Structural assessment of Companhia Aurífícia, a 19th century industrial building located in northern Portugal

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ABSTRACT

Companhia Aurifícia is located in Porto, Portugal, and was founded in 1864. It was a pioneer factory in the industrial production, casting, rolling and stamping of metallic objects and laboured for about 150 years, in areas as jewellery, manufacture of parts in silver and gold or the production and casting of various metals. In 1866, it began labouring in Rua dos Bragas, its present location, and in 2003 ceased all activities.

Companhia Aurifícia is an industrial complex including several buildings, all located in the same block. It is a precious example of the industrial architecture in Porto, where the still existent retaining walls, structures, machinery and decorative elements, make it one of the last examples of nineteenth century industrial life of the city.

The present work aims to evaluate the safety condition of one of the buildings included in this industrial complex, in order to propose the necessary strengthening interventions.

KEYWORDS

Industrial heritage, Portuguese industrialization, site inspection, material characterization, cast iron columns, timber trusses, masonry walls, safety assessment, strengthening.

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Introduction

1.1 Brief summary of *Companhia Aurifícia's history.*

Companhia Aurifícia is located in Porto, Portugal, and was founded in 1864. It was a pioneer factory in the industrial production, casting, rolling and stamping of metallic objects. Initially, it was located in one of the predominantly working neighbourhoods of Porto, known as Bairro da Fontinha, but in 1866, it began labouring in Rua dos Bragas, its present location. The company laboured for about 150 years, in areas as diverse as jewellery, manufacture of parts in silver and gold (sacred art, flatware, tea services and coffee, etc.) or the production and casting of various metals.

Companhia Aurifícia is also an industrial complex that includes several buildings, all located in the same block. It is a precious example of the industrial architecture in Porto, where the still existent retaining walls, roof and slab structures, machinery and decorative elements, make it one of the last examples of nineteenth century industrial life of the city.

In December of 2012, *Companhia Aurifícia* was ranked as part of the block of Álvares Cabral street (between Cedofeita Street and Republic Square) in the center of Porto and the entire block was declared of Public Interest. The existing buildings in this old area were constructed by families of middle and upper class with precious architectural features. The Portuguese government recognized the complex as an exceptional heritage and emphasized its historical value as the best-preserved example of a coherent industrial installation of the 19th and 20th centuries in the metropolitan area of Porto (PCM 2012).

In 2003, the company ceased its activity and all business was closed. At present, the buildings remain unused and only light maintenance activity has been performed to preserve their current condition.

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1.2 Objective of the present work

The present work aims to evaluate the safety condition of one of the buildings included in the *Companhia Aurifícia* industrial complex, in order to propose the necessary strengthening interventions. This building includes a combination of metallic, timber and masonry structural elements (Figure 1.a).

The developed study was subdivided into six steps as follows: historical background, geometrical survey, structural identification, damage survey and non-destructive testing, structural analysis, structural assessment and strengthening works.

2 Historical background

2.1 Industrialization in Portugal and in Porto

During the 19th century, Portugal experienced a period of reforms and growth. After the new constitution of 1820 and the "triumph" of liberalism, new manufacturing establishments were set up and, especially after 1835 and the industrialists' movements, the new leaders pushed the mechanization of the northern Portugal starting from the textile sector. The turning point for the city of Porto was the establishment of the new port in Matosinhos along with the economic development promoted by Fontes Pereira de Melo (the Portuguese prime-minister) known as *fontismo.*

In 1847, foundries established themselves in the city and the metallurgic sector started to grow. Casting iron became a resource of employment for many handcrafters. Bourgeois decided to invest in the sector involving also Brazilian high society, which had great esteem of Portuguese manufacturing.

Jewellery factories started, as *Companhia Aurifícia*, with the help of good handcrafters, carpenters and smiths. To protect the initial investment, the owners also established other related metal productions such as: nails, screws or wires for domestic use. Until 1900 the factory

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remained more or less the same, then the owners decided to buy new fields for future enlargements. During the Salazarian period (1932-1968), financiers and contractors accepted extensive bureaucratic controls in return of assurances of certain monopolistic privileges, particularly in the highly industrialized Porto, where industrial dynasties were allied by marriage with the traditional landowning families of the nobility. During the late 1950s new influential Europe-oriented industrials and technicians persuaded Salazar to enter in the European Free Trade Association (EFTA). Scandinavian countries took advantage of the cheaper Portuguese workers, in return of advanced technological knowledge. After the dictator's fall, in 1974, Portugal quickly fell into several years of negative growth. Many factories were closed or bought by bigger companies to annihilate their competition (Sampaio 1993). *Companhia Aurifícia* survived thanks to the cautious owners' management, but eventually, in 2003, closed its production.

2.2 Technological, architectonic and artistic aspects related to *Companhia Aurifícia*

Originally, *Companhia Aurifícia* was constituted by three buildings; during the years the complex was increased, finding nowadays numerous buildings that cover the whole plot with two main façades, one to the Rua dos Bragas and another one to Rua de Álvares Cabral, in Porto city (Pires 2004).

Passing the wall of the property, through the imposing wrought-iron gate, access is given to the patio and garden fronting *Companhia Aurifícia*. The spatial organization reflects the prototype of the era that is characterized by the construction of the factory itself, the house of the boss and managers, and sometimes the homes of the workers. So that, in the main position, the centre of the industrial site, one can find the office of the manager surrounded on the right by the building that contained the administrator office, with the main façade in Rua dos Bragas; remaining the original doors that allowed the entrance to the workers rooms (the jewellery ones) and also the stores with two little rails that connect the interior with the street. In the upper part, it is possible to find an open space with several windows that provided optimal conditions to the jewellers for the job. On the left hand, there is a big space with the machinery used to laminate lead.

Behind these three buildings, the complex expands in a kind of triangular shape, in which one can find pretty different buildings and spaces, that were designed looking for the functionality as the main purpose: depending on the machinery and the production system they needed, the spatial definition were different. Also, the whole complex has several interesting architectural details as arches, nice staircases and big windows that allowed the entrance of light and good ventilation (Sequeiros 2013). The factory buildings are characterized by architectural solutions marked by traditional typologies, mixed with some modern features. The spatial organization of some classical spaces - in galleries and courtyards, coexist with innovative solutions - the large spans, overhead lighting and use of iron columns very characteristic of the industrial architecture of the second half of the 19th century (Birkmire 1892) and (Burn 1877). Materials such as stone, masonry and wood, are used profusely.

Over the years, the company had to adapt to the market and paid attention to the new technology developments; in that sense they performed adaptations, remodelling and changing some sections and also, created new ones. Even so, most of the original machinery is still in operation, even when they used electrical energy; so the factory can be seen as a little museum and also as a life portrait of a factory of the 19th century.

This company is a good example of the production conditions of the Portuguese industry at the end of 19th century. Thanks to the successive administrations, who have been concerned in keeping part of the original machinery and architecture, an exceptional example of the metallurgical industry of the 19th century still exists.

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2.3 The importance of Industrial Heritage

The industrial heritage, as the Nizhny Tagil Charter (TICCIH 2003) written by the International Committee for the Conservation of the Industrial Heritage (TICCIH) exhaustively defines, is intended as the ensemble of remains of the industrial culture embodied by settings, infrastructures or buildings related to the productive activity. Since the earliest periods of human history the achievement of fundamental changes in producing objects and delivering goods or services, defined the transition from an epoch to another.

The Industrial Revolution is one of the most important turning-points of human development, and influenced in many different ways the technical, economic, social and also political aspects of human populations. Industrial buildings must be protected as well as towns and landscapes where they are. According to the Venice Charter (Venice 1964), the most significant and characteristic examples should be identified and maintained.

3 Geometrical survey, construction survey and state of conservation

The building has a very clear structural scheme that can be appreciated in the cross-section (Figure 1.b and Figure 1.d) and it is composed by several elements representing different materials and historical constructive techniques.

The graphical information available on *Companhia Aurifícia* was limited to a ground floor plan of the first construction phase and some drawings found in the database of Oporto Municipal Archive (OPM 2011) (15 building permits issued between 1873 and 1938). It is a modular building (with some irregularities) with a rectangular plan, in which the standard module is $13{,}20 \times 3{,}25\ \mathrm{m}^2$ (Figure 1.b). The total dimensions of the building are $44{,}20 \times 13{,}20\ \mathrm{m}^2$ with a maximum high of 9,50 m (Figure 1.c and Figure 1.d). It is composed by the ground level that occupies the whole surface of the plan; and a mezzanine in the first level (3,80 m) that runs parallel to the exterior walls and crosses perpendicularly in two points (Figure 1.c).

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3.1 Foundations

Given the difficulty in accessing the foundation, some assumptions can be done about them, thanks to several drawings found during the *historical survey*. The drawings found show continuous masonry (or concrete) foundations for walls; and punctual masonry (or concrete) foundations for columns. Considering the lightness and age of the building, it is considered that the damages observed in the structure do not seem to be related to foundations' problems.

3.2 Masonry walls

The perimeter of the building is composed by a 6,70 m tall single-leaf well-coursed granite ashlars wall, finished with lime mortar. Minor material decay, due to penetrating water, has been detected along the masonry walls, mainly localized on lime mortars, such as: chromatic alteration, algae colonization, deposits as adhesion of dust and oil from the industrial activity, detachments, partial loss of material, loss of adhesion/cohesion, and biological growth (Figure 2). Mechanical damage is not compromising the load-bearing walls, though minor cracks and deformations resulted from walls' modifications (opening and filling doors and windows along the years) have been detected.

3.3 Cast iron columns

Twenty-four cast iron columns (Figure 1.c) are disposed in pairs along the plan, and separated 6,75 m one from the other to act like intermediate supports, shortening the span between the granite walls (13,20 m). Their function is to shorten the trusses' span, acting as intermediate supports. Columns are 4 cm thick, characterized by a hollow circular shape. Junction vertical lines suggest they have been casted as two different halves.

It was possible to identify the origin of the columns, because the Maker's name and location was cast onto the element (in plaque at foot of column); unfortunately, the load capacity was not indicated. The columns were cast in the Massarellos foundry, another interesting industrial heritage case in Porto (Sampaio, 1993).

Visual inspection evidenced a pitted texture on the columns' surface and small manufacturing 'blowholes'. The columns are affected by a generally uniform corrosion that increases at the foot level, in places where rain water can accumulate. This causes iron layering and powdering.

3.4 Mezzanine

The mezzanine floor is composed by double 8×22 cm^2 timber beams supported by the bearing walls and iron columns and covered by a double layered structural timber floor. The timber floor is heavily damaged by penetrating rain water, especially in the western part of the building where a major hole is present in the roof. In the eastern part, the timber floor appears severely damaged, whereas the timber beams do not present such heavy decay conditions. A visual damage survey is presented at Figure 3.a and Figure 3.b. In both figures a direct relation between water entrance and rate of decay is clearly visible, showing a severe material damage at the floor level affecting the timber beam structure. This severe material damage is classified as "high material decay" in Figure 3.a and Figure 3.b.

3.5 Timber roof

The timber roof is composed by twelve Riga pine timber trusses (laying on the granite lateral walls and a pair of cast iron columns), purlins, common rafters, secondary purlins and ceramic tiles. The trusses look like king-post ones, but without struts and with three "tie beams".

Penetrating rain water is mainly affecting the trusses' supports, whereas visual inspection did not detect wood-boring beetles.

Visible deformations are affecting almost all trusses. These have been caused by an incorrect load transmission between the king-post and the tie-beam, increased by previous interventions that tried to solve the problem by filling the intentional void between the two elements and nailing the

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straps to the tie-beams. All trusses have been numbered and present similar loading and structural conditions, with the exception of Truss 9. This truss is a single span truss, only supported by the bearing walls, showing a more severe damage localized at the supports level and greater deformations.

The material decay survey is presented at Figure 3.c. The survey has been carried out by visual analysis considering the material rate of decay (low, medium, high). As previously explained, the main cause of damage is penetrating water, affecting the integrity and bearing capacity of the elements. The most affected zones are localized around the broken roof windows that are allowing rainwater entrance. In Figure 3.c losses of connections are also highlighted with “high loss of connection”, showing the localized points where the joint integrity is not guaranteed by the interlocking of the elements and the connection is weakened by a high water presence.

4 Material Characterization

4.1 Timber Truss and Timber Floor – *on-site* tests

The cross section dimensions were easy to measure, due to the fact that almost all the pieces have the same cross-section, except for the main rafter and the purlins. Estimation of timber properties begins with the identification of wood species. This task can be easily performed, thanks to the historical information is available (date of construction, region and wood species normally used in that period and region). As was found in the Building permits, the species used was Riga Pine, Larch wood. However, it was considered important to perform some tests and to compare the results with information available.

So, after the specie identification, tests were performed to obtain some results of wood density, strength and modulus of elasticity for comparison purpose.

The wood density is important since its mechanical performance is closely related to it. The wood moisture content was measured using an electronic hygrometer and two minor destructive tests, Resistograph® and Pilodyn, were performed to assess density and mechanical properties trough available correlations.

4.1.1 Hygrometer

This test measures the wood moisture content and was carried out in the same location of the Pilodyn, in order to correlate the results later, and a mean result of 12,6% was obtained.

4.1.2 Pilodyn

The Pilodyn releases a spring into the material, transforming the elastic potential energy into impact energy. This way, the penetration of a metallic needle with 2,5 mm of diameter is measured and the depth is inversely proportional to the wood density (Gorlacher 1987). Moreover, it can be used for correlating the density and the elastic properties with the depth reached with the needle (resistance to superficial penetration) (Kasal *et al* 2004).

Tests were performed in all the main tie-beams, in three different points each. For an estimation of the mechanical properties of the timber, some generic formulas (established for oak) were applied to the outputs resulting from the Pilodyn test (Feio 2005). The test was performed in twelve different locations of the timber structure and the values of penetration depth varied from 9 to 13 mm. Therefore, the average value of density obtained was 426 kg/m^3, the average value of bending strength, f_m, was 30,7 MPa and the average value of modulus of elasticity in bending was 9.328,7 MPa.

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4.1.3 Resistograph

This test method provides relative graphical information concerning the integrity, density, and/or soundness of wood members within a structure. It consists of a small drilling machine, which is run by a battery. The drilling machine is held perpendicular to the test material and drills. The details about the material structure can be studied based on the graph obtained. It measures the resistance to penetration and based on it, the density of the material can be obtained (Rinn 1992). The tests were carried out in two different beams, one very damaged and another not damaged (no degradation found on the surface). The results obtained are as shown in Figure 4 for comparison purposes.

The test reported in Figure 4.a was carried out at the point where the material was more degraded (connection between Beam 9 and wall). It shows that the there is no uniformity within the material, and that its resistance is very low. Here, the timber is highly degraded.

The test reported in Figure 4.b was performed in the same corresponding position of the second beam. Here, it is possible to see that there are only two parts where the resistance measured is almost zero i.e. at the position of the connection between the timber materials. This implies that the testing timber material is made of 3 parts. Apart from the gap between the three parts, the material has almost a uniform resistance.

4.2 Laboratory tests on an element collected on-site

It was not possible to take a lot of wood material from the structure and therefore only one piece was collected. The available material was a triangular piece of the truss with dimensions of $31 \times 20 \times 7{,}5\ cm^3$. Due to the natural variability of timber, such sampling cannot be considered representative. However, even if the number of samples is very reduced, the goal was to provide information on the moisture content and density and also to check if strength values of clear

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wood fall within the expected range or to restrain the quality of wood in a certain visual strength grade (Cruz 2008).

As was done before, the same MDT's were performed into the specimens made from the sample collected on-site: Hygrometer, Pilodyn and Resistograph. A total of 30 points of measurement were considered for the Hygrometer and Pilodyn tests performed in the laboratory. The mean result obtained for moisture content was 14,3%. The average value of density obtained is equal to 365 kg/m^3, that lead, based on the correlations proposed by Feio (2005), to an average value of tensile strength equal to 21,1 MPa and an average value of modulus of elasticity of 5.909,4 MPa.

The Resistograph confirmed that the cross section has a uniform density and resistance to be drilled; so it is possible to say that the section is not damaged and there are no voids or knots.

In addition to the MDT performed on-site, ultrasound (UPV) measurements were made at laboratory on the piece recovered from the timber roof. UPV is based in the measurement of the ultrasonic wave velocity between a transmitting and a receiving transducer. It is possible to correlate wave propagation with physical and mechanical wood properties: detect cracks, knots, decay and grain deviation; density and modulus of elasticity in bending (Sandoz *et al* 2000) (Kasal *et al* 2004).

The sample was prepared (smoothing and cleaning the surfaces) and tests were carried out considering both direct and indirect methods. Using the correlations purposed by Feio (2005), and based on 30 measurements, a modulus of elasticity of 1.032,7 MPa and 13.714,0 MPa was obtained for the directions perpendicular to the grain and parallel to the grain, respectively.

Using the sample collected from the structure, bending tests were carried out following the specifications of ISO 3133-1975(E). According to this standard, a total of 6 specimens were considered, with the dimensions of 20 mm × 20 mm in cross-section and 310 mm length. Before the tests, each specimen was measured and weighted. After tests, each specimen was dried and then measured and weighted again, allowing the determination of the density and moisture content of each specimen, according to ISO 3131-1975(E) and ISO 3130-1975(E), respectively.

The setup used for the bending tests consists in a simply supported beam with a free span (L) of 240 mm where the load is applied in the midspan, on the radial surfaces. The load was applied continuously (load control) with constant test speed of 782,2 N/min, providing the specimens failure in approximately 90 s (1,5 min). A strength class of C35 was assumed in order to define the test procedure.

Six specimens were successfully tested. An average value of 66,22 MPa was obtained for the bending strength, together with an average value of 5,4 GPa for the modulus of elasticity. The results presented were corrected for a reference moisture content value of 12%, according to ISO 3133-197(E).

Again, it is important to point out that bending tests were made on clear specimens and therefore, a high bending strength value was expected. On the other hand, the value obtained for the modulus of elasticity obtained (5,4 GPa) is more realistic and represents better the age and state of degradation of the timber specimens.

4.3 Tests on Cast Iron Columns

Hardness is generally defined as a material's resistance to permanent deformation under an applied load. Macro-hardness is an easy and quick property to measure, which can be correlated to tensile strength, wear resistance, ductility and a number of other physical characteristics. This is especially useful in the assessment of existing structures since direct tensile strength tests are destructive.

The Leeb hardness test was performed and the results obtained were directly converted to a Brinnell scale; as it is easier to establish comparisons with results existent in the bibliography. The Leeb test is based in a dynamic rebound hardness method. The test was performed in three columns, with around thirty measured points in each of them. As the tested surface should be smooth, first of all, the surface of the columns was carefully prepared. Correlating the results with the conversion tables conveniently provided in ASTM E140 (2005), a tensile strength value of 270 MPa was obtained.

An iron sample was collected from one of the columns and was subjected to X-ray diffraction analysis. X-ray diffraction is a tool used for identifying the atomic and molecular structure of a crystal, in which the crystalline atoms cause a beam of incident X-rays to diffract into many specific directions. The results show three basic elements into the composition: mostly Iron (Fe) and also Iron Oxide Hydroxide ($Fe+_3O(OH)$) and calcium Carbonate ($Ca(CO_3)$) in the part of the sample that was in the exterior surface of the column.

4.4 Masonry Walls

There weren´t any tests performed on the masonry walls, as it was considered from visual inspection that their current state is stable and safe: no important cracks or signs of severe decay that could affect the integrity of the wall were found.

5 Structural Analysis

A complete understanding of the structural and material behaviour is necessary for any conservation or strengthening project. The numerical models to be developed should try to consider all the information collected through visual inspections on the real state of the structure, damage survey, laboratory tests and *in situ* tests.

An elastic structural analysis was chosen to be performed on the structure, as there were no major problems found during the inspection of the building. Since the structure is single storied and in a good condition, it was considered adequate to use the loading values proposed in the current codes in order to have a proper evaluation on the safety condition for current loading values. This approach facilitates the possibility of finding a new use for the building and guaranteeing its future existence.

The heritage structure in analysis is located in the zone with smaller seismic activity in Portugal. Portugal is divided into 6 seismic zones and the city of Porto falls in zone 1.6, which is the safest seismic zone. The seismic evaluation is not considered in the analysis performed, because the permanent loads acting in the building roof and mezzanine are rather small. It was considered that the dead weight loads, the live loads and the wind loads have more unfavorable effects.

5.1 Global Structural Behaviour

The original structural system behaviour of the trusses is complex to understand as they do not correspond to a normal king-post truss structure. After analysing them carefully, it is possible to say that they exhibit some degree of improvisation, with some basic conceptual errors, and also that they have been altered through the years. In a first step, it can be assumed that the truss was designed as a simple king-post truss with a tie beam in the upper part. Later, probably due to some problems (perhaps an excessive bending of the main tie-beam), a second tie-beam was added between the original ones. This assumption is based in the fact that the tie-beam added is double, so, the execution could be done without removing the original structure. Moreover, little columns were placed between the "new tie-beam" and the cast iron columns in order to add a transversal beam for the support of machinery. Nowadays, almost all the trusses are working in an erroneous way, because the king-post is transmitting loads to the tie beam, which suffers bending. Also, probably some interventions carried out with a lack of knowledge, aggravated the

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problem: the spaces between the straps and the tie-beams were filled with some material and the straps were nailed to the tie-beams. The result of these interventions is an erroneous geometry and an inadequate load transmission (Figure 5).

Another point important to study are the joints, that frequently have some kind of damage (metal corrosion, timber splitting or crushing) and original defects like missing plates or fasteners, distances of fasteners, or gaps between elements that should be in contact.

For example, the connection between the masonry wall and the timber truss is weak. The trusses enter in the wall no more than 15 centimetres and, in most of the cases, the head of the beams is in really bad condition due to humidity (see Figure 2).

The connections between the timber elements are made with bolts. It was impossible to find a structural meaning for the position of the bolts that in one case are near to the upper part of the beam and in other, are the opposite. This is another clue that may lead to think that the second tie-beam is posterior. Apart from the position, they seem to work properly, but present some minor cracks.

5.2 Structural models of the roof trusses

In order to analyze the global behavior of the structure, two dimensional numerical models were studied using SAP 2000 (2012) with linear static analysis. Geometrical survey was carried out to simulate the real conditions in the model. The components that needed to be considered were the timber trusses and the cast iron columns. Their structural properties were evaluated during the geometrical survey. Finally the different loads and their combination were considered acting on the structure.

As the roof structure is modular, quite regular and works in a principal direction, a simpler 2D analysis of the most loaded trusses or the trusses with the worst statical disposition was considered adequate.

5.2.1 Material characteristics considered in the numerical models

As presented before, experiments were conducted in order to identify the different mechanical properties of the elements. A modulus of elasticity of 5,4 GPa was experimentally found for timber in bending. The main section properties of timber are of single or double layer, with 8×22 cm^2 and 8×30 cm^2, as represented in Figure 5. The metallic columns consist of a hollow section of varying cross section with a base diameter of 20 cm and 12 cm at the top. The thickness of the section is 4,0 cm.

The different loads considered are the dead load of the structure, dead load of the roof tiles and the timber supporting it, service load and wind load, determined according to EN 1991-1-1 (2005) and EN 1991-1-4 (2005).

5.2.2 Wind loads and Load Combinations

The wind load calculations for the structure were carried out according to EN1991-1-4 (2005) and were complemented with the corresponding Portuguese National Annex. The necessary values were calculated based on the location of the building and considering the code requirements. Two loading cases were considered for wind action according to EN1991-1-4 (2005), at 0º and at 90º, as shown in Figure 6a.

A first analysis is performed considering the 2D SAP 2000 model and taking into account the loads coming from the self-weight of the structure (multiplied by a safety factor of 1,35), dead load coming from tiles and timber substructure (safety factor of 1,35) and service load (safety factor of 1,50), corresponding to Load Combination 1 (LC1).

A second analysis is performed taking into account the combined actions of the self-weight of the structure (multiplied by a safety factor of 1,35), dead load of tiles and timber substructure (safety factor of 1,35), service load (safety factor of 1,50) and all different wind load combinations specified in the code (safety factors of 1,5 and 0,9 that are applied, depending on the situation analysed), corresponding to Load Combination 2 (LC2).

5.2.3 Analysis and Results

All the structure is analysed, but Trusses 8, 9, 10 and 11 are studied more in detail due to higher loadings applied. Truss 9 is also studied because it has different supporting conditions: it is just supported on the walls, without any columns and is subjected to deterioration. A 3D overview of the roof and the various trusses is shown in Figure 6.a.

The load combinations were considered according to EN 1991 (2005). An envelope was considered in order to show the maximum and minimum stress values acting on each element, for all load combinations.

Two load combinations were considered for the analysis: self-weight of the structure and service load for the first combination and the second combination with self-weight, service load and wind load. Truss 8 and Truss 9 are submitted to higher bending moments and higher axial forces than all the other trusses. Axial force, shear force and moment distribution of Truss 8 and Truss 9 resulting from Load Combination 1 are shown from Figure 6.b to Figure 6.i and the corresponding maximum values are reported in Table 1.

The rafter of Truss 9 is submitted to a maximum axial compressive force of 61,40 kN and the rafter of Truss 8 is submitted to a maximum axial compressive force of 37,90 kN. Similarly, a maximum shear force of 99,00 kN and a bending moment of 1,81 kNm are acting in the main rafter of Truss 9, for LC1. As explained above, the higher forces and bending moments in Truss 9 result from the absence of intermediate columns.

Table 2 presents the results obtained for the mezzanine structural elements.

5.3 Structural models of the masonry wall

Finite element analysis was carried out on a specific part of the masonry wall that was submitted to various interventions along time. This part of the wall initially had window openings that were transformed in door openings and later transformed in arch openings (Figure 7). As this part was subjected to many interventions, phase analysis was chosen to be carried out in DIANA software (2012) to study the behavior more accurately. The reaction loads obtained in the SAP 2000 model were used as inputs for these models, as the roof trusses are supported by the masonry walls of the building.

5.3.1 Modelling

The geometry was created in AutoCAD software and imported to DIANA, where all the material types, material section properties, loads and boundary conditions were assigned to the respective elements. Phase analysis was performed to incorporate the geometrical changes on the structure over time. The first phase is called the "window phase" (original construction); in the second phase, the windows were converted into doors and finally, in the third phase, two windows were combined to form an opening in the form of an arch. This last phase (current state) is called the "arch phase".

The walls were created with eight-node quadrilateral curved shell elements of type ‘CQ40S’ so that the results could be analyzed in outer, middle and inner layer. The different load cases considered are the self-weight of the structure, dead load coming from the roof (obtained from SAP model), dead load of the smaller roof (this smaller roof is positioned on the exterior side of the wall and the loads are obtained from SAP model), self-weight of the mezzanine floor and dead load of the machines in the mezzanine floor.

Different combinations of density, modulus of elasticity and Poisson’s ratio values were assigned to the DIANA numerical model in order to check the influence of the masonry properties in the structure´s global behavior. The variation of these parameters was performed since the original data of the structure was not available and there was no scope to carry out field experimental testing on the masonry walls. Reference values of density, modulus of elasticity and Poisson’s ratio of 23,0 kN/m^3, 1,0 GPa and 0,20 were selected and then some comparisons were established by varying these parameters and evaluating how they influence vertical deformation.

The different values considered for each model are listed in Table 3. Linear phase static analysis was considered to take into account the stress redistribution that result from the alterations imposed to the walls and from parametrical analysis performed. Modulus of elasticity was varied between 0,5 and 3,7 GPa, Poisson's ratio ranged from 0,08 to 0,20 and density varied between 20 and 27 kN/m^3. These values were based on the bibliographic survey on existent masonry walls.

5.3.2 Analysis and results

In the “window phase”, the maximum deflection considering serviceability limit state was 1,20 mm. The maximum principal tensile stress was 0,13 MPa and the maximum compressive stress was 8×10^{-3} Pa, as presented in Figure 7. These values show that the stresses acting on the masonry wall are very low and the structure is completely safe in the first phase (window phase).

In the second phase (door phase) the maximum deflection measured in the numerical model is 1,29 mm which is close to the deflection measured in the first phase (1,20 mm). Similarly, there are no major changes in the tensile stress distribution of the structure in comparison with the first phase. This is due to the fact that existing minor tensile stresses are concentrated in the mezzanine floor and in the corners of the window above the mezzanine floor. As a result, converting the windows into doors in the lower floor did not affect the tensile stress distribution in the structure. In case of compressive stresses there is an increase in the stress due to the changes in the structure. The maximum compressive stress is 0,14 MPa.

In the third phase, the “arch phase”, the deflection increased up to 3,25 mm, corresponding to an increment of 270% with respect to phase one. The maximum principle tensile stresses reached 0,50 MPa at the corners of the windows and the compressive stresses went up to 0,18 MPa. In general, the range of the tensile stresses is around 0,1 MPa, except for the stress concentration at the corner of the windows. Compared to the first phase there is a decrement in the tensile stress from 0,13 MPa to 0,10 MPa.

The distribution of stresses was studied with monitoring points (MP), positioned in different layers of the wall. There were no major differences between layers, since very low stresses are acting on the structure.

In order to better study the deflections in different phases, some nodes were chosen to be plotted and compared. The position of nodes and the corresponding deflection are shown in Figure 8.a. The deflections obtained for different combinations of density, modulus of elasticity and Poisson's ratio are shown in Table 3. It can be concluded that the variation on Poisson's ratio had a minor effect on deflection in the different phases, the increase in density led to a small increment of deflection (around 14.4%) and that modulus of elasticity had a major effect with 6,50 mm deflection for 0,5 GPa and 0,80 mm deflection for 3,70 GPa.

As discussed above, the results obtained from phase analysis show that for the "arch phase", the maximum deflection is 3,25 mm, the maximum tensile stresses are in the range of 0,1 MPa and the maximum compressive stresses are less than 0,1 MPa. These low values of stresses are due to the minimal loads acting on the structure.

6 Safety Assessment and Strengthening Solutions

When assessing the safety of a structure, there are many unknown aspects taking part and therefore increasing the difficulty of a realistic evaluation. In the last 10 years, the structure of *Companhia Aurifícia* probably experienced an important degradation due to lack of maintenance and severe weathering. During the six months of the survey here presented, it was possible to detect the increasing rate of its deterioration, underlining the urge of conservation works.. As it will be shown, most if the elements are safe at the moment, but some of them need to be strengthened as soon as possible.

Strengthening actions that will be proposed are based on the main principles of conserving historical buildings: compatibility (material, physical, aesthetical etc...), removability and retreatability (all the strengthening are totally removable with minimal loss of material) and minimal intervention (the main objective is to maintain the structure as much as possible in the original state, without removing, but adding if necessary, or reinforcing), as recommend by ICOMOS (2001). Every intervention is designed to be as simple as possible, fast and cheap.

The elements analysed are the ones with the higher ongoing deterioration. As was already discussed, it is not the purpose of this work to carry out massive replacements or alterations on the present structure. Only and exceptionally, it is necessary to propose a more invasive conservation technique, if no other option is feasible.

6.1 Roof timber elements

Timber mechanical characteristics were previously obtained from testing. Tests made on a sample collected from the structure indicated a density value of 494 kg/m^3 and a modulus of elasticity in bending of 5,4 GPa. On the other hand, an average value of 66,22 MPa was obtained for the bending strength from tests, as already presented. However, taking into account that the specimens evaluated have non-structural dimensions and have no defects, a reference value of 40 MPa was considered for the characteristic value of the bending strength, assuming then, that the timber belongs to the C40 strength class (EN 338 2009). A cross-section reduction of 30% was considered in every element in order to represent as much as possible the degradation process.

As previously presented, two different trusses were taken into consideration and two different loading combinations were applied to them (LC1 and LC2). The safety assessment of timber truss elements shows that none of them actually needs strengthening. In fact, all the elements in all the possible combinations are safe for the reduced section. For this reason the main recommendation is to clean them and act on the internal environment, by lowering the interior relative humidity and increasing ventilation.

Figure 9 shows the locations of connections that need strengthening interventions and details the interventions that can be considered in the strengthening process. In Figure 9.c, Detail 3 shows a fast and non expensive way to restore the original gap between the king-post and the tie-beam (presently lost due to the rafters movement and deformation). It is suggested to cut the king post and place a new steel collar. The bedding of Truss 9 is partially lost and is currently in a bad condition. In the past, there was already an unsuccessful attempt to strengthen this element. At the present moment, this is probably the most problematic deterioration of the whole structure. In Figure 9.d, Detail 4 proposes to prop the truss and open the bedding to clean the cement placed in the past and to dry of the stone. Then, it is suggested to place a new granite element after reshaping the destroyed stone to permit a safe support. Over the new granite stone a timber wedge will avoid water to affect the timber again and a free space for ventilation will be kept. A new steel L shaped element will be placed to strengthen the support, as it has been proven that the timber in that zone is partially lost. To conclude, a new water system will avoid new water entering inside the structure.

In Figure 9.e, Detail 5 proposes a steel strap to confine the timber element and avoid crack's opening. The ridge in the greater span presents a previous strengthening intervention, consisting on a steel plate at the mid span of the element. Considering the results obtained with the SAP model and the visual inspection, the ridge seemed to be in a possibly dangerous situation, requiring further evaluation. In fact, the resulting deflection was 57 mm in a maximum allowable deflection of 30 mm. A strengthening technique that consists on the span shortening is proposed in Details 1 and 2 (Figure 9.f and Figure 9.g). This option is fast, non invasive and easy to remove. In case of a shorter span, as the one present in the factory in most cases, no strengthening is necessary as the calculated deflection is under the maximum value required.

Some of the timber roof elements present high deformations and an altered surface. In order to estimate their rate of degradation, the real deflection of the elements was taken into consideration, with the help of photographic support and visual inspections. This method is used to give qualitative values for evaluating elements' degradation. The worst condition detected is in Purlin 1 located between Trusses 8 and 9. The element is a simply supported beam 4,52 m long and has a cross-section of 80×200 mm^2. The rate of degradation was evaluated considering the real deflection visible on the element and comparing it with the deflection that results from the model. In order to match the obtained deflection values, the purlin cross section area was reduced of 33%. The maximum deflection in the middle-span was calculated considering short term and creep deformations as proposed by EN 1995-1-1 (2004). The maximum deflection is higher than acceptable, especially when long term effects are taken into consideration, as presented in Table 2. However, the same beam results totally safe for bending parallel to the grain and shear stresses. A similar situation is obtained for Purlin 2 (Table 2). Considering the same element type (Purlin 1) in the main span (3,30 m), with same cross-section, the maximum deflection results acceptable and does require neither repair nor strengthening actions. Secondary rafters, although smaller and more deteriorated, do not present problems.

6.2 Mezzanine timber elements

The factory presents a timber mezzanine made of a principal structure of coupled beams and a secondary structure that consists on two layers of timber boards, positioned perpendicularly to one another. As shown in Figure 3.a and Figure 3.b, the board structure is highly deteriorated, principally due to the presence of water. At present, it is not totally safe to walk on these boards. It is possible to observe important deformations and parts of the floor that already fell because of the machinery weight. Apart from the timber boards dead weight, a major dead load is present on the mezzanine floor that corresponds to the machineries with an estimated weight of 2,0 kN/m^2.

Mezzanine beams are considered as simply supported elements with 3 m length and mechanical characteristics that were already discussed. The full cross-section of the coupled elements is 160×220 mm^2 and a reduction area of 30% is applied. The result is a safe element for bending and shear but with a deflection in the middle-span higher than the maximum acceptable. Loads applied include a variable action of 3 kN/m^2, considering that this area is intended to be used in the future. As the mezzanine is an accessible platform, it is preferable to avoid unpleasant deformation, and therefore a strengthening action is proposed. New boards should be used, reproducing the pattern that already exists. These boards should present a thickness of 4 cm for the first layer and 2 cm for the covering. The chosen wood specie should resemble as much as possible the mechanical, physical and aesthetical characteristics of the previous one, for this reason the use of a Pine quality is recommended. This new surface will increase the strength of the whole floor bringing back under the limit the deflection rate. Moreover, it is also recommended to connect the two coupled beams in order to make them work together and double the resisting cross-section (Figure 8.i).

6.3 Cast iron columns

Cast iron columns are very important structural elements in *Companhia Aurifícia* as they give support to the roof trusses. According to the results obtained in the SAP 2000 model, the maximum loads are achieved when the wind load is also acting on the structure. The maximum values are obtained in the columns that support Truss 8. Considering the experimental results obtained in Part 4.3 and reference values presented in (Bussel, 1997) and (Bates, 1991), a reduced tensile strength of 240 MPa and a modulus of elasticity of 90 GPa were used in the numerical models and in the safety evaluations. The column has a mean external diameter of 200 mm and a mean thickness of 40 mm. The free buckling length is equal to 6,34 m which coincide with the total length. The safety factors are taken according to EN1993-1-1 (2004), in all the verifications.

The resistant bending moment in *y* direction (as the cross section is symmetric, in *z* direction it would be the same) is about 164 kNm and the acting bending moment, $M_{Ed,\ support}$ is only 0,26 kNm.

Buckling verification was also performed considering the element consists of a hot laminated hollow section and summing the contribution of the axial forces and the bending moments and considering the columns buckling length. The safety verifications are performed according to EN 1993-1-1 (2004). It results that the columns are safe and do not require any strengthening or repair action, apart from surface cleaning and painting.

7 Conclusions and Recommendations

Companhia Aurifícia is not only a perfect example of how was the industrialization process in Portugal and the social organization during the 19th century, but also until today with its closure. By studying *Companhia Aurifícia*, it is possible to establish the technical features of a period. In order to bequeath this portion of history, the idea is to perform some minimal interventions to stop the decay process of the building (avoiding future major interventions) and propose a brief maintenance plan. There is a responsibility to maintain this legacy for the future generations.

The repair, strengthening and upgrading of old buildings can represent a large parcel of a building contractor's activity. An important point that should be understood by the population is that the continuous maintenance can avoid major expenses.

During almost 150 years, the analysed building was working properly, and since 2003, when the industrial activity was closed, the decay process accelerated. Little problems, such as water leaks, are easy to repair immediately, but can lead to future irreparable damages if nothing is done, as is currently occurring in *Companhia Aurifícia*. Up to the moment when this work was developed, the only structural element proposed for complete replacement is the timber mezzanine; however, if the process continues, timber trusses or cast iron columns may follow the same path.

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In the authors' opinion, water is the element causing the major part of the damages founded in the building, until now. Considering this, all the solutions should initially focus in avoiding the water entrance. Performing some specific interventions in the timber roof trusses, repairing and replacing part of the roof, repairing the mezzanine structure and replacing the mezzanine floor will guarantee that the building can stand up for many years. Buildings need to be used. Without use, the little problems that naturally appear during the service life of every building are not going to be noticed.

7.1 Recommendations

Some minimal interventions can be done, so that the building can be used, such as:

- cleaning the ground floor with a water pressure machine;

- remove all the tiles from the roof for make an inspection of the upper part of the wall; - repair the parts when it is necessary, repair the sky lights and substitute the tiles that are broken (the rest of them could be replaced again);

- a system of gutters should be designed so that rain water does not flow over the facade, avoiding erosion of joint mortar, detachment of render and paint or biological colonization;

- the environmental conditions (temperature and moisture content) should be controlled in order to protect the timber from biological attack;

- cleaning the mortar and removal its most degraded parts. These parts must be replaced with lime mortar in order to protect the stone (not for aesthetics reasons);

- cleaning and coating the cast iron columns;

- inspection of the mezzanine elements and substitution of the timber floor;

- repairing and replacement of the handrail elements;

- repair and treatment of the carpentries when needed;

- repair of the connection between the wall of the building and the roof of the annex building.

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