, such as temperature variation or moisture content, which cause the deterioration of the mortar and the decrease of the masonry walls resistance (Figure 111). For example, the light-shafts are usually an important source of pathologies. Built in ceramic bricks (most of the cases) or rubble stone masonry, these interior columns are badly illuminated and ventilated leading to important moisture problems.

Figure 111 – Anomalies on masonry walls: a) and b) Advanced disintegration of the element (Branco *et al.*, 2011); c) Moisture pathologies on a light-shaft.

The source of deterioration and deformation of the wooden structures are mainly related with the action of the water or with aging. The floors are often deformed on the middle of the spans, possibly due to the inadequate design of the elements (under-sized beams for the spans involved or overload) and as result of the fluency of the structures when loaded for several years (Figure 112.a).

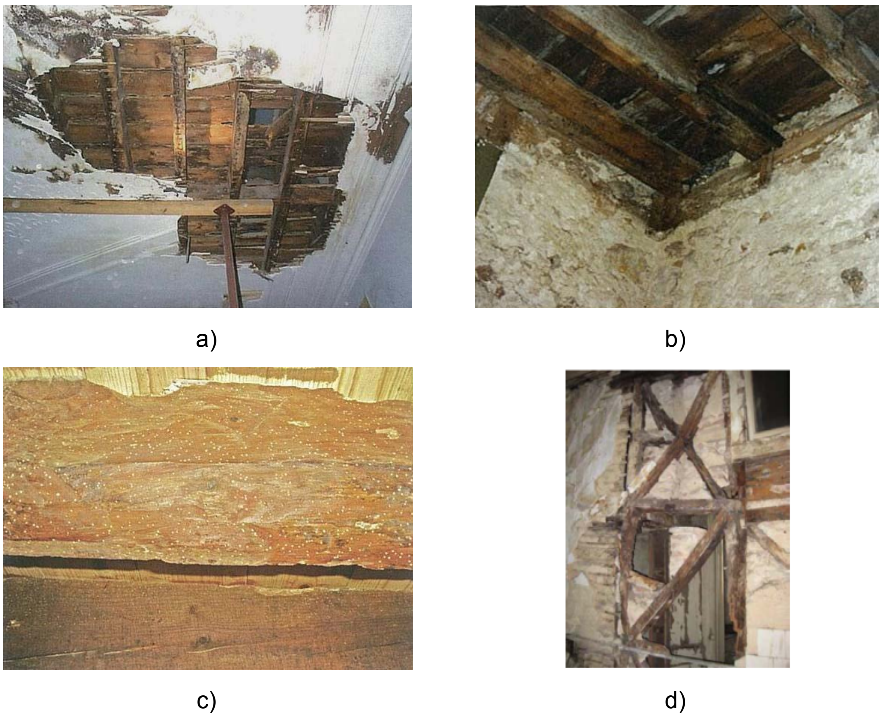


Figure 112 - Example of wooden degradation: a) Excessive deformation of the floor (Paiva *et al.*, 2006); b) Decay of the floor beams (Silva, 2007); c) Timber beam attacked by worms (Paiva *et al.*, 2006); d) 'Frontal' wall with lack of joist due to timber rot (Pena, 2008).

The deflection of the floors is usually followed by the vibration of structure, damaging the ceilings coating. In addition, the rotten of the beams instigates important vertical displacements and warp, especially in floors with the beams edges embedded on the masonry walls (Figure 112.b) or on the roof structure in result of coating failures and clogged gutters.

The presence of water on the interior increases the development of fungi and xylophages (termites and worms), which are also responsible for the degradation of the wood structures. The action of fungi (mould) is visible by the change of the colour on the wood and by the formation of a grid of contraction, consequence of the loss of material and resistance. The attack of xylophages is responsible for the reduction of the resistant section of the elements visible by the presence of small holes and dust (Figure 112.c).

On the 'frontal' walls, the reduction of the resistant section of the joist (and the lack of them) affects the connection between the elements, also helped by the corrosion of the nails, increasing therefore the panel deformability (Figure 112.d).

Originally, old masonry buildings did not have bathrooms, and for that reason some interior rooms were afterwards used for that purpose. First there was a tendency to create concrete slabs in these rooms, raising the floor level to make possible the introduction of pipes. Second, these walls and floors were not reinforced to accommodate these new function and loads. Moreover, the old timber floors were used as lost formwork being subjected to the mortar moisture, which makes them susceptible to the attack of insects.

The degradation of the steel balconies and stairs from the 'gaioleiro' buildings is very common (Figure 113). The oxidation of the metal structures is actually related with the initial treatment of the iron (reduced thickness of the protection) and with the lack of maintenance actions, due to the costs involved or due to the reduced visibility of these structures. Nevertheless, the current state of oxidation can cause failure by forcing apart adjacent elements or sections, invalidating the use of these equipments. In the worst possible scenario, these balconies can drag the back façade wall of the buildings into a global collapse.

Figure 113 – Oxidation of the steel structure.

On the 'placa' buildings, the reinforced concrete elements used were very slender and poorly reinforced. In addition to that, the covering concrete layer was very thin leaving the reinforcement vulnerable to the oxidation and corrosion. The expansion of the steel material results on the spalling of the concrete cover exposing the reinforcement (Figure 114).

Figure 114 – Reinforced concrete slabs and beams slightly compact concrete with a thin layer covering the reinforcement. The corrosion of the steel is common and leads to the spalling of concrete cover (Lopes *et al*., 2008).

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