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ANALYSIS OF THE SEISMIC VULNERABILITY OF A MASONRY BUILDING

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Dedico esta dissertação à minha mulher, à minha filha e aos meus Avós

Homens com principalmente quatro materiais construíram as construções mais antigas e esses materiais duraram milhares de anos para fins de construção. Tratava-se de terra (de onde provêm tijolos, por exemplo), pedra, madeira e fibras naturais de plantas. Com estes quatro elementos, os homens foram capazes de construir construções espetaculares, algumas das quais ainda estão de pé hoje em dia.

(Rodrigues & Ferreira, 1991)

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Resumo

A verificação da segurança sísmica dos edifícios existentes requer uma abordagem meticulosa que engloba vários fatores-chave, tais como a definição de ações sísmicas adequadas, a modelação precisa da estrutura, a utilização de métodos de análise adequados e a realização das verificações de segurança necessárias. No entanto, a análise de edifícios de alvenaria existentes, com as suas características únicas, vulnerabilidades e complexidades associadas a um conhecimento técnico limitado sobre este tipo de construção, apresenta um desafio maior em comparação com estruturas construídas mais recentemente aderentes à legislação aprovada.

Este artigo centra-se no desenvolvimento de uma análise de vulnerabilidade sísmica para um edifício de alvenaria do século 19 construído no estilo pombalino, localizado em Lisboa. Foram utilizados métodos de análise não lineares utilizando o programa de cálculo automático TREMURI, de acordo com as recomendações específicas descritas no Anexo C do Eurocódigo 8, Parte 3, juntamente com informações complementares do Anexo Nacional e publicações nacionais informativas.

A análise foi realizada permitindo a modelação tridimensional e a avaliação sísmica através de análises estáticas lineares e não lineares. O modelo foi calibrado com base em testes no local realizados, especificamente neste edifício. Foi realizada uma verificação de segurança abrangente, considerando o estado limite final de danos graves.

Os resultados deste estudo fornecem informações valiosas sobre a modelagem e a vulnerabilidade sísmica dos edifícios de alvenaria do século 19 em Lisboa, lançando luz sobre seu comportamento estrutural e potenciais fraquezas. Através da utilização de técnicas de análise não lineares e da adesão às disposições relevantes do código, obteve-se uma compreensão abrangente da resposta do edifício às ações sísmicas. As soluções de reforço propostas visam aumentar a resiliência do edifício e mitigar potenciais riscos associados a futuros eventos sísmicos.

Esta investigação contribui para o campo da engenharia sísmica, abordando os desafios específicos associados à análise de edifícios de alvenaria existentes. Os resultados destacam a importância de empregar métodos avançados de análise e considerar as características únicas das estruturas históricas para garantir avaliações precisas de seu desempenho sísmico.

Abstract

The verification of seismic safety for existing buildings requires a meticulous approach that encompasses several key factors, such as defining appropriate seismic actions, accurately modelling the structure, employing suitable analysis methods, and conducting necessary safety checks. However, the analysis of existing masonry buildings, with their unique characteristics, vulnerabilities, and complexities associated with limited technical knowledge on this construction type, presents a greater challenge compared to more recently built structures adhering to approved legislation.

This paper focuses on the development of a seismic vulnerability analysis for a 19th century masonry building constructed in the Pombalino style, located in Lisbon. Nonlinear analysis methods were employed using the automatic calculation program TREMURI, in accordance with the specific recommendations outlined in Annex C of Eurocode 8, Part 3, along with complementary information from the National Annex and informative national publications.

The analysis was carried out by enabling three-dimensional modelling and seismic evaluation through both linear and non-linear static analyses. The model was calibrated based on on-site tests conducted, specifically on this building. A comprehensive safety check was performed considering the ultimate limit state of severe damage.

The results of this study provide valuable insights into both the modelling and the seismic vulnerability of the 19th century masonry buildings in Lisbon, shedding light on its structural behaviour and potential weaknesses. By employing nonlinear analysis techniques and adhering to relevant code provisions, a comprehensive understanding of the building's response to seismic actions was achieved. The proposed reinforcement solutions aim to enhance the building's resilience and mitigate potential risks associated with future seismic events.

This research contributes to the field of seismic engineering by addressing the specific challenges associated with analysing existing masonry buildings. The findings highlight the importance of employing advanced analysis methods and considering the unique characteristics of historic structures to ensure accurate assessments of their seismic performance.

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1. INTRODUCTION

In 1755, a catastrophic earthquake followed by a major tsunami struck the capital of Portugal causing severe damage to the city. The event completely destroyed the heart of the city, which was set on a valley area close to the river Tejo and is composed of a shallow layer of alluvial material. The disaster required an urgent solution. The Prime Minister at the time, Marquis of Pombal, was put in charge of rebuilding the city and restoring it back to normality as fast as possible. He delegated to a group of engineers the development of a structural solution that would guarantee the required seismic resistance of the buildings. Based on the know-how of that time and on the empirical knowledge gathered from the buildings that survived the earthquake a new type of construction was created, which is now generally referred to as Pombalino construction.(H. Meireles et al., 2011a)

Ever since, these buildings have suffered several changes and even new constructions systems have been adopted and mixed to the existing ones, downgrading their structural resistance, bringing a sensation that the 1755 earthquake has been forgotten. For that reason, new laws and regulations came into effect, bringing assurances that buildings within certain parameters are assessed and that rehabilitation (when required) will be undertaken following the rules.

With the entry into force of “Decreto Lei No. 95/2019, of 18 July,” which establishes the regime applicable to the rehabilitation of buildings, and the legal diplomas contained in it, it introduces a very significant change to the structural engineering of buildings in Portugal, not only for the use of EUROCODES following the normative dispatch 21/2019, of 17 September, as well as the situations which are now mandatory to have a structural assessment following “Portaria nº 302/20219, of 12 September of Habitation offices. (Candeias et al., 2020a)

In this dissertation, an existing Pombalino construction was assessed following the current laws and regulations. The building under analysis is a building of year 1871, which suffered some alterations since. A global vulnerability analysis was carried out, using a software developed for the purpose by STA DATA, called TREMURI®. Following a survey and various visits to the site, it was possible to ascertain the structural element’s types, dimensions, and current conditions. To carry out the assessment there was a need to combine the existing structure with the structure that would be modelled in Tremuri, making it compatible by calibrating it. To calibrate the model, the frequencies of the structure modelled in tremuri, would approximately be matched with the structure obtained on site, meaning that some characteristics of the materials of the structure would change accordingly.

This dissertation is divided into the following chapters:

Current chapter (chapter 1), presents the theme of the dissertation, its aim and how the document is organized.

Chapter 2 brings the context of state-of-the-art where will be presented the current legislation as it applies, the parameters for a seismic vulnerability assessment, the limit states, the analysis methods, macro elements models and a quality method.

In Chapter 3, the description of the case study and how it fits within the context can be bound. In this chapter, the main construction systems and the evolutionary construction in Lisbon and Portugal, together with its main components, are presented

Chapter 4 involves the modelling of the structure, where there is a brief description of the modelling programme used (TREMURI®), the structural assessment and therefore the structural elements definition, by floor, walls and the loads considered in line with the EUROCODES as it applies. In this chapter, it is also included the definition of the seismic action and the importance of it when providing the assessment to the structure.

Chapter 5 is one of the main chapters of this dissertation, as it complies the vulnerability analysis the application and the results, based on all the information contained in the previous chapters. It starts with the calibration of the model, pushover analysis, capacity curves as a result of the pushover analysis and the N2 method following NP EN 1998-1:2010 and finally the results obtained.

The final chapter, chapter 6, brings the conclusions on the results obtained as well a range of possibilities to use in structural strengthening to the retrofitting of the building, in line with the current regulations.

2. STATE-OF-THE-ART

Context

The exteriors of the “Pombalino Buildings are protected by law, but their interiors and therefore much of their unique construction are not, and many of them have been altered beyond recognition. Action is still needed to protect those which remain. (Dias et al., 1996)

The most sensible aspect of carrying a refurbishment of an existing buildings, is the rehabilitation of its structure. The assessment of the existing structure is generally a very complex task when looking at the methodologies adopted for new builds, that are different. Not only that, the cultural value of the building must be respected when strengthening is required.(Guia CIB REEH-140322P, n.d.)

Both statements above are still valid nowadays, as well not taken on board in some of the conservations/refurbs or change of use, carried in old masonry buildings. We are living in times that a lot of research has been done and that a justification to not do things properly is difficult to find. Universities and individuals have spent and dedicated several days to understand principles and the reason behind the constructions systems adopted after the great earthquake of 1755 that affected Lisbon.

The majority of the research done to date is available to anyone who wants and shows interest to this matter. It is important to note that a lot of efforts have been put into place for rehabilitation of old and historical buildings that in its essence represents the cultural identity of a country. There were some international meetings that had some relevance to Portugal and mainly Lisbon, in 1995 the “Carta de Lisboa 1995” that fills in recommendations to keep the historical value of a building when adapting a building to the new use or function and in 2000 the Krakow recommendations, that bring more emphasis to the conservation side of the buildings, where interventions within certain parameters are acceptable to improve the safety of the building.

A lot more is still to be done and in 2019 a new legislation is implemented in Portugal (DL 95/2019, 18th of July), and this regulates that within certain terms, buildings that are required to have a seismic vulnerability assessment adopting the Eurocodes (DL 21/2019, 17th of September) as it applies, and as result the elaboration of a strengthening project.

Legislation

The DL95/2019 defines the terms in which building extensions, making alterations and/or re-building, are subject to the assessment of seismic vulnerability, as well the situations where a strengthening project is required. Therefore, it is mandatory that the seismic assessment report establishes the resistant capacity of the building when using the seismic action defined in NP EN 1998-3:2017 applying the local conditions as defined, as always as one of the following is verified (Candeias et al., 2020)

- Clear signs of degradation of the building structure;
- That as a result of the works and change of use, the structural behaviour is altered;
- Where the works affect more than 25% of the gross construction area, including demolitions and extensions
- When the construction costs exceed at least 25% of the construction cost for an equivalent brand new similar building.

When the report concludes that the building does not satisfy the safety demands to 90% of the action defined in NP EN 1998-3:2017, a strengthening project must be done under the same norm (NP EN 1998-3:2017)

Seismic vulnerability assessment

Following the legislation and as a result the Eurocodes, when carrying a seismic vulnerability assessment it is important to define some parameters that are strictly necessary. Below is the explanation of those:

Seismic action

In Portugal two types of seismic action are considered as indicated on the national annex of NP EN 1998-1:2010, since earthquakes may have epicentres in different zones, with different propagation law. Seismic action type 1 designated as “faraway” refers to earthquakes with epicentres in the Atlantic area. In other hand the seismic action type 2 designated as “near” refers to earthquakes that have epicentre in continental area or Azores archipelago. (NP EN 1998-3:2017, 1998)

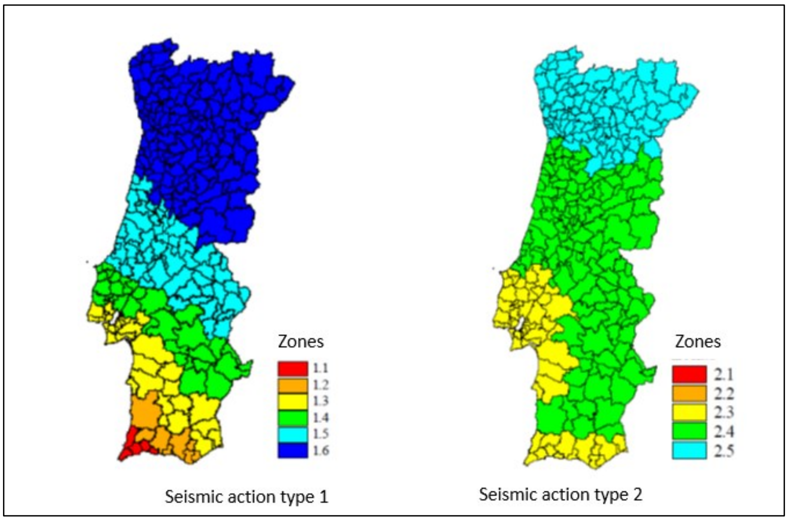

Figure 1 – Seismic zones for the different types of seismic actions (NA-3.1.2(2) of NP EN 1998-1:2010)

Response Spectrum

The elastic response spectrum represents a seismic movement in a certain point at ground level. To determine the response spectrum (either for seismic action type 1 or 2) for any part of Portugal, it is important to understand the ground make and identify it by comparison to table 3.1 of NP EN 1998-1:2010 . As shown on Figure 1 the location of the building is important to determine the seismic zone which it relates to.

Table 1 shows the reference peak ground acceleration a_{gR} based on the seismic zone which is being analysed. This table can be used as a reference as there is the annex NA.I which gives a most accurate figure council by council.

Table 1 – Reference peak ground accelaration a_{gR} (m/s^2)

Seismic action type 1		Seismic action type 1	
Seismic zone	a_{gR} (m/s^2)	Seismic zone	a_{gR} (m/s^2)
1.1	2,5	2.1	2,5
1.2	2,0	2.2	2,0
1.3	1,5	2.3	1,7
1.4	1,0	2.4	1,1
1.5	0,6	2.5	0,8
1.6	0,35	–	–

After defining the reference peak ground, it will be possible to determine the value of the soil parameter (S). The Soil parameter is given by the following:

For agR < 1: S=Smax (1)

For 1m/s^2 < a_g <4m/s^2: $S=S_{max}-\frac{Smax-1}{3}(ag-1)$ (2)

For a_g >4m/s^2: S=1 (3)

The design ground acceleration on type A ground (ag) is given by the following expression:

Ag=γI x agR (4)

Where:

- γ_I is the importance coefficient shown on Table 3.

Accordingly to NP EN 1998-1:2010 S_{max} is either given by tables NA 3.2 or NA 3.3 depending on the seismic action being analysed.

Table 2 – showing tables NA 3.2 (left) and NA 3.3 (Right) (NP EN 1998-3:2017, 1998)

Ground Type	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,1	0,6	2,0
B	1,35	0,1	0,6	2,0
C	1,6	0,1	0,6	2,0
D	2,0	0,1	0,8	2,0
E	1,8	0,1	0,6	2,0

Ground Type	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,1	0,25	2,0
B	1,35	0,1	0,25	2,0
C	1,6	0,1	0,25	2,0
D	2,0	0,1	0,3	2,0
E	1,8	0,1	0,25	2,0

In table 4.3 of NP EN 1998-1:2010, the class of importance of each building is presented which is determined based on its intended use. This classification allows to obtain the corresponding importance coefficient γ_I as shown on Table 3 for each seismic action.

Table 3 – importance coefficient γ_I (Table NA. II of NP EN 1998-1:2010)

Importance class of the Building	Seismic Action Type I	Seismic Action type 2	
		Portugal	Azores
I	0,65	0,75	0,85
II	1,00	1,00	1,00
III	1,45	1,25	1,15
IV	1,95	1,50	1,35

Following the design lines as specified on NP EN 1998-1:2010 the elastic horizontal acceleration response spectrum is given by the following expressions:

$$0 \le T \le TB:\ Se(T) = a_g \cdot S \cdot [1 + T/TB \cdot (2{,}5 \cdot \eta - 1)$$

$$TB \le T \le TC:\ Se(T) = 2{,}5 \cdot ag \cdot S \cdot \eta$$

$$TC \le T \le TD:\ Se(T) = 2{,}5 \cdot ag \cdot S \cdot \eta \cdot (TC/T)$$

$$TD \le T \le 4s:\ Se(T) = 2{,}5 \cdot ag \cdot S \cdot \eta \cdot (TC \cdot TD/T^2)$$

Where:

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

Se(T) – is the elastic response spectrum

ag – is the design ground acceleration on type A ground as shown in equation (4)

TB – Is the lower limit of the period of the constant spectral acceleration branch

TC – is the upper limit of the period of the constant spectral acceleration branch

TD – is the value defining the beginning of the constant displacement response range of the spectrum

S- is the soil factor

η – is the damping correction factor with a reference value of η=1 for 5% viscous damping (see (3) of 3.2.2.1 of the NP EN 1998-1:2010)

Level of acknowledgement of the building and confidence factor

There are three knowledge levels established on NP EN 1998-3:2017, which are limited knowledge (KL1), normal knowledge (KL2) and integral knowledge (KL3). These levels of knowledge are fully determined based on the quantity and quality of the information obtained in regard to geometry, structure layout and type of materials used. The Annex C of NP 1998-3:2017 provides complimentary guidance in what information should be gathered. (Candeias et al., 2020a) (Candeias et al., 2020a)(Candeias et al., 2020)

The level of knowledge about a building plays a crucial role in determining the appropriate method of analysis and the corresponding confidence factor, as indicated in on Table 4.

Table 4 – Relation between levels of knowledge, analysis methods and confidence factor (adapted from (Candeias et al., 2020b)

Level of Knowledge	Analysis method	Confidence factor (CF)
Limited knowledge	Linear analyses method (LF, MRS)	1,35
Normal knowledge	Linear and non linear analyses methods	1,20
Integral knowledge	Linear and non linear analyses methods	1,00

The confidence factors serve to adjust the average values of material properties obtained through in-situ inspections or/and from responsible researches. When determining the capacity of structural elements, the average properties of its components are divided by the corresponding confidence factor achieved. When defining the capacity of the existing structural elements to compare with the demands on these, its average properties values must be multiplied by the confidence factor. (Candeias et al., 2020)

Limit state and class of importance

The assessment of seismic performance of existing buildings involves evaluating specific limit states corresponding to certain return periods return of the seismic action. These limit states are defined based on the structural damage state, considering factors such as strength, rigidity, ability to withstand horizontal and vertical loads, displacements, and damage to non-structural elements. The selection of return periods is guided by the goal of achieving appropriate levels of protective measures when combined with the identified limit states. These concepts are identical used in NP EN 1998-1:2010 for the design of new buildings, but with the necessary adaptations to make the assessment and rehabilitation of existing structures.(Candeias et al., 2020)

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The Limit states are defined as below according to (NP EN 1998-3:2017, 1998)

State limit of Imminent collapse (NC). The structure is severely damaged, with reduced lateral and residual strength and stiffness, although the vertical elements maintain the ability to withstand vertical loads. Most of the non-structural components collapsed. Important permanent displacements are observed. The structure is close to collapse and probably would not resist the action of another earthquake, even if of moderate intensity.

Limit State Severe Damage (SD). The structure exhibits significant damage, with some residual lateral strength and stiffness, and the vertical elements are able to withstand vertical loads. The non-structural elements are damaged, although the partitions and the filling elements have not been damaged outside the plane. Moderate permanent displacements are observed. The structure can withstand moderate-intensity seismic replicas. A repair of the structure is probably not economic.

Limit state of Damage limitation (DL). The structure presents slight damage, having maintained its structural elements without significant incursions into the plastic regime and preserving its characteristics of strength and rigidity. Non-structural elements, such as partitions and filling elements, may have distributed splits and their repair is economical. Permanent lateral allocations are despicable. The structure does not require any repair.

Table 5 summarizes the requirements performance to be verified in Portugal in terms of the period of return and limit state for each of the importance classes.

Table 5 – Requirements performance in terms of the period return and limit state for each importance classes (adapted from (Candeias et al., 2020)

Return Period	Probability of Exceeding	Limit State: Damage Limitation (DL)	Limit State: Severe Damage (SD)	Limit State: Imminent colapse (NC)
73 years	50% in 50 years	Only classes of importance III and IV	-	-
308 years	15% in 50 years	-	All classes of importance	-
975 years	5% in 50 years	-	-	Only classes of importance III and IV

It is very important to note, that depending on the state limit being verified as shown on Table 5 and to carry a vulnerability assessment according to the NP EN 1998-3:2017, the reference peak ground acceleration obtained in Table 1 will have to be multiplied by the factor as show on table NA.I of the national annex of NP EN 1998-3:2017 and represented below on Table 6.

Table 6 – Factor values to multiply to the reference peak ground acceleration to obtain the maximum acceleration to apply NP EN 1998-3 (adapted from (Candeias et al., 2020)

Limit State	Seismic actio type 1	Seismic action type 2: Portugal	Seismic action type 2: Azores
Imminent colapse (NC)	1,62	1,33	1,22
Severe Damage (SD)	0,75	0,84	0,89
Damage Limitation (DL)	0,29	0,47	0,55

Seismic assessment

The seismic assessment of existing masonry buildings is based on the procedures included in the NP EN 1998-3 : 2017(Annex C) and corresponding to Portuguese National Annex which establish the performance requirements and compliance criteria for existing buildings subjected to a certain level of seismic action. (Bernardo et al., 2021)

There are three methods that can be used for the seismic assessment of a masonry building with flexible floors. Method I relies solely on geometric parameters, while Method II incorporates both geometric and material mechanical properties. On the other hand, Method III is the reference method, involving comprehensive numerical analysis. Additionally, Method III is employed when either method I and/or II cannot be applied (Bernardo et al., 2021)

Table 7 shows the principles and criteria that will assist in choosing the appropriate method for conducting a seismic assessment of an existing building. Analysing the table, Method III imposes the fewest restrictions. However, it is important to note that a significant number of buildings can be adequately assessed using Method I and Method II, without necessitating the more extensive and exhaustive Method III.

Table 7 – General principles and criteria to apply Method I to III (Table adapted from (Bernardo et al., 2021)

Criteria of applicablility	Method I and II	Method III
Masonry type	Traditional masonry	all
Admissible Mechanisms	In Plan	In Plan *
Flooring structure	Flexible	All
Importance class	I and II	All
Number of storeys	Up to 5 Storeys	no limit
Building area	up to 350m^2	no limit
Building type (i.e terraced/detached...)	Detached or Terraced	omitted
Structural regularity	Yes	optional **
Interaction between buildings	See note***	no restriction
Local geotechnical conditions	Restricted to soil type A,B and C	In Line with NP EN 1998-3:2017
Foundation type	Rigid	omitted
State limit checked	Severe damage (SD)	Damage limited (DL), Severe damage(SD), Near colapse (NC)

* the actual version of NP EN 19983:2017 does not provide colapse mechanisms out of plan
** depending on the type of analyses adopted in line with NP EN 1998-3:2017
***see figure xx

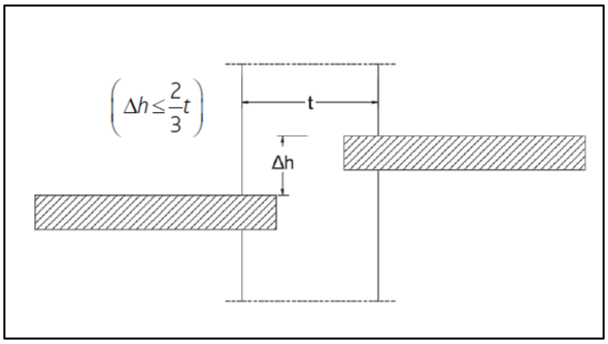


Figure 2 – Constructive requirement between buildings to apply the rapid assessments (adapted from (Bernardo et al., 2021)

Figure 2 represents one of the conditions to apply Method I and II in the assessment of an existing masonry building. For terraced buildings, the alignment between floors should not exceed two thirds of the party wall thickness. If this condition is not met, the reference method (Method III) must be used. (Bernardo et al., 2021)

Method III (Reference Method)

For the case study presented in this dissertation, Method III will be used due to the presence of irregularities between floors with adjacent buildings.

Within Method III there are five methods of analysis that can be used to assess the seismic actions on existing buildings, however the approach by coefficient of behavior "q" is not applicable in masonry buildings. The assumptions on which their application is based, in particular the energy dissipation capacity of the structure assigned to new buildings (3.2.2.5(1), (2) and (3)P of NP EN 1998-1:2010), may not be properly secured in existing masonry buildings. (Candeias et al., 2020). Hence, in this study, will be discussed two linear methods and two linear methods as per below:

i) Lateral forces (LF) linear analysis
ii) Modal linear analysis by response spectrum (MRS)
iii) Nonlinear static linear analysis (pushover)
iv) Nonlinear temporal dynamic analysis

Linear Methods

(i) Linear analysis by lateral forces and (ii) Modal Linear analysis by response spectrum (MRS):

In order to apply these methods, it is necessary to verify a set of additional conditions for masonry buildings, as provided for in Annex C to NP EN 1998-3:2017. It should be noted that these methods should be used with great caution as they are simplified methods, based on the linear elastic behavior of the materials, but they can serve to identify reinforcement needs and to analyze the structure with the introduction of the necessary reinforcements.

To make this analysis it is necessary to make the separation between dubious and fragile materials since they must be interpreted differently independently of the results of the analysis.

Looking at the structural elements as to their capacity *(Ci)* and taking into account the requirement in the same elements based on the demand of the structural element *(Di),* according to the conditions of applicability of linear analysis (4.4.2 and 4.4.3 of NP EN 1998-3:2017) a ρi relationship is made between these two components:

$$\rho_i = \frac{D_i}{C_i} \tag{5}$$

Where:

Ci - Average values of the properties of materials and in case of vertical elements takes the value of axial stress due solely to vertical loads

Di - Demand

For "dubious" elements the $Di_{requirement}$ may be superior to the $Ci_{capacity}$ in a linear analysis, which is permissible since the dubious elements have some deformation and energy dissipation capacity when they reach the give

$$\rho_i = \frac{D_i}{C_i} > 1 \tag{6}$$

When an analysis is made to ρi superior to the unit (ρi> 1) of the primary and dubious seismic elements, the lower *(ρ_{min})* and largest *(ρ_{max})* limits should be taken into account:

$$\frac{\rho_{max}}{\rho_{min}} \leq 3 \tag{7}$$

The imposition of this limitation is justified by the very high values that can be obtained from this, which would represent a great dispersion in the exploitation of the ductility of its elements and in turn such situations, may not be well evaluated. In Portugal the recommended value is 2.5.

On the other hand, masonry structures, in these structures that have an essentially fragile behavior, require the use of methods appropriate to the particularities (4.4.1(5) of NP EN 1998-3:2017). In this way the $Ci_{capacity}$ in all elements of the structure should be higher than the corresponding Di requirement, obtained from the analysis for the seismic combination:

$$\rho_i = \frac{D_i}{C_i} \leq 1 \tag{8}$$

However, the use of this condition should be taken into account, since the conformity criteria must be applied to all structural elements (2.2.1(3)P of NP EN 1998-3:2017), regardless of the method of analysis used (4.4.2(1)P note 2 of NP EN 1998-3:2017

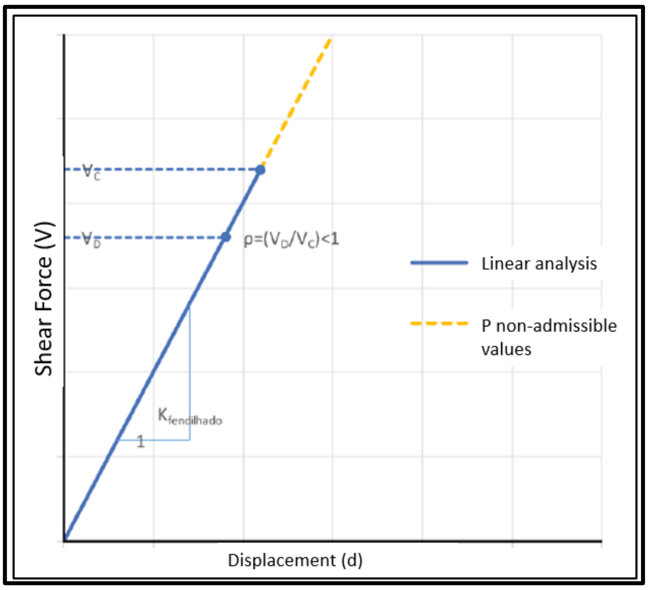


Figure 3 - "fragile" element with a requirement lower than the capacity in relation to transverse effort.

Annex C to NP EN 1998-3:2017 imposes a set of additional conditions for linear methods to be applied. These conditions are related to the regularity of the layout of the walls in plant and their continuity in height, the rigidity of the floors in their plane, the resistance of their connections to the walls and their continuity. Conditions are also imposed on the differences in stiffness between walls and on the characteristics of the masonry latels included in the model (C.3.2 of NP EN 1998-3:2017).

Once the checks are made, linear methods can be applied as described in the relevant parts of NP EN 1998-1:2010 (4.4.2 And 4.4.3 of NP EN 1998-3:2017), in which for this case the response spectrum to consider is the elastic response spectrum (Se(T1)).

For the static method (LF) you have:

$$F_b = S_e(T_1).m.\lambda \tag{9}$$

Where:

F_b - cutting force at the base

$S_e(T_1)$ - ordered from the calculation spectrum for period T1

m - total mass of the building above the foundation

λ – correction factor, the value of which is equal to 0.85 if T1<2Tc and the building is more than 2 floors, or 1 in other cases.

For the multimodal method (MRS) one has (4.3.3.3 of NP EN 1998-1:2010):

$$F_{bk} = S_e(T_k).m_k \tag{10}$$

Where:

k- Number of modes considered

F_{bk} - Cutting force at the base corresponding to the number of modes considered

$S_e(T_k)$ - horizontal elastic response spectrum of acceleration to the surface of the ground for a number of modes

m_k - effective modal mass corresponding to a number of modes

Nonlinear methods

The conditions of the linear methods indicated in section 4.4.1 (C3.3 of NP EN 1998-3:2017) do not apply to non-linear methods as these are more general, although more complex and according to Table 7, require a higher level of knowledge of the building.

(iii) Nonlinear static linear analysis (pushover)

In this analysis, at least two vertical lateral load distributions should be used in accordance with EN NP 4.4.4.2 1998-3:2017:

- A "uniform" distribution based on lateral forces proportional to the mass regardless of height (uniform response acceleration)
- A "modal" distribution, proportional to the lateral forces corresponding to the distribution of the lateral forces determined in the elastic analysis.

Once the distributions are applied, you can calculate the capacity curve (relationship between the transverse effort at the base and the control offset:

- The capacity curve is determined by pushover analysis for control offset values between zero and the value corresponding to 150% of the target offset (4.3.3.4.2.3 of EN NP 1998-1:2010). The target displacement defines the seismic requirement determined from the elastic response spectrum according to 3.2.2.2 of EN NP 1998-1:2010

(iv) Nonlinear temporal dynamic analysis

In line with 4.4.5 of NP EN 1998-3:2017, it must be applied the procedure indicated in NP EN 1998-1:2010. This type of analysis requires additional information about the behavior of the masonry elements under post elastic cycles of loading and unloading, to which is not provided any guidance.(Candeias et al., 2020b)

Structure modelling (equivalent bars model /finit elements/ macroelements)

In this section there will be a brief description of some numerical models used for masonry buildings:

Equivalent Bar models

The façade walls are modeled through linear pieces that simulate the nimbus and the lintels. The connecting "nodes", given their usual dimensions, are simulated by means of rigid sections, with an appropriate length, located at the ends of the bars that bind to the node.(Candeias et al., 2020a)

Finite elements models

The walls are reduced to their medium plane and modeled through triangular or quadrangular surface elements. The selection of finite element density and appropriate maximum and minimum dimensions depends on the overall characteristics of the walls, such as dimensions, presence of openings, interconnections, and floor connections.. Similarly, the floors are also modeled by means of surface finite elements either with homogenized geometric properties or eventually combined with linear parts to represent floor beams. The constitutive models assigned to finite elements aim to simulate the bending and shear behavior of the walls. These numerical models are generally better adapted to linear analysis methods although the resulting forces from the elasticity theory must be further processed to obtain the generalized forces *N*, *V and M in* the structural elements, nimbus, lintels, nodes and walls. These forces are required for conducting the necessary safety checks on the structure and its elements. These numerical models can also be used in non-linear analysis methods although, in this case, it is necessary to incorporate damage formulations or plasticity at the level of mechanical behavior assigned to the finite elements in order to simulate the condition of weak tensile strength of masonry.(Candeias et al., 2020)

Macro elements models

In these models, the structure is discretized into a series of macro elements, each simulating a distinct part of the overall structure. There will in principle be different macro elements to simulate nimbus, lintels, nodes, and walls, each of them with a behavior model. Floors can be modeled as deformable or non-deformable diaphragms.

These models are commonly used in calculation programs developed specifically for nonlinear analysis static and/or dynamic masonry buildings. The results are usually presented in terms of capacity curves, transverse stress at the base as a function of the displacement at the top and the damage to the various structural elements (walls, nimbus, lintels, knots) represented in elevations to be able to evaluate the state of the structure.

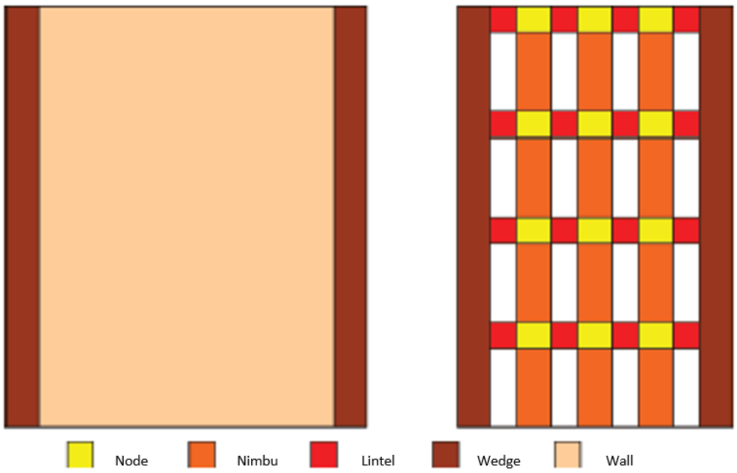


Figure 4 – Example of zones to consider in walls for masonry building. On the left, party walls and on the right main façade. Adapted from (Candeias et al., 2020b)

3. CASE STUDY

On the 1st of November 1755, the city of Lisbon was hit by a severe earthquake, followed by a tsunami and consumed immediately afterwards by a fire which lasted several days. As a result, these events caused substantial damages to the city. (Dias et al., 1996)

Figure 5 – Lisbon Downtown before the 1755 disaster (Picture from Lisbon Museum to the public)

The Prime minister at the time, Marquis of Pombal, was put in charge of rebuilding the city and restoring it back to normality as fast as possible. He delegated to a group of Engineers the development of a structural solution that would guarantee the required seismic resistance of the buildings. Based on the know-how of that time and on the empirical knowledge gathered from the buildings that survived the earthquake, a new type of construction was created, which is now generally referred to as Pombalino construction. (Meireles et al., 2011)

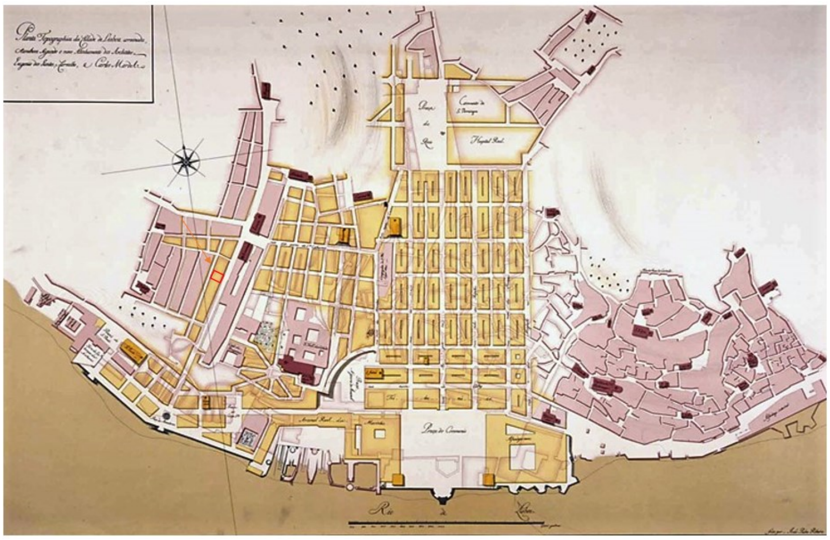


Figure 6 – Proposal Plan for down town Lisbon from Marquis Pombal Team (Image by Ribeiro, João Pedro MC. GRA.0035)

Figure 7 – Lisbon downtown 3D image Nowadays (Image from 3D Cad Browser)

Henceforward, there will be reference to Pre-Pombalino buildings as the buildings that were built before the earthquake of 1755, the “Pombalino” buildings as the construction solution found to rebuild the city of Lisbon by the team headed by Marquis of Pombal, and the “Gaioleiro” buildings that appeared from 1880 onwards as a result of the real estate interest and demand due to the expansion of the city. (Appleton, 2005) The case study presented in the document in Chapter XX is located in one of the named two worst cities in Europe where the impact of an earthquake could be really damaging, affecting the population due to the quality of its construction, by Professor Mario Lopes (interviewed by (Coelho, R., 2017).

There are three important key facts that have contributed to the change in the construction types in Lisbon. These are: the devastation caused by 1755 Earthquake when most of Lisbon turned to rubble the demand of property due to the growth of Lisbon as an economic and commercial hub centre) and the advent of reinforced concrete as a construction solution.

As shown on Figure 8, nowadays there are various types of construction in Lisbon which have been adopted based on the evolution of materials and due to the demand for more houses. (Dias et al., 1996).

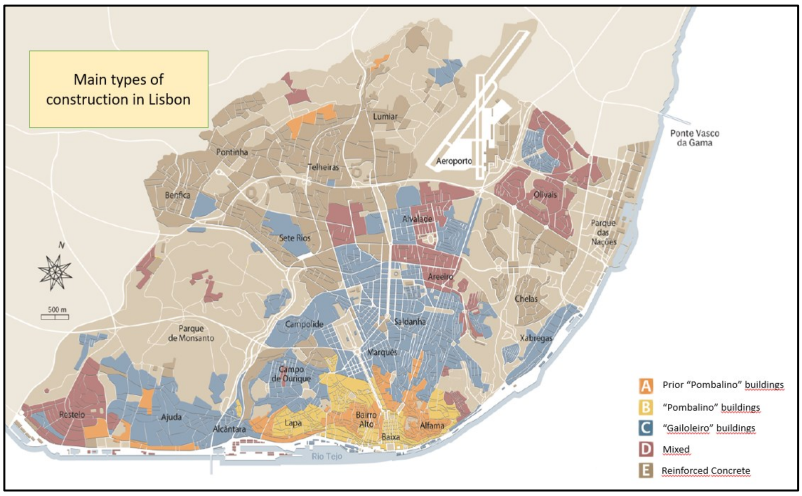


Figure 8 – Types of construction in Lisbon (adapted from CML, 2018)

Figure 8 represents what Lisbon is nowadays showing the evolution of construction techniques to date. However, to describe and clarify the building corresponding to the case study it is important to show what Lisbon was prior to the earthquake. So, in Figure 9 there is a map of the city before the earthquake showing the location of the case study building. Important to note, that prior to the 1755 earthquake there seems to be a building in this location. Looking at Figure 8 and Figure 9 , it is clear that Lisbon has grown quite substantially since the earthquake, however there was a period in which the demand for property gained emphasis leading to the adoption of a new construction system, called today the "Gaioleiro buildings", which was a quicker way of building perhaps but not as robust as a "Pombalino Building". Today, there is a several number of historical buildings ("Pre-pombalino, Pombalino and Gaioleiros) which have been subjected to alterations to a variety of changes that perhaps did not had the due care as it is recommended nowadays structurally. Fortunately today, the current legislation demands the assessment of these buildings by competent people in order to reduce the damage of the existing patrimony.

Figure 9 – Lisbon map prior to the 1755 earthquake (adapted from picture of "Biblioteca Nacional de Portugal")

After the earthquake, Marquis of Pombal, Portugal´s Prime Minister at the time, set up a group (ahead referred as "The Group") under the name of "Casa do Risco das Reais Obras Publicas" (Meaning, the Planning House for Royal Public Works). The Group consisted of competent and determined professionals who worked with industry and commitment, each one of them contributing significantly to the creation of rentable building. (Dias et al., 1996)

Constructions Systems chronology

In Figure 10 and Figure 11 is showna representation of the evolution of the contruction processes in Lisbon. Wars and natural disasters have been in the past the main causes that have contributed for the destruction of the current Portuguese patrimony, being particularly Lisbon suffering the most transformations since the very beginning. Not only that, the evolution in science and techniques have taken a big part in the progress of the construction processes, becoming reinforced concrete currently the most desired solution since mid XX century. (LNEC)

There are two type of buildings prior to the “Pombalino building type”. Houses from the medievel period with overhung floors, have been ordered to have its balconies removed as of the letter of “Regia from D.Manuel de 1499” which stated that the balconies had to have its balconies demolished, the facades and top of the buildings to be aligned and wherever there was timber, it should be replaced with masonry and stone, in order to prevent fires and to fight the insalubrity of these timbers. The majority of these building have been destroyed with the earthquake, however, there are some examples which can be observed in the oldest parts of Lisbon. To comply with the Letter of D.Manuel 1499, the “Renascentistas” buildings start to appear from year 1500 to 1755(O.A., 2016). This system is a new structures that appears to comply with the new rules established by The Group and these sort of building were implemented in the urban centres that were destroyed by the earthquake of 1755.(O.A., 2016)

By mid of XIX centrury, the memories of the earthquake started to fade away and with it the robusteness and strength of the buildings. Bearing in mind that Lisbon was experiencing the expansion of downtown, the “Gaioleiro” building came to simplify the construction system saving time and money, bringing taller buildings and greater areas. This system has been so simplified that for many it has been considered as adulteration of the “Pombalino” system. (O.A., 2016)

In 1930, the implementation of the general regulations for urban construction in Lisbon, required that decaying materials should not be used neither combustible materials, with preference to reinforced concrete. Since this regulation has been implemented, concrete was specified to wet areas to prevent the decay of timber joists and on stair case lobbies to avoid the spread of fire. (O.A., 2016)

The use of concrete slabs as a solution to avoid rottenness and the spread of fire, brought in new techinques to the concrete structures design. So, in 1930´s a solution of reinforced concrete columns and beams has been developed for public buildings and difused between Architects and Engineers. As a result, a new regulation in reinforced concrete came into place in 1935. (O.A., 2016)

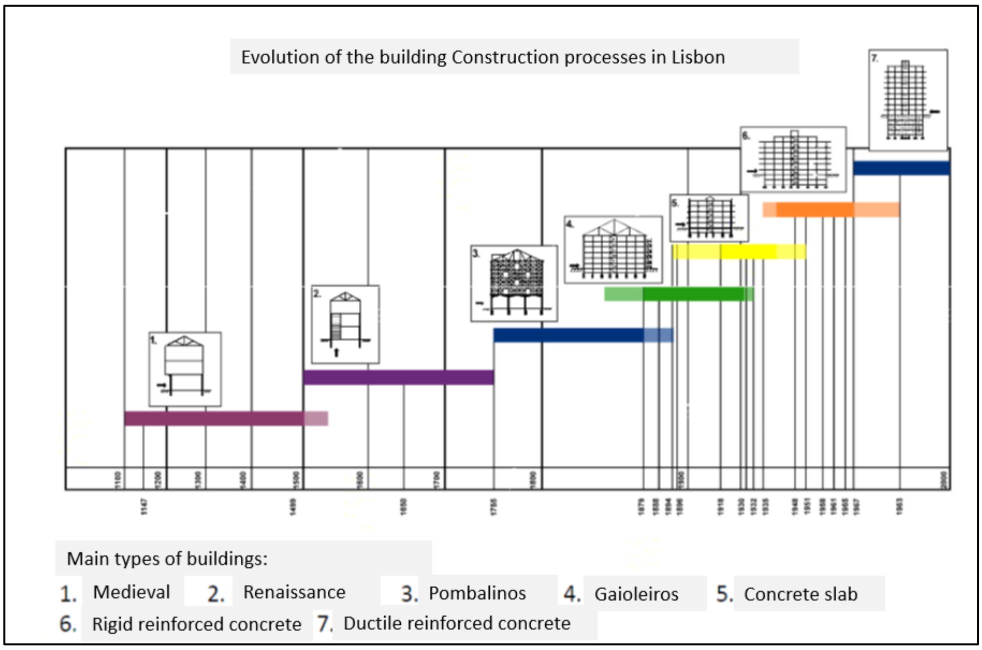


Figure 10 – Construction Processes evolution (Còias, Vitor, Reabilitação estrutural de Edificios antigos: adapted to English)

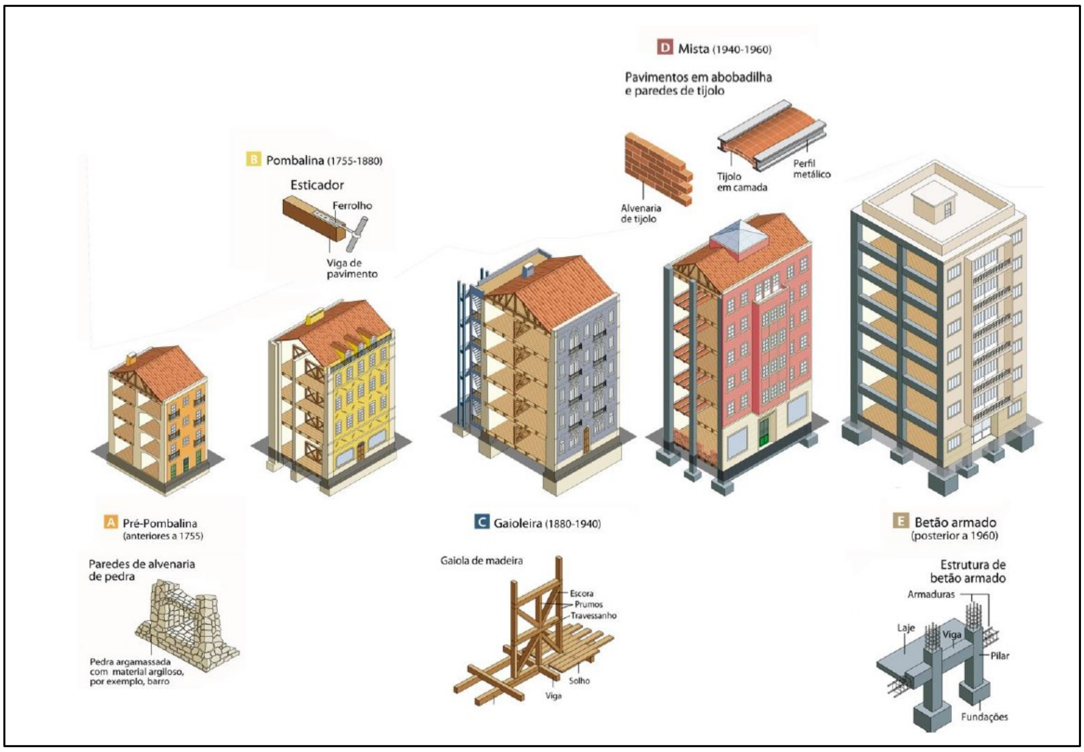


Figure 11 – Construction Processes evolution in Lisbon (Font: Adapted from “Jornal Expresso”, based in information from CML)

Construction systems main components

This section presents some of the main elements of each construction system relevant for the conducted analysis. Due to case study’s location, history and construction evolution, this discussion will focus on “Pre-Pombalino”, “Pombalino” and “Gaioleiro” Buildings.

Medieval and Renaissance buildings (Also referred to as Pre-Pombalino Buildings)

After the acceptance of the proposal by Marquis of Pombal to re-build the city, all the new buildings built from 1755 till about 1880 were designated as “Pombalino” buildings. Therefore, any new buildings referred prior 1755 are classified as Pre-Pombalino buildings. The Pre-Pombalino building refers to a wide range of variety in construction, either in materials either in structure, techniques used, etc., here divided in two main types: Medieval Building and Renaissance Building. Figure 12 represents a general schematic representation of a typical Medieval Building, which are known to be the first masonry buildings in Lisbon, which commonly have the following features (O.A., 2016)

- Masonry walls only at ground floor level, and upper floors made of timber
- The typical step out at 1st floor is supported by a timber beam along the front elevation
- Continuous and direct Foundations
- Flooring supported by beams with circular section with a maximum span of 4m.

From the XV century there was a requirement by the letter of “Regia from D.Manuel de 1499” who ordered the balconies to be demolished, the facades and top of the buildings to be aligned, the replacement of timber with masonry and stone to have buildings that could be more robust to fires and insalubrity, therefore the following defines the Renaissance Building" as per Figure 13(O.A., 2016))

- Structural brick and stone walls up to the roof
- Infilled timber walls with masonry from ground to top floor surrounding the stairs
- Floors and roof structure in timber
- Support arches at ground floor made of masonry stone or masonry brick

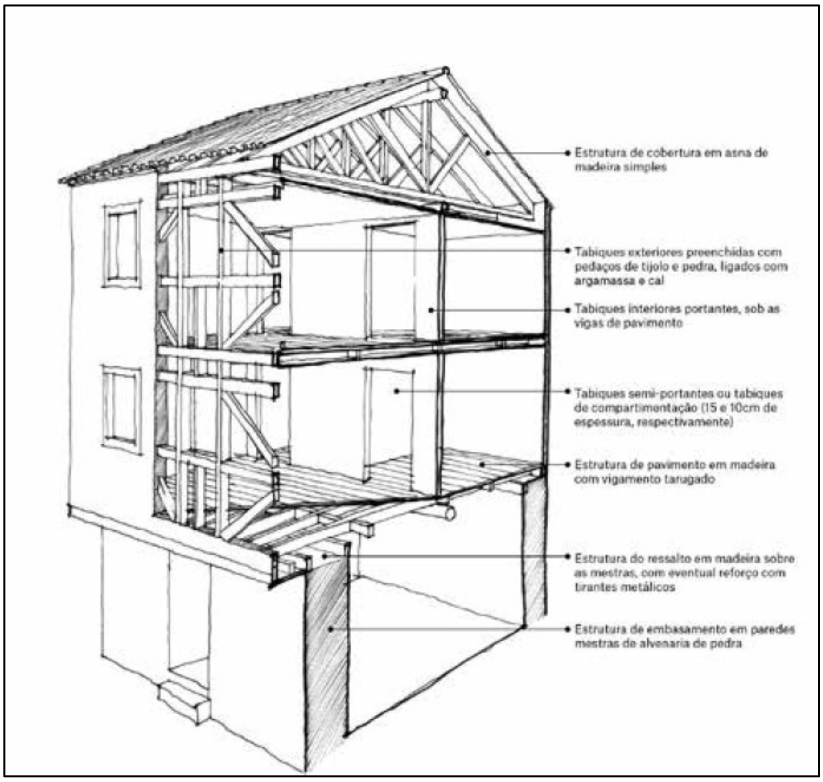


Figure 12 – Medieval building stepped at 1st floor ((O.A., 2016)

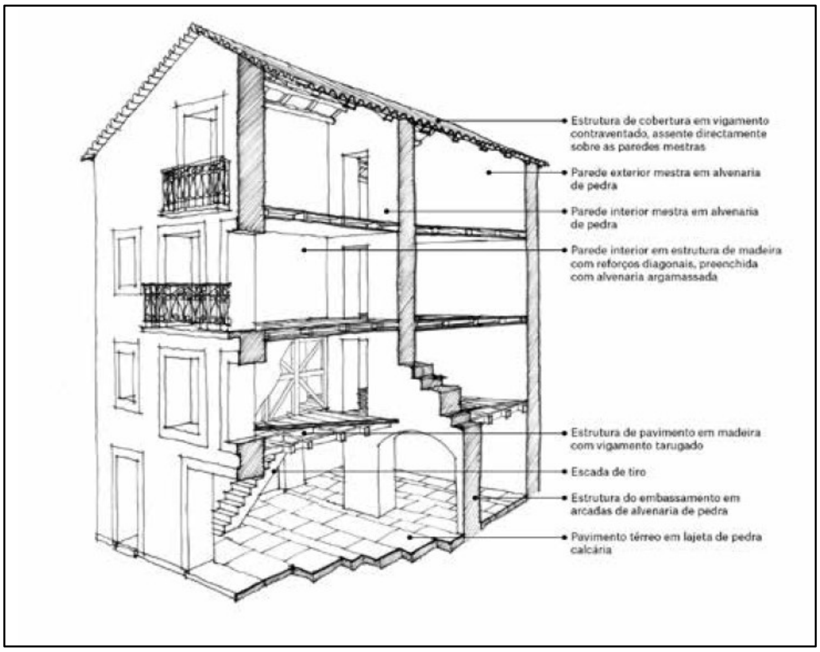


Figure 13 – "Renascentista" building (O.A., 2016)

Pombalino Buildings:

. Pombalino buildings are the start of a new “era” of construction in Portugal. After the 1755 earthquake that hit Lisbon and surrounding areas, Marquis of Pombal had in his hands the duty of rebuilding Lisbon. However, he needed to create a robust construction system which could survive an earthquake and therefore reduce the human impact if another earthquake occurs. Based on that The Group developed a construction solution which is described as per below: (O.A., 2016)

- Each building is within a block, to have the structural behaviour to work as a block in lieu of an individual building.
- Timber structure in gantry and braced, perpendicular in Plan.
- Ground floor external walls in stone, Robust external walls and stone or masonry domes, supported by walls, arches or columns made of stone.
- Cross vault if usually the most common vault seen around this kind of building
- At ground floor level the external walls are built of rendered stone and linked into the internal structure made of oak timbers. However holm oak can also be found. The external walls have in average 90cm thickness, reducing as the building goes up.
- Columns are made of giant stones

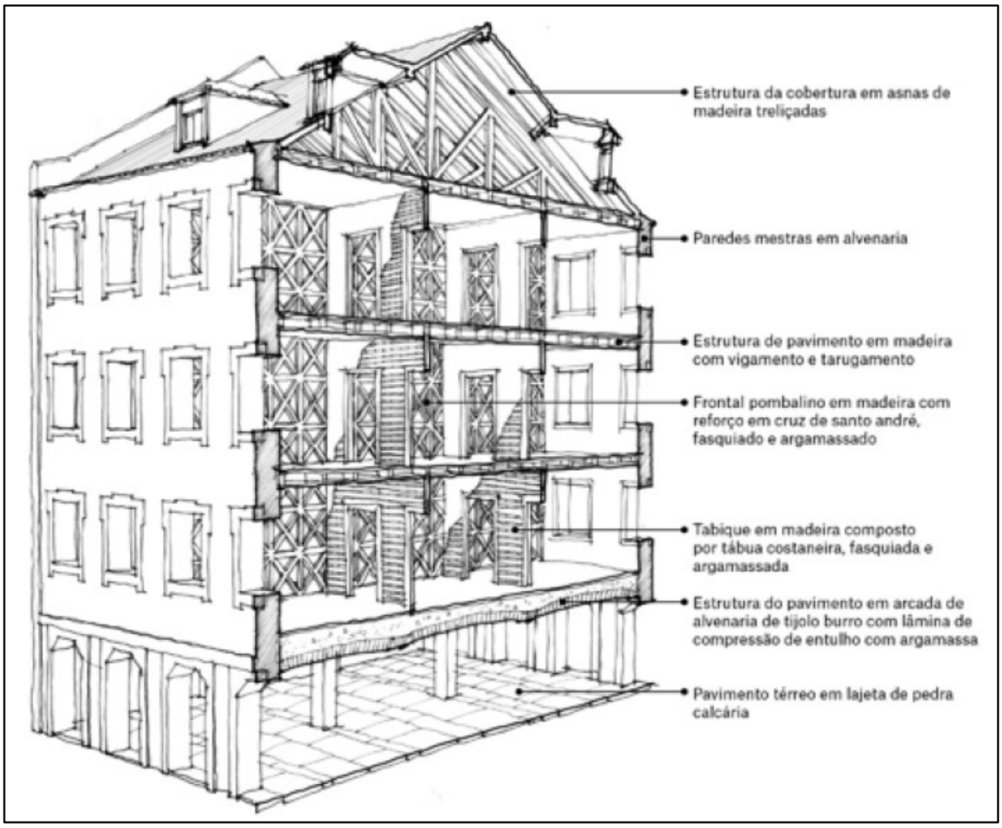


Figure 14 - Typical Pombalino Building (O.A., 2016)

Foundations on Pombalino Buildings

Understanding the principles and global concept of the foundations for Lisbon downtown is essential. The city of Lisbon is located next to Tagus River, not far away from its mouth to the sea. Piles were used to allow the commencement of construction on flooded soil, as well as to prevent the building from sinking into the alluvium. In addition, it was taken into consideration the necessity of proving capacity to the building to resist small earthquakes, through a solid construction of the walls, the foundation arches and the vaults which covered the ground floor, preventing the lower part of the building from being crushed (Dias et al., 1996)

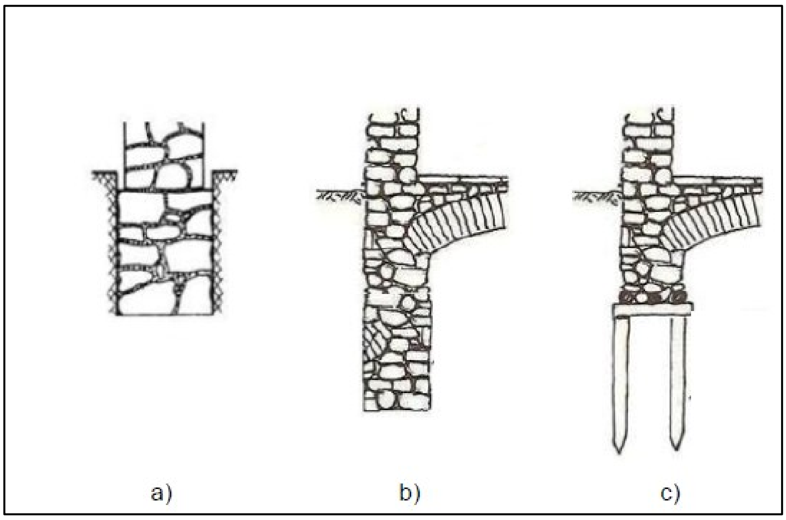


Figure 15 – a) direct foundation b)semi-direct foundation c) semi-direct foundation with timber piles (Typical types of foundation for a Pombalino Building (Image from (Simões & Bento, 2012))

Floor construction between Ground and 1st floors on "Pombalino" Buildings

Looking at the research done by (Pedro & Gouveia, 2014), there are a few types of floor construction adopted as a construction system at the lower level, however, it is not possible to conclude the reason for the use of each type to each location. What is known is that using vaults and arches can bring more space and that cross vaults are found to where stores and stables were located usually in secondary streets. The arches and crossed vaults and barrel vaults solutions enabled the building to withstand the movement of the land in the event of tremor or any imbalance caused by neighbouring buildings, and protected all the construction from the spread of any fire that start at ground floor level, which was occupied by storehouses or coach houses. In the main streets, where there are no vaults over the ground floor rooms, there are two variants of the form of construction: arches in parallel lines and arches perpendicular to one another forming a grid in plan. Is also common, to find timber beams resting on structural walls. Figure 16 and Figure 17 shows the different types of floor construction found in a typical "Pombalino" building.(Dias et al., 1996)

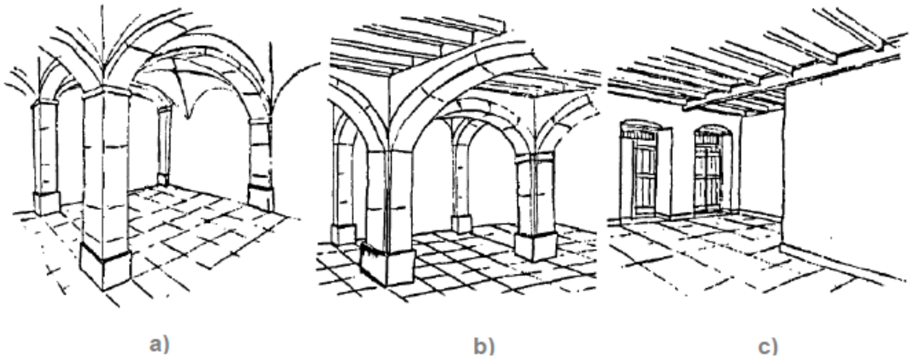


Figure 16 – a) Vaults and Arches; b) Arches and timber beams; c) Timber beams on walls (Dias et al., 1996)

Figure 17 – From left to right: Solution of arches and beams, Barrel vaults made in masonry, crossed vaults made in masonry (Simões & Bento, 2012a)

External Walls

The external walls at ground floor were entirely built in stone masonry, to prevent rising of dampness from ground and to prevent fire propagations between warehouses. Typically, these walls do have a thickness that varies from 0.9m to 1.1m. Above ground, floors were built in rubble stone masonry bounded by air lime and sand mortar with poor strength capacity. The façade openings were covered by relieving arches made of stone masonry or ceramic blocks (Simões & Bento, 2012)

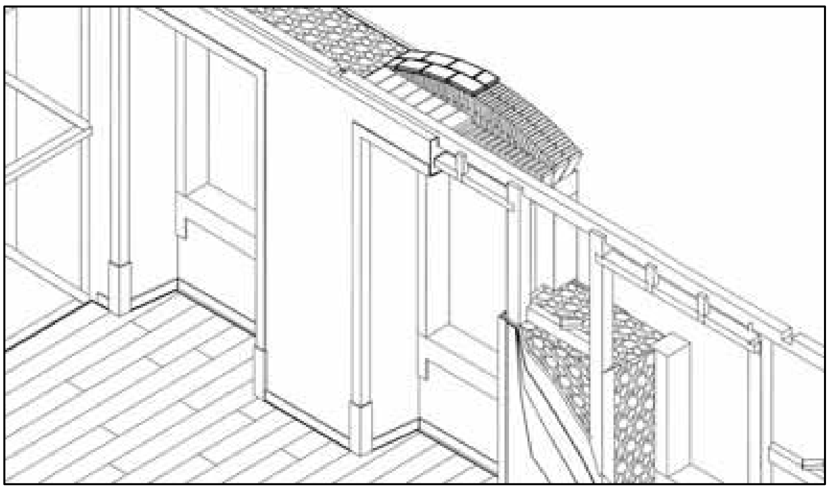


Figure 18 – Section in 3D for an external wall from internally (Lisboa, 2005)

“Frontal” walls (Structural Internal partition walls)

“Frontal” walls were composed by vertical, horizontal and diagonal elements connected between assemblies and nailing fixation, forming a truss. Joists are positioned in an empirical principle that is difficult to deform a triangle, without changing the length of its sides. 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. (Simões & Bento, 2012).

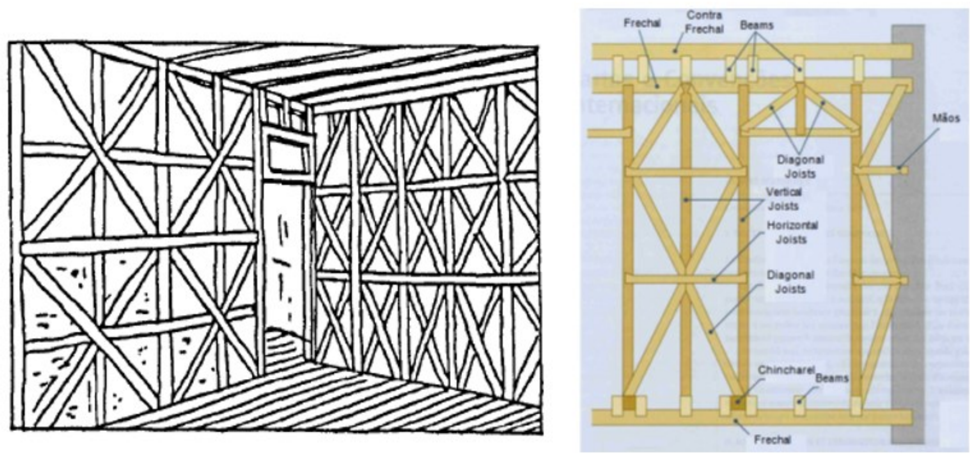


Figure 19 – Look of a cage (gaiola in Portuguese), substructure of a frontal wall that would be infilled with rubble stone (Dias et al., 1996; Simões & Bento, 2012a)

“Tabique Walls” (Partition walls)

“Tabique” walls have a lighter structure made of timber laths nailed to vertical joists, filled afterwards by masonry and air lime mortar. Tabique walls were plastered on both sides but can easily be distinguished by its thickness, which varies between 0.10m and 0.12m and its used mostly for compartment divisions (Simões & Bento, 2012).

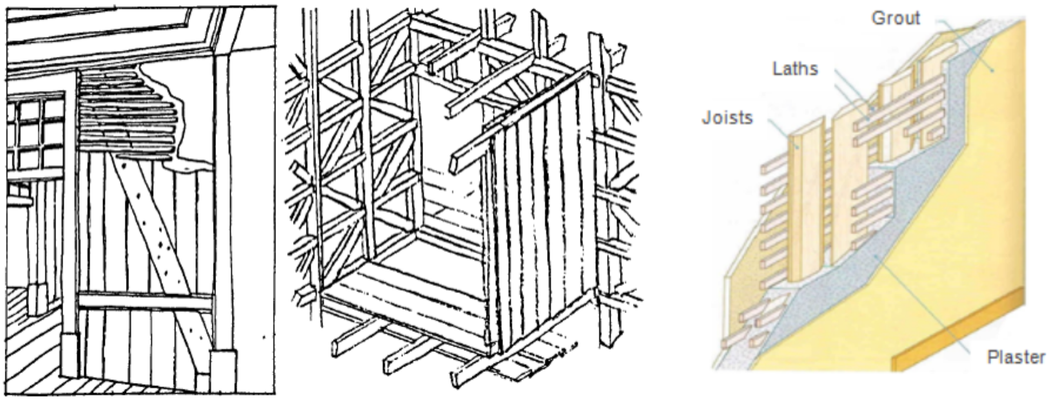


Figure 20 – Example of a partition wall with timber laths nails horizontally to the vertical joists (Dias et al., 1996; Simões & Bento, 2012a)

Roof and Skylights

The “Pombalino” rentable buildings are covered by mansard or dual-pitched roofs with the party walls rising 0.80m above the roof to prevent fire spread. Buildings in main streets and squares have mansard roofs which had a more complex structure, allowing for reasonable ceiling height throughout the attic as well as easy access to the windows. The roof structure was connected to the structure of the cage. In some case trusses act as supports to the heavy load of the roof. (Dias et al., 1996)

The skylight construction process usually consisted of extending the stairwell structure above the roof and covering the inside with planks and the outside with metal plates. The structure of riveted metal struts held the glass which was fixed to it with lead, except for the lower edge, which was rounded and was free to move with the expansions caused by the heat. (Dias et al., 1996)

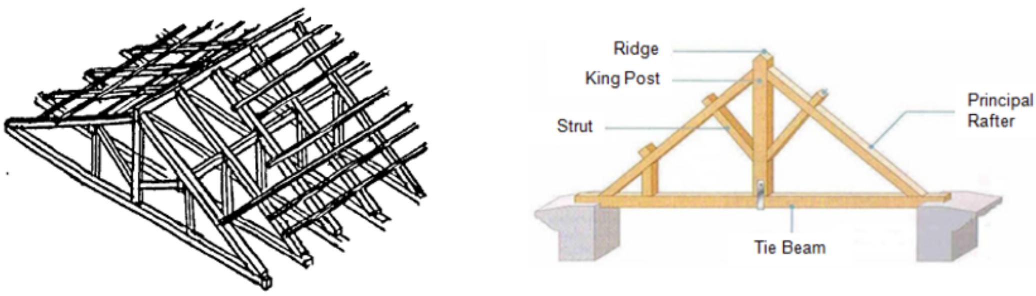


Figure 21 – Typical Roof structure of a “Pombalino” building (Dias et al., 1996; H. Meireles et al., 2011b)

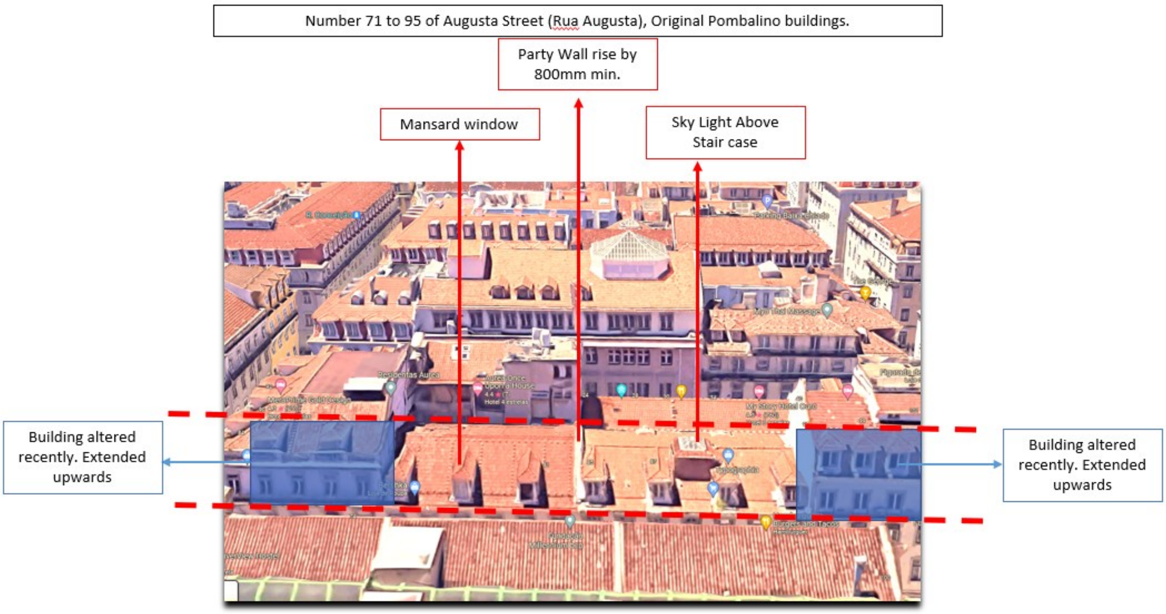


Figure 22 – Original Pombalino Building showing the different features of a “Pombalino” building on its roof (Image google maps 3D adapted)

Stairs

Originally the stairs of “Pombalino buildings” were internal without any natural light apart from the skylight installed straight above the stairs. Mainly because of fire safety the ground floor stairs are made of stone masonry. From 1st floor upwards stairs have been built in timber supported at landing levels and also the stair well walls. (Antunes & De Miranda, 2011)

“Gaioleiro” Building

The expansion to the city of Lisbon starts to grow quite substantially after the earthquake, however there was a period in which the demand for property gained emphasis leading to the adoption of a new construction system, called today the “Gaioleiro buildings”, which was a quicker way of building perhaps not as robust as a “Pombalino Building”. Note that nowadays “Gaioleiro” (meaning cage in a depreciatory way) is related with the buildings built to be sold or to be 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 (Simões & Bento, 2012a)

“Gaioleiro” buildings appear from 1880 as a result of the real estate interest and demand due to the expansion of the city. (Appleton, 2005), looking at the case study the one story above third floor level belong to the “Gaioleiro” construction system therefore the importance of referring to the features as below and also shown on Figure 23:

- The masonry walls were made of irregular rendered stones
- The thickness of the façade walls varies from ground to top floor, there are cases where walls have 90cm at ground floor and 30cm on top floor.
- Party wall made of solid bricks
- The internal walls are made of orthogonal frames infilled with masonry at floor level, midspans and also around the doors.

- Non-structural walls ("Tabique"), had its timber structure which were cladded with upright timber boards and then rendered.
- Flooring structure supported by external walls, sometimes not supported by the perimeter beam, or supported on these only with nails and also supported by internal walls.
- Timber beams perpendicular to the façade and sometimes underestimated.
- Roof structure made of a simplified timber truss.
- Direct foundations
- Party wall foundations of bad quality
- Rear façade with closed balconies made of steel.

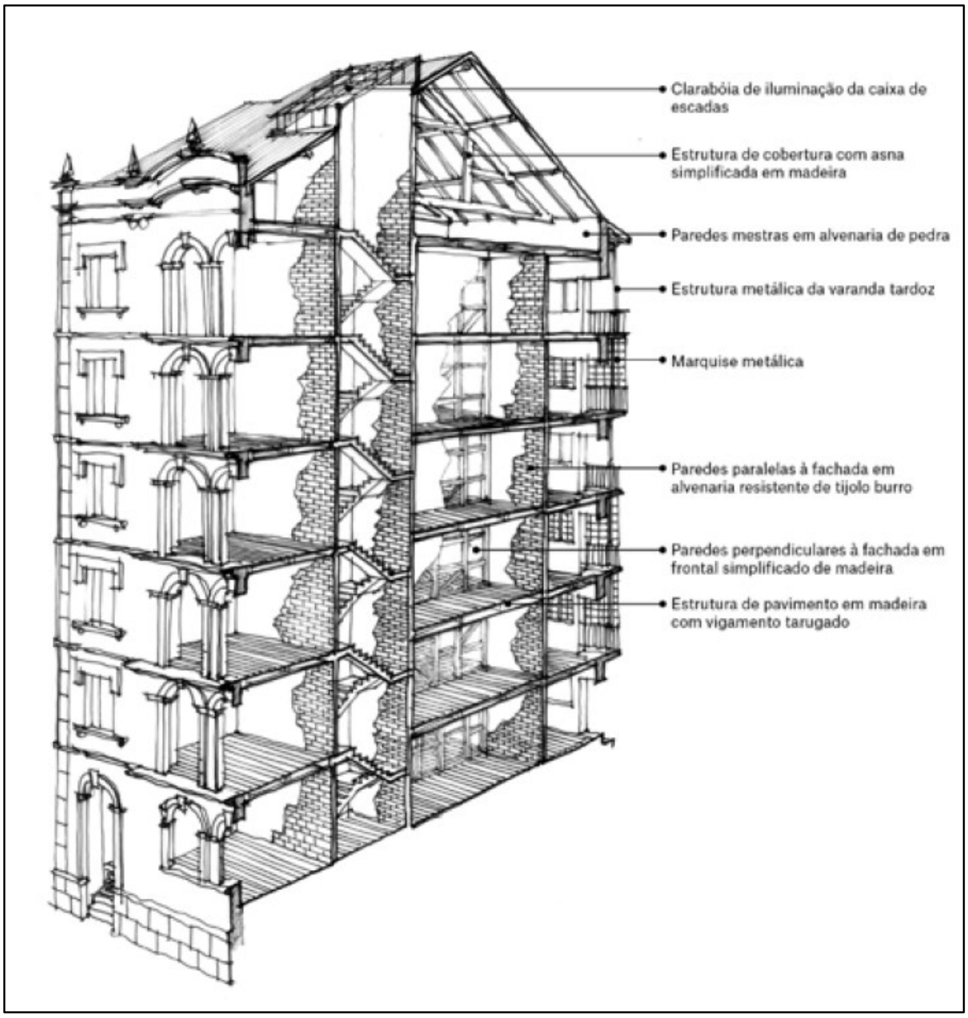


Figure 23 - Typical Gaioleiro Building (O.A., 2016)

Main anomalies or pathologies found in old buildings

Until the use of reinforced concrete, the use of a Structural Engineer and the Architect does not apply. Elements or construction systems are simultaneously resilient and architectural components. Within an old building, the finish floor also contributes for the stiffness of the structure. Therefore, the main pathologies and/or anomalies found on these old buildings are always a threat to the structural behaviour. (O.A., 2016)

Pathologies found in the existing patrimony are result of aging, lack of maintenance, building abandoning, natural disasters, bad construction quality and interventions by non-qualified people.

The main reason for aging of the building is the water ingress to the building, either from the roof or ascending water from the ground. Nevertheless, interventions such as inadequate maintenance, adulterations to the original construction system, adding floors, suppression of internal walls, enlarge lower floor openings to allow for shops, adapting the building for utilities, will also bring issues to these buildings, bringing visible cracks as there are differential settlements to the walls and also foundations that become undersized for the extra loads. (O.A., 2016)

Descending water will also accelerate the decay of timbers joists to where these link into the main facades. Fungus and xylophagous insects will destroy the timber joists over the time, making these quite weak, as a result and in the worst case scenarios bring the collapse of the structure. (O.A., 2016)

Below are some pictures of common issues found in old buildings:

Figure 24 – Example of a building with Pombalino Façade that has been extended with Gaioleiro type construction Rua da Assunção 34-40 Lisbon (Image from (Lisboa, 2005))

Figure 25 – Bad examples of rehabilitation with cementitious solutions which did not last long (Lisboa, 2005)

Figure 26 – Vegetation on facades which will cause cracking and detaching of elements (Lisboa, 2005)

Figure 27 – Timber joists decay on its edges due to water ingress (Lisboa, 2005)

Case study

Lessons are not only to be learnt from mistakes but from important events that should not ever be forgotten, like the 1755 earthquake. Further ahead there will be a description of a building full of history due to the events it has been subject and its location.

Shown in Figure 29 **Erro! A origem da referência não foi encontrada.**, is the case study building which is located in Lisbon facing “Rua do Alecrim” and at the rear “Rua das Flores”. The oldest record of ownership for this building is dated 1871 which is 116 years after the 1755 earthquake. Is not known if the building has been acquired by the insurance company “Tranquilidade” on this year or its construction finalized on that year.

It is known that for the last few years, the building has been in use, which suggests that some kind of maintenance has been applied to keep it going. The lower ground floor has been used as commercial/store area, the floors above (up to 3rd floor) as offices and the fourth and attic area were being used as residential.

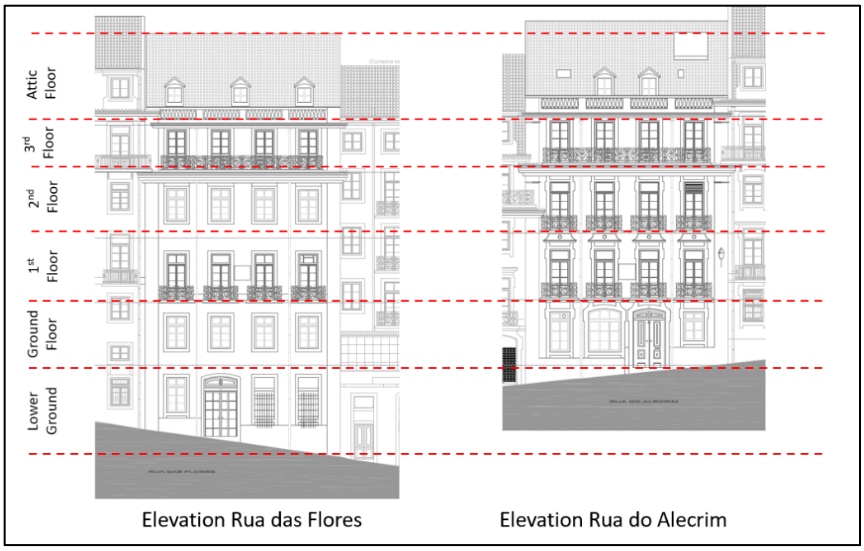


Figure 28 – Floor identification (Architect Drawings adapted)

In 2018, the building has been acquired by an investor who will change the current use of the build to seventeen residential apartments.

Currently, the building is being subject to a structural and aesthetic refurbishment where strip out works will unveil the carcass of the building and allow for further analysis and to see that the planned structural and aesthetic design will work with what is on site.

Figure 29 – Case study location in Lisbon (Images from google maps-adapted)

Taking in consideration the location of the building and comparing it with Figure 6, the proposal suggests there was a "Pombalino" building planned for this location. Considering the proposal and the date shown on the façade of the building (1871), is clear that the first assumptions is that the structure of the building will sit within the "Pombalinos".

Figure 30 – Rear and Front façade showing the 1871 year (Onsite picture)

Refurbishment is one of the most interesting jobs to do in construction, as there is always a lesson to be learnt, mainly to where the building being refurbed is a "Pombalino" building, which has been subjected to changes of use since it has been built. Changes that include the incorporation of utilities and services to the building keeping up to date with the evolution and perhaps another incident linked to a fire. Lessons to be learnt is that high demand has always been an enemy of the perfection, which in the majority of the times a competent person is not used to provide the service. Note that some of the details are not confirmed but justified assumptions. It is important to note that the case study is a very good example of a Pombalino building and as it has revealed to be in a very good condition taking in consideration that has been built approximately 200 years ago and it has had very little changes to its structure.

Foundations

As described above on the "Pombalino" buildings, the foundations can either be direct, semi-direct or semi-direct with timber piles. However when carrying out a trial pit (Figure 31) on site it looks like that something else might be there on the ground apart from a traditional "Pombalino" foundation with arches. Looking into more details as the history and the location of the building lets refer to Figure 9, where is represented the Map of Lisbon prior 1755, suggesting there was a Pre-Pombalino building in this same place that has been badly damaged by the earthquake as Figure 6 suggests a new Pombalino building for this location. This suggests that the existing foundations of the Pre-Pombalino building may have been kept in situ.

The area of implantation for the building extends further at ground floor, towards "Rua do Alecrim", which means there are foundations present on this level. There is no information on these foundations and for that reason is assumed for modelling purposes that a traditional foundation is present.

Figure 31 – Trial pit to the existing foundations at lower ground floor (Picture taken on site)

Quadripartite Vaults

The rear the building is facing “Rua das Flores”, has got a lower floor which were perhaps used for commercial services, perhaps as stables or stores as the space is maximized with crossed vaults as shown on Figure 32 **Erro! A origem da referência não foi encontrada.**. It is known that traditional Pombalino buildings were designed to have the lower level for commercial services, therefore the maximum room it could have the better (Dias et al., 1996)

Figure 32 – Quadripartite Vault (Pictures taken on site)

New findings on site (1)

While strip out works were ongoing, it has been found a link between ground floor and lower ground floor that was not expected by the designers or the client.

Pombalino buildings were planned to behave as a block of buildings together and not individually. (Dias et al., 1996).When these blocks were too long tunnel links were created between streets to allow access from the front to the rear of the building. Although, is not totally justified the existence of this tunnel link as there is a road (Rua Ataíde) literally next to the building under analysis.

However, referring to Figure 6 as the official plan to rebuild Lisbon of the 1755 earthquake it suggests a long block of Pombalino buildings were planned for the area justifying the existence of the tunnel. Therefore, following this line of thought is not clear if “Rua do Ataíde” has originally been built due to an alteration of the plans, that there is no information of, or if the block of buildings has been built has shown on plan as per Figure 6 and for reasons not known a new road has been created (“Rua do Ataíde)

Figure 33 – Tunnel access from ground floor at Rua do Alecrim to lower ground floor at rua das flores

Internal and external Walls at ground and lower ground floor

All the internal walls at ground floor in contact with the ground and where there is risk of ascending water through the walls, are made of stone masonry. A typical Pombalino building do have stone masonry walls that vary in thickness from 0.9m to 1.1m. However the case study does have some of the walls thicker than the common Pombalino building. (see Annex A – Floor plans showing thickness an type of walls)

Figure 34 – Columns in contact with the ground made of stone masonry (picture taken on site)

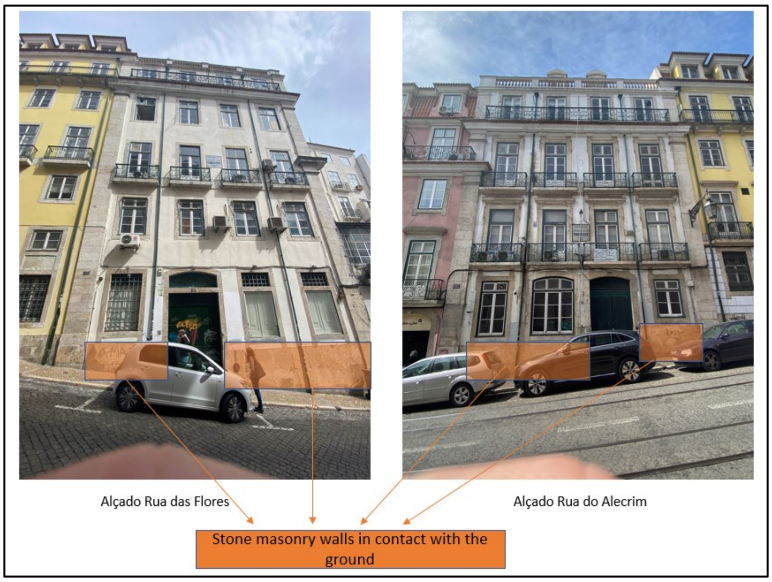


Figure 35 – Façade elevations showing stone masonry in contact with the ground to avoid ascending water (on site pictures adapted to explain the stone masonry in contact with the ground)

Party Walls

Party wall by its name are walls that are shared by different buildings. Once again, these walls do have the same features as an external wall. In the case study the common thickness of these walls sit in line with the specification (these thicknesses can be seen in the Annex A – Floor plans showing thickness an type of walls). During the works on site, it has been noticed that there was a crack on the wall and also what is called a “gato metálico” (“gato metálico” was used to bring more stiffeness to the buildings by connecting the floors to the facades). In this particular case, the building next door has got the floor level at a different height which might be the cause of the cracking. When these kinds of issues are found, further assessments must be carried to check the crack origin and also if it changes in size and thickness.

Figure 36 – Party wall showing a “Gato metálico” indicated by the green arrows and crack indicated by the red arrows. (picture taken on site and adapted for the case study)

“Tabique Walls”

“Tabique” walls have a lighter structure made of timber laths nailed to vertical joists, filled afterwards by masonry and air lime mortar. Tabique walls were plastered on both sides but can easily be distinguished by its thickness, which varies between 0.10m and 0.12m and its used mostly for compartment division (Simões & Bento, 2012a)

The majority of the walls on the case study are within the spec as per the above. However, as these do not have any structural input to the building, they have not been considered as part of the modelling analysis, but it is important to understand where these are in the building and must not be confused with a structural wall.

Figure 37 – Examples of “Tabique Walls” on site (Pictures taken on site)

Frontal walls

Frontal walls start to appear at ground floor area, above the lower ground floor, on top of the arches that form the quadripartite vaults. Frontal walls can be seen on this building from ground floor to roof level.

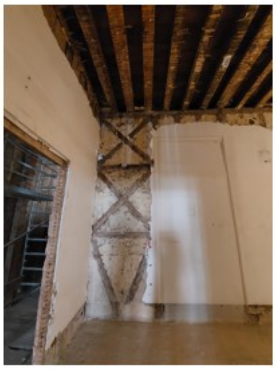




Figure 38 –From left to right , frontal walls at roof level; front walls on levels below (Pictures taken on site)

New findings on site (2)

Initially, before the strip out works it was believed that the two levels at the top would be Gaioleiro type construction, however after the strip out works everyone involved was surprised that frontal walls are present from ground floor to the roof area. It's understood, however not confirmed that this building has been initially been built with 4 stories plus the roof at the time of the rebuilding of Lisbon. After that, the building has been extended up, where the Builder/Designer/Architect at the time, had the care of continuity by using the existing construction system to carry on the works.

Roof Structure

There is no information available about the finalized building on this site after the 1755 earthquake. For that reason, after inspecting the building to its roof structure is possible to see that there is a "Gaioleiro" truss. So, having a "Pombalino" structure all the way up to the roof is not tying up with a "Gaioleiro" truss, which suggests something may have happened during Gaioleiro "Era". Looking further into the roof structure it is possible to see some burnt joists which might mean that during the Gaioleiro "Era" a fire destroyed the roof and this has been rebuilt with a Gaioleiro Truss.

Figure 39 – Roof trusses, clearing showing burnt timbers that have been left in position (Picture taken on site)

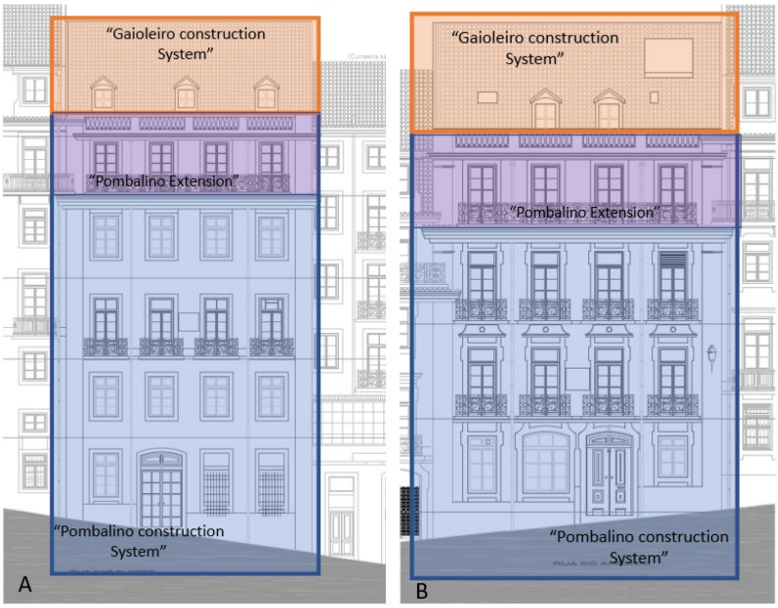


Figure 40 – Showing the different types of construction at different times, A: Façade elevation from Rua das Flores; B: Façade elevation from Rua do Alecrim (adapted from the Architect drawings)

4. STRUCTURE MODELLING

About the modelling programme

The structure has been modelled using Tremuri®, a software tool for structural analysis of masonry buildings .In this program, the structure was modelled through an equivalent frame consisting of a particular type of element, called a macro element, which makes it possible to best capture the seismic behaviour of masonry structures and provide all the information necessary for the designer to accurately examine the structure itself. (2022 S.T.A.DATA srl, 2022)

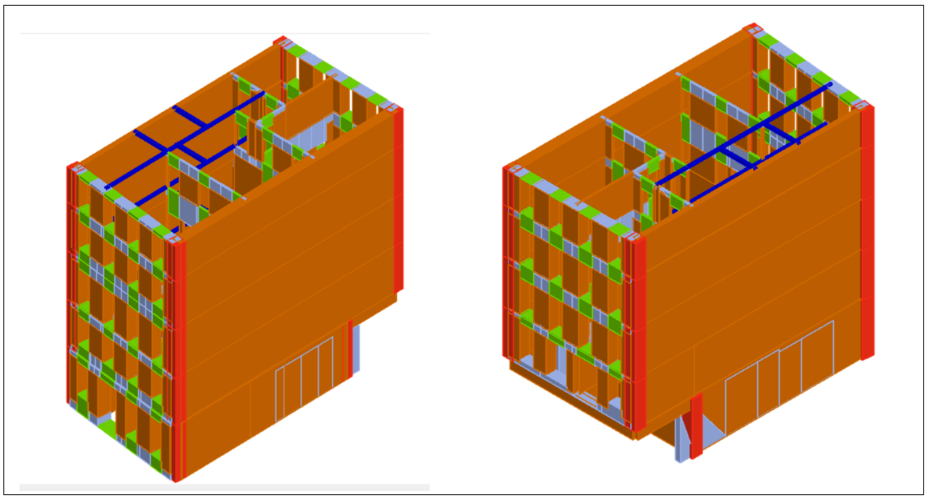


Figure 41 – Case study 3D model in 3Muri showing the different macro-elements.

The structural assessment

The adequate knowledge of the geometry, construction proposition and the materials type, are a mandatory requirement to proceed with a seismic assessment of an existing building. Nevertheless, the alterations introduced to the building over the years and the state of conservation are also relevant. Following NP EN 1998-3:2017, gathering information about the building under analysis must be as exhaustive as getting any specific documentation available, on-site surveys and any information that could come from a safe source. Is important to match the information gathered as much as possible, this way minimizing any uncertainty that may persist. (Candeias et al., 2020b).

Once all the information was gathered, the model has been developed using floor plans previously prepared with grid lines. These grid lines have been marked using existing architectural floor plans, following a site survey where the structural walls have mostly been identified (Figure 42). By marking up the structural walls on plan and transferring these plans into Tremuri it was possible to create structural objects .Once all the structural elements had a definition and openings were represented an automatic mesh could be created and a nonlinear analysis be processed. Figure 43 represents the diagram of the input and analysis adopted for the case study.

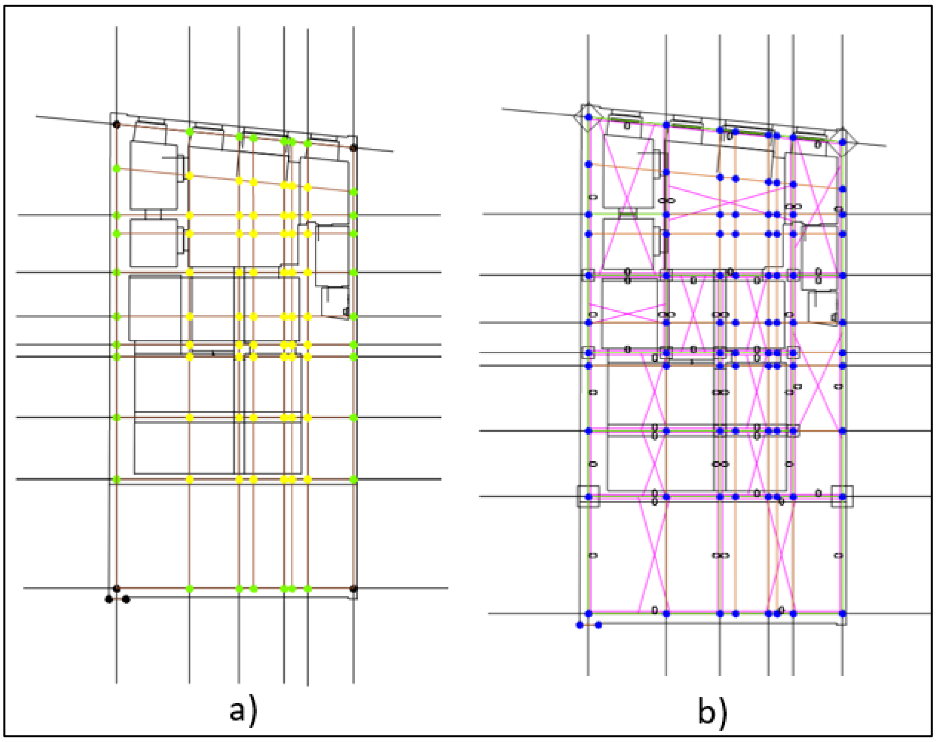


Figure 42 – a) Floor plan previously prepared with grid lines in representation of the structural walls, b) floor plan in Tremuri with defined structural elements

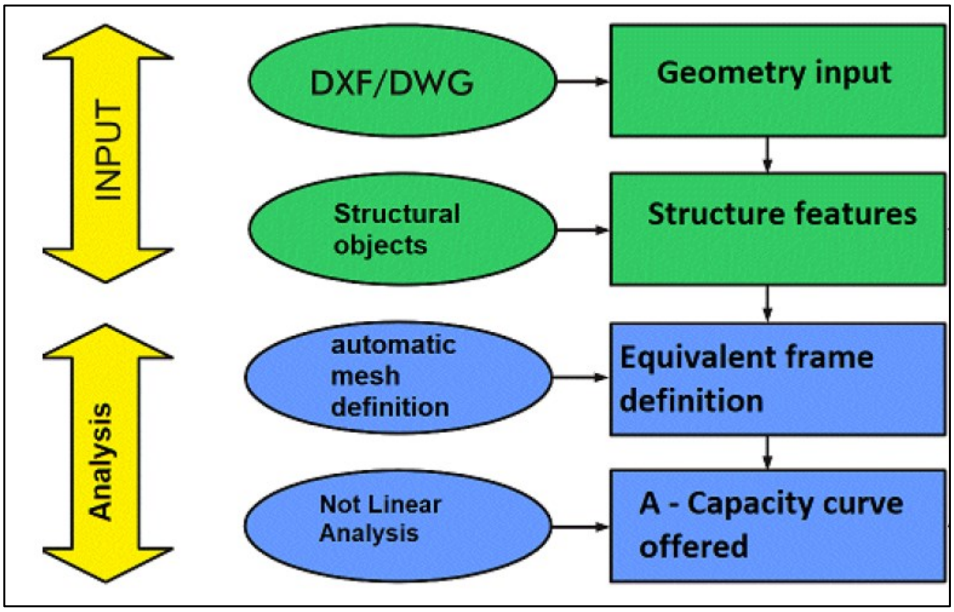


Figure 43 – Diagram Showing the analysis process adapted from (2022 S.T.A.DATA srl, 2022)

Structural elements definition – Walls geometry and modelling parameters

Following a site visit, a survey has been conducted to ascertain the walls geometry and the construction materials considered. Table 8 shows the geometry of the walls to each level and on Figure 44 is represented a floor plan for level 2 as an example of the assessment carried and its compatibilization with Tremuri. In Annex A, there is the survey carried for the remaining levels showing the different geometries and types of wall. It is important to note that, on this survey the

“Tabique” walls have been identified, however these have not been considered as structural walls and as a result, are not represented in the modelling program. As referred to in chapter 3, “Tabique” walls do not have structural input into the structure, firstly because these are not vertically continuous and secondly the strength and stiffness of it are highly lower than “frontal walls”, rubble masonry or stone masonry, meaning the contribution of these to an earthquake would be nil.

Table 8 – Walls, material type and thickness on each floor

Level 1 - Lower Ground floor		
Assigned to	**Material type**	**Thickness (m)**
Front elevation	Stone masonry	0,54
Rear elevation	Stone masonry	1,24
Party walls	Stone masonry	0,41-1,00
Internal walls	Stone masonry	0,43-0,82
Corner Columns	Stone masonry	1,0x1,0
Level 2 - Ground floor		
Assigned to	**Material type**	**Thickness (m)**
Front elevation	Stone masonry	0,91
Rear elevation	Rubble masonry	1,00
Party walls	Rubble masonry	0,34-0,81
Internal walls	Rubble masonry	0,32-0,57
	Stone masonry	0,52-0,60
	Frontal Walls	0,15
Corner Columns	Stone masonry	1,0x1,0
Level 3 to 5 - 1st floor to roof		
Assigned to	**Designation**	**Thickness (m)**
Front elevation	Rubble masonry	0,91
Rear elevation	Rubble masonry	0,88
Party walls	Rubble masonry	0,34-0,81
Internal walls	Frontal Walls	0,15
Corner Columns	Stone masonry	1,0x1,0

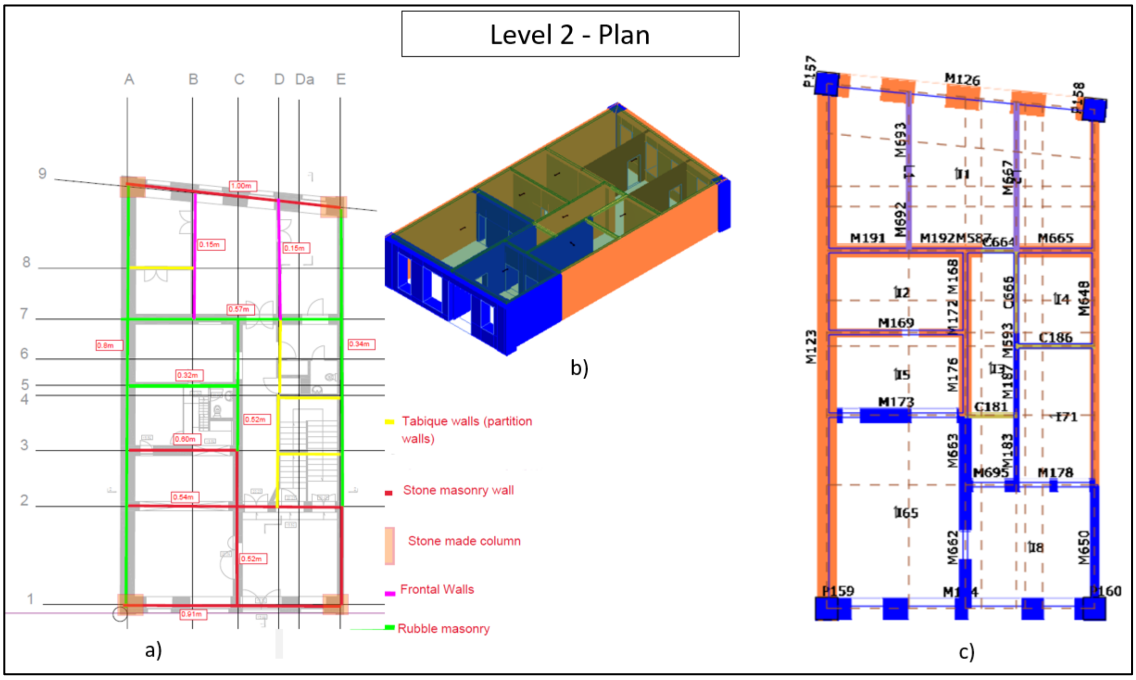


Figure 44 - Identification of structural walls and its representation in Tremuri

According to H.A. Meireles and R. Bento, who formulated a validated a macro-element for the equivalent frame modelling of internal wall in Pombalino type Buildings, the mechanical characteristics considered are in line with Table 9 . Is important to note, that the values considered on Table 9 are not reduced in any way when inserted into Tremuri®. The Model, will be calibrated based on a site survey, where frequencies have been measured.

Table 9 – Mechanical characteristics of masonry types (H. A. Meireles et al., 2012)

Masonry type	Average young modulus E [Gpa]	Average Shear Modulus G[Gpa]	Weight W [kN/m3]	Average compressive strength fm [Mpa]	Average Shear strength to [Mpa]
Stone Masonry	2.8	0.86	22	7	0.105
Rubble Masonry	1.23	0.41	20	2.5	0.043
Brick Masonry	1.5	0.5	18	3.2	0.076

Structural elements definition – Floors geometry and modelling

The geometry of the structural floors have been defined according to (Simões & Bento, 2012b). The joists of the floors have a section of 10x20 centimetres and the wood pavement a thickness of 2 cm. In this basic configuration, the stiffness contribution of floor is mainly related to the pavement contribution: thus, they result as quite flexible orthotropic membrane finite elements. The stairs have been modelled as floors having the following cross sections: 10x10 cm for the joists and 2 cm for the pavement. The joists run every 30 cm for both stairs and floors (Simões & Bento, 2012b). Figure 45 is a representation of the floor sections adopted according to (Simões & Bento, 2012b)

Reading the floor plans as shown on Annex A and also the example shown Figure 44 - Identification of structural walls and its representation in Tremuri, is noticeable that frontal walls are not always continuous in plan, where, for Tremuri® this is a requirement. To make Tramuri® run a calculation, it will need to understand that a box is being read. This away and to make it more accurate as possible a timber joist with the same section has been considered.

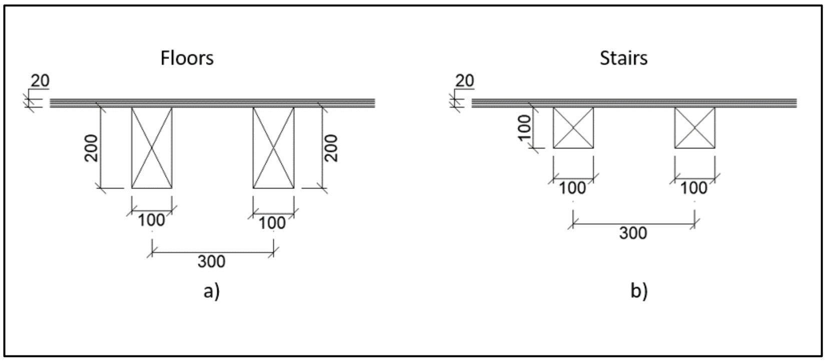


Figure 45 – a)Timber joists considered for floors, b) timber joists considered on stair well.

Timber in Pombalino style buildings are present in almost every other element, these can be “tabique” walls, frontal walls, floors and stairs. Is relevant to note that assessing a building that has been erected so many years ago, the materials may not be as good as it seems, therefore when looking into the characteristics of them these may need to be downgraded. The sort of timber used in the past, could be compared like for like as structural maritime timber pine sawn timber that is used nowadays. The NP 4305-1995 by LNEC, states that are two kinds of timber that can be used based on its quality, which are for structures (E) and special structures(EE), being the last the highest grade. Table 10 shows the parameters adopted for the lowest grade of timber, from an extract of file M2 (table IV – Shown in Annex B) developed by LNEC (Cruz et al., 1995)

Table 10 – Parameters adopted for the wood in frontal walls and floors

E_{wood} (MPa)	12000
ρ_{wood} (kg/m^3)	580
ν	0.2

Structural elements definition – Frontal walls

The “frontal” walls have been modelled according to the study developed by Simões A. et al, where is proposed to model a frontal wall, manipulating the properties of an equivalent masonry knowing these can be located in any floor above the ground floor (H. Meireles et al., 2022)

To model these walls, is important to carry a survey on site and if possible have these walls completely exposed. Although, the possibility of this walls not being exposed in its integrity is high, the study allows the user to make an assumption as it is also based on the geometry of the wall. When carrying out a survey a floor plan should be marked up to highlight the walls that are actually a frontal wall. Frontal walls do have different configurations, the most common are presented in Figure 46, along with the length and hight. Also on **Erro! A origem da referência não foi encontrada.** can be seen a wall example for the case study.

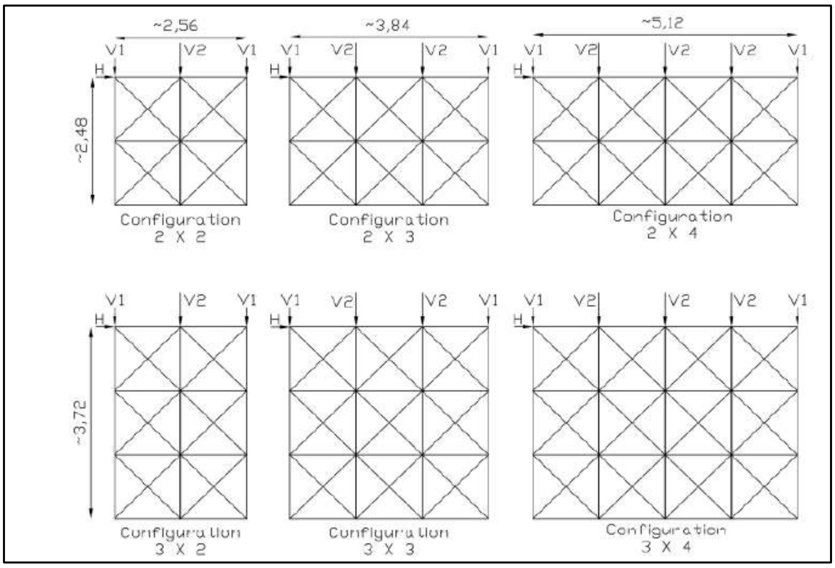


Figure 46 – Most common configurations of frontal walls image from (H. A. Meireles et al., 2012)

To summarize, each configuration should be modelled with an equivalent reinforced masonry wall. The following steps have been considered:

- A thickness of 120-150mm and the remaining dimensions (H e L) (high and length), should have values approximately the same as the 2nd column of Table 11;
- The E and τ_o Values (E-Young modulus and τ_o Shear resistance), shown on columns 4th and 7th of Table 11, respectively;
- fm (average resistance to compression), does not affect the behaviour of the reinforced wall, therefore a 6Mpa can be adopted;
- At each end of the wall a reinforcement bar should be added, this vertical bar with an area (Ac) shown on 8th column, should be added at 10cm from the edge
- A displacement interfloor should be considered at 3,5% (drift=3,5%) for shear and bending.

Table 11 – Proposed properties to model "frontal" walls in Tremuri® (adapted from (H. Meireles et al., 2022))

Configuração	h/L/t [m]	$K_{0,bilinear}$ [kN/m]	E [MPa]	N [kN]		$V_{u,bilinear}$ [kN]	τ_0 [kN/m²]	A_c [cm²]	D_c [cm]
C2x2	2,55	2842	**252**	Piso 1	76,8	46,6	**70,8**	**2,32**	**10**
	2,55			Piso 2	51,0	50,14	**100,0**		
	0,12			Piso 3	25,5	53,44	**100,0**		
C2x3	2,55	7007	**339**	Piso 1	114,8	65,8	**62,5**	**2,00**	**10**
	3,825			Piso 2	76,5	72,08	**83,5**		
	0,12			Piso 3	38,3	78,39	**109,0**		
C2x4	2,55	14685	**492**	Piso 1	153,6	81,06	**56,0**	**1,67**	**10**
	5,1			Piso 2	102,4	87,98	**72,0**		
	0,12			Piso 3	51,2	94,34	**98,0**		
C3x2	3,825	1418	**265**	Piso 1	76,8	46,5	**133,0**	**3,38**	**10**
	2,55			Piso 2	51,0	50,77	**160,0**		
	0,12			Piso 3	25,5	52,05	**170,0**		
C3x3	3,825	3022	**270**	Piso 1	114,8	62,77	**58,0**	**2,81**	**10**
	3,825			Piso 2	76,5	67,82	**85,0**		
	0,12			Piso 3	38,3	73,09	**105,0**		
C3x4	3,825	6036	**345**	Piso 1	153,6	81,79	**55,0**	**2,48**	**10**
	5,1			Piso 2	102,4	87,58	**75,0**		
	0,12			Piso 3	51,2	93,38	**100,0**		

Loads Definition

Once the model is calibrated, the loads shown on Table 12 need to be added, therefore considered in the pushover analysis. These loads will allow the push over analysis to be more accurate, as the analysis will cover for a more conservative side. These loads are in line with the study developed by H.A. Meireles and R. Bento and the NP EN 1991-1 (actions on structures).

Table 12 – Loads considered for the model, from NP EN 1991-1

Load due to:	Dead Load	Variable load	Applied in:
Insternal walls	0.1 kN/m2	-	Floor
Wooden floor	0.7 kN/m2	2,0 kN/m2	Floor
Ceilings	0.6 kN/m2	-	Floor
Stairs	0.7kn/m2	4.0 kN/m2	Stairs
Roof	1.1 kN/m2	0.4 kN/m2	Roof

Seismic action

The seismic assessment of existing masonry buildings is based on the procedures included in the NP EN 1998-3 : 2017(Annex C) and corresponding to Portuguese National Annex which establish the performance requirements and compliance criteria for existing buildings subjected to a certain level of seismic action. (Bernardo et al., 2021)

Knowing the location of the building, the presumed ground type and the importance class of the building, allowed to determine the response spectrum. On Table 13 is presented the values considered for the case study for both Earthquake (EQ) type 1 and type 2.

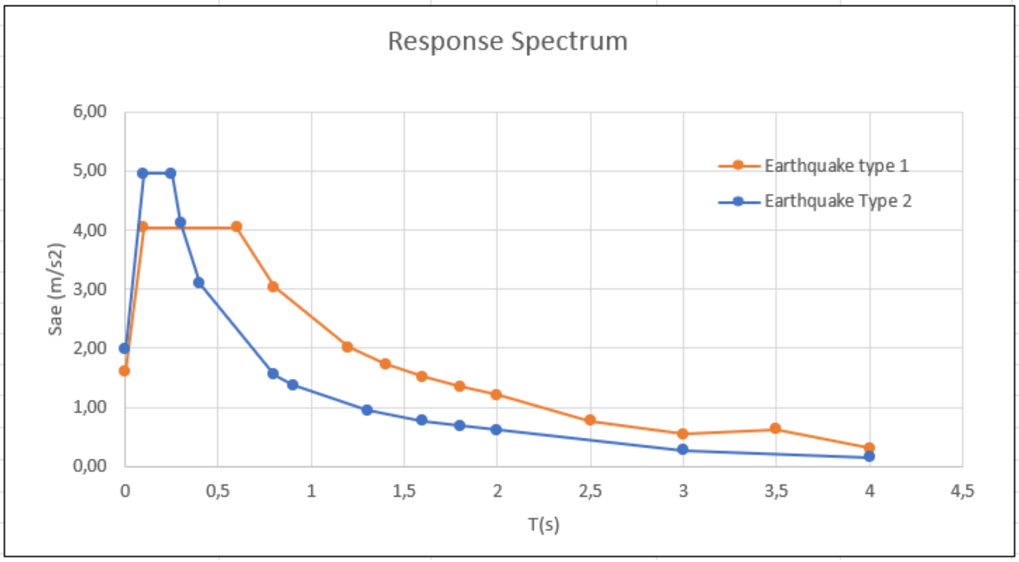
Is important to always determine the response spectrum for both types of earthquake. There is a possibility of on type of Earthquake to be more damaging than the other, also there is a possibility of an analysis to be carried out for both types if it justifies.

Graph 1 below, shows both response spectrums for EQ type 1 and 2. Is clear that for the major part of periods of the structure, EQ type 1 is more demanding, however for the low values of the period (where there is high frequencies) the EQ type 2 is more demanding. This way, both seismic actions must be considered on the analysis.

Table 13 – Parameters adopted following NP EN 1998-3 base on the building location and its future use

Seismic Action definition		
Location	**Ground type**	**Importance class**
Lisboa	C	II
	EQ type 1	**EQ type 2**
Seimic zone	1,3	2,3
Acceleration agr (m/s2)	1,0125	1,2852
Smáx	1,6	1,6
TB(s)	0,1	0,1
TC(s)	0,6	0,25
TD(s)	2	2
γ_I	1	1
Ground acceleration agr (m/s2)	1,0125	1,2852
Parameter S	1,5975	1,54296
η	1	1

Graph 1 – Response Spectrum for Seism type 1 and 2 in Lisbon down town



5. VULNERABILITY ANALYSIS, APPLICATION AND RESULTS

Model calibration

The calibration of the model under analysis is a very important step to validate the work. Once the model is all ready to run (meaning that all the checks have been carried out as Tremuri requires), a modal analysis can be carried out. Running a modal analysis means that we can obtain the model frequencies which should be compared with the experimental frequencies obtained on site.

To compare the frequencies obtained from the model and the frequencies obtained on site (the natural frequencies of the building), it is imperative that the model is prepared in line to what is on site. This way, the final results can be more accurate. For a better understanding, at the time the frequencies have been measured the building was completely stripped out, being left with no furniture or any other equipment.

With the assistance of Professor Carlos Sousa Oliveira, a survey to register the natural frequencies of the building has been conducted. It is important to define the axis to which the frequencies are being read, therefore the axis from Rua do Alecrim to Rua das flores has been considered as the Y-Y axis, and the X-X axis refers to the grid along Rua do Alecrim. Figure 47 shows the axis definition adopted in plan.

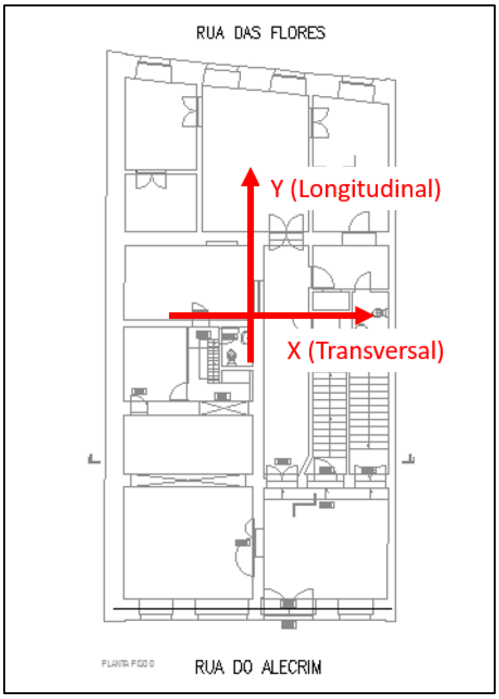


Figure 47 – Axis definition

To measure the frequencies on site an accelerometer has been used. The accelerometer has been installed on level 5 onto the parapet to the east side (Rua do Alecrim side). With the assistance of 16 people, push tests have been carried onto a wall on longitudinal direction and the same has been done for a wall on transversal direction. These push tests allowed the accelerometer to register the frequencies as shown on Figure 48.

Mod1: f=2.95Hz (longitudinal)
Mod2: f=4.3Hz (Torsion)
Mod3: f=4.9Hz (Transversal)

Figure 48 – Natural frequencies obtained on site following the push by 16 people

Following NP EN 1998-3:2017, by carrying a seismic assessment to a building, the stiffness of the walls should be assessed taking into account both their bending and transverse deformability, based on the fixed stiffness. In the absence of more accurate assessments, the two contributions to stiffness may be considered equal to half of their uncharted values. As there is an assessment carried on site where frequencies have been registered, the two contributions to stiffness have been adjusted in Tremuri®, enough to obtain values close to the ones read on site.

By running a modal analysis with the properties shown on previous chapter, Tremuri® delivered the following Period values for both directions as shown on **Erro! A origem da referência não foi encontrada.**.

Table 14 - Modal analysis 1 with the initial value properties for the materials.

Active in pushover		Mode	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
X dir.	Y dir.								
☐	☐	1	0.64940	1 550.451	0.05	3.256	0.00	2 495.835	0.08
☐	☐	2	0.54824	1 613 855.496	54.02	589.031	0.02	11.803	0.00
☐	☐	3	0.50617	4 104.025	0.14	0.360	0.00	1 220.143	0.04
☐	☐	4	0.49199	4 432.401	0.15	0.070	0.00	1 465.461	0.05
☐	☐	5	0.33445	11 381.171	0.38	21 059.856	0.70	14.079	0.00
☐	☐	6	0.28642	660.531	0.02	1 878 580.294	62.88	146.413	0.00

As shown on Table 1 mode 2 represents the mode in axis X and mode 6 represents the mode in direction Y. With the Period (T[s]) results, it is clear that the initial properties adopted for the materials should be changed. Table 15 shows the frequencies (f) obtained as a result of the modal analysis 1.

Table 15 - Frequencies (f) from modal analysis 1

Mode	T(s)	f(Hz)
2	0,548	1,824
6	0,286	3,491

Looking into more detail to Table 15, the frequency values are not yet in line with the target shown on Figure 48. This means the stiffness of the materials are required to change. By playing with the materials stiffness, Tremuri processed the new properties values, which brought values that are very close to the reality. Table 16, shows the properties of the materials worked out to calibrate the model.

Table 16 - Properties adjusted to calibrate the model in line with the site results

Material properties (Gpa)		
Material type	**E (Elasticity modulus)**	**G (Distortion modulus)**
Stone masonry	5,46	1,68
Rubble masonry	2,39	0,79
Brick masonry	2,92	0,97
Frontal Walls	4,51	4,51

It is important to note, that the block effect has not been considered in this model, which means the building is being assessed on its own. By assessing the building on its own, means the influence of the adjacent buildings is not being taken directly into account. To do that, a full survey of the block needed to be considered and inserted into a model analysis. For the analysis conducted, it is acknowledged the values in Table 16 are higher than the reality, although this was adopted to consider the block influence into the model.

Table 17 – Modal analysis 2 with the material properties adjusted.

Active in pushover X dir.	Active in pushover Y dir.	Mode	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
☐	☐	1	0.64869	56.704	0.00	0.527	0.00	2 309.123	0.08
☐	☐	2	0.50612	124.210	0.00	1.208	0.00	1 150.728	0.04
☐	☐	3	0.49051	230.764	0.01	1.202	0.00	1 204.085	0.04
☐	☐	4	0.37327	1 609 747.499	54.59	1 400.073	0.05	22.711	0.00
☐	☐	5	0.23625	31 118.345	1.06	6 963.139	0.24	40.799	0.00
☐	☐	6	0.21336	1 717.196	0.06	1 938 228.965	65.73	1 142.517	0.04

Table 17 mode 4 represents the mode in axis X and mode 6 represents the mode in direction Y. Looking at the results, the frequencies are now closer to the results obtained on site as shown on Table 18.

Table 18 - Frequencies (f) from analysis 2

Mode	T(s)	f(Hz)
4	0,373	2,681
6	0,213	4,687

Now, that the model has been calibrated, the remaining loads can be applied accordingly and proceed to the push over analysis.

Pushover analysis

For the case study, only a global verification will be done to verify the safety of the building. In case of the building not verifying the safety requirements, according to NP EN 1998-3:2017, the demand and capacity values can be compared to the individual structural elements, to understand which of these need to be reinforced. The process to do this verification, will be done using pushover analysis according to the N2 Method on NP EN 1998-1:2010.

The Pushover Analysis is a nonlinear static method for the seismic analysis of structures. A pre-determined lateral load is applied onto the structure and steadily increased to identify yielding and plastic hinge formations and the load at which failure of the various structural components occurs. (FajFar, 2018)

Pushover curve is a representation of a nonlinear static analysis under constant gravity loading and monotonically increasing lateral forces. The pushover curve is represented with base shear force and the control displacement (top displacement), in which is provided the capacity of the structure. Is important to note, the curve is only a representation to the capacity of the structure and does not give directly the demands associated with a particular level of seismic action. In this analysis is assumed that the response is governed by a single mode of vibration and that it is constant during the analysis. (Dogariu, 2011)

The distribution of lateral forces (applied at storey masses) for the analysis will be according to the NP EN 1998-1:2010 as shown on chapter two of this document. Lateral forces can be modal (triangular/static)) or uniform (lateral forces proportional to the storey masses). In Figure 49 is an example of these loads.

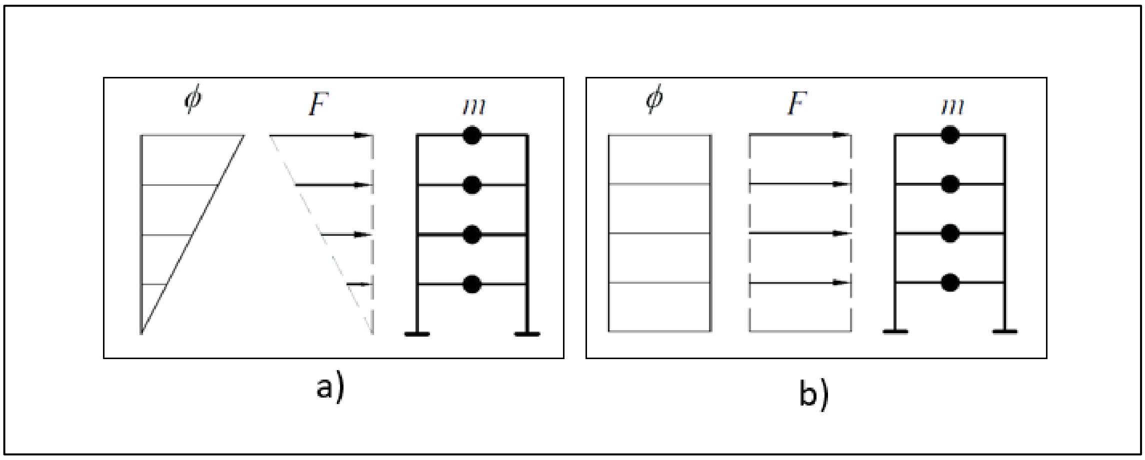


Figure 49 – Representative example of a) Lateral forces modal and b) uniform loads, adapted from (Dogariu, 2011)

Capacity curves

The Capacity Curve or Pushover Curve represents the nonlinear behaviour of the structure and is a load-deformation curve of the base shear force versus the horizontal roof displacement of the building. Pushover analysis transforms a dynamic problem into a static problem.(FajFar, 2018)

As the model is now calibrated and the loads are put in place for the category of the building, it is now possible to extract the capacity curves as a result of the distribution of the lateral forces.

All the capacity curves are defined in relation to a control node on the top level of the structure. For the case study the control node is the 24 as shown on Figure 50 in representation of level 5 (top floor). For a better understanding, Figure 70 in annex A, shows the thickness, type of walls and its distribution on the plan.

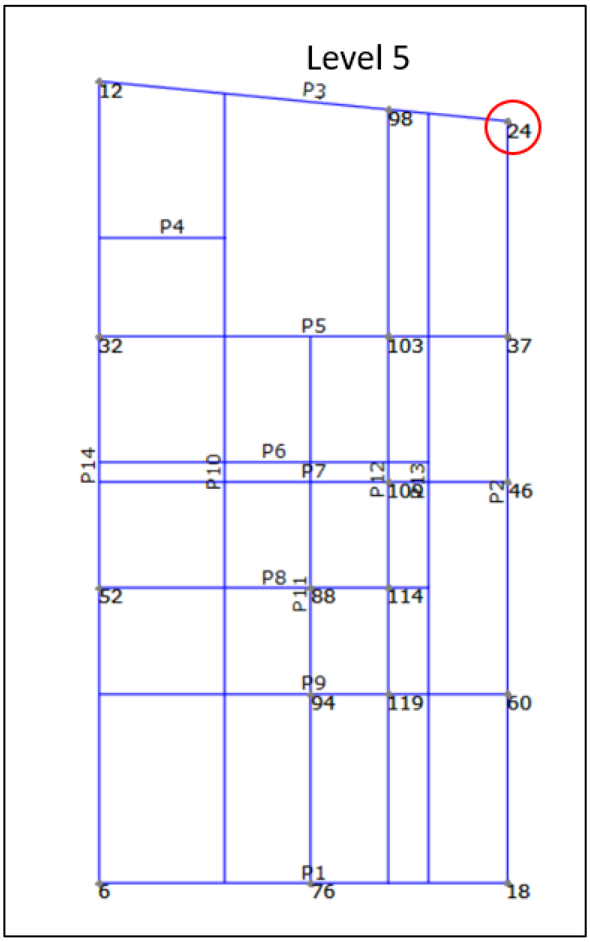
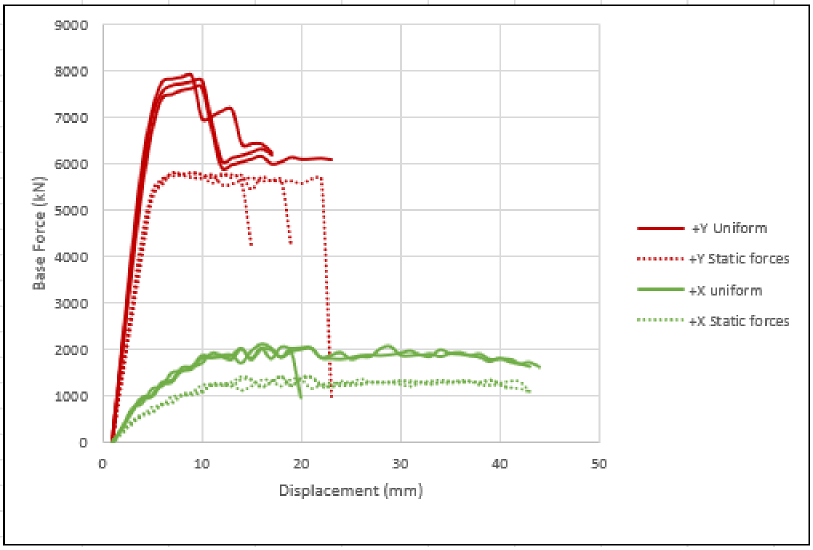


Figure 50 – Level 5 (top floor) showing the control note adopted for the case study

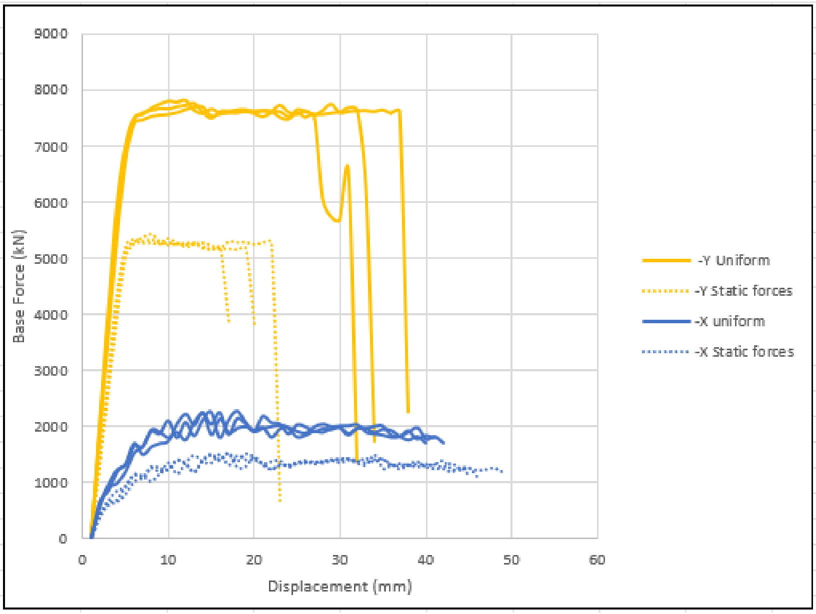
At the end of chapter 4, the analysis of the response spectrum demanded that capacity curves must be compared for both earthquake type 1 and 2. Graph 2 and Graph 3, represent the capacity curves in relation to the control node. Looking into more detail to these two graphs, it is clear that the forces on X direction are the ones showing more sensibility, as the yielding point is lower. Nevertheless, considering the perimeter walls as the main walls as these go all the way up from the ground foundations, the X direction represents the lower values of both directions. Perhaps, one of the reasons for this to happen, might be linked to the fact that in this direction, (when looking at the elevations), does have more openings than the Y direction (which has got solid walls on both of elevations).

For both directions, a static/triangular load and a uniform load are shown on Graph 2 and Graph 3, where for the vulnerability assessment, the static loads are showing lower values to its yielding points, as a result the analysis will be done based on a static/triangular load.

Graph 2 – Capacity curves for EQ type1 the positive direction “X” and “Y”



Graph 3 - Capacity curves for EQ type 2, for the negative direction “X” and “Y”.

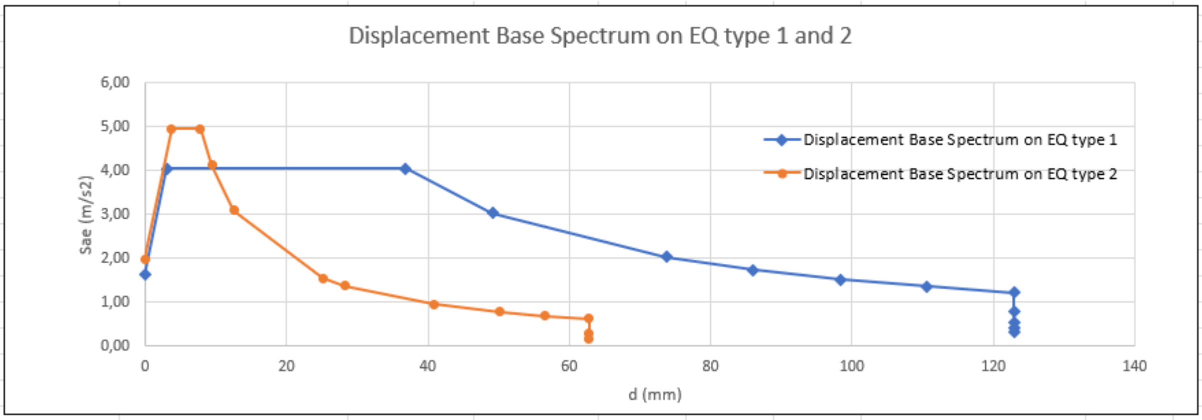


N2 Method following NP EN 1998-1:2010

The annex B of NP EN 1998-1:2010, shows a method to determine the target displacement based on pushover (non-linear static analysis).

The target displacement is determined from the elastic response spectrum (as shown on the previous chapter) and the intersection of this with the capacity curve. On Graph 4 is represented the displacement base spectrum on earthquake type 1 and 2, determined according to 4.3.3.4.2.3 of NP EN 1998-1:2010, which will be used to compare with the ideal capacity curve of each analysis, therefore to understand visually if the analysis is on Low or medium/long Periods (this will be explained ahead on this chapter).

Graph 4 – Displacement base spectrum on EQ type 1 and 2



Having the response spectrum determined, there will be a need to transform the structure that corresponds to a multiple degree of freedom system (MDOF) into a single degree of freedom, therefore obtaining a capacity curve based on an equivalent system of a single degree of freedom system. As a starting point, it is important to determine the mass of the equivalent system (m*) based on the following:

$$m^* = \Sigma m_i \varphi_i = \Sigma F_i \tag{11}$$

where:

m*- mass of the equivalent system

m_i - mass per floor

φ_i – displacement of floor (i)

F_i – Lateral forces acting in floor (i)

The mass of the equivalent system will be used to determine the transformation coefficient needed to obtain the capacity curve of MDOF to a SDOF. Following the annex B of NP EN 1998-1:2010 the transformation coefficient is given by:

$$\Gamma = \frac{\Sigma i\ mi\ \varphi i}{\Sigma i\ mi\ \varphi i^2} = \frac{m^*}{\Sigma i\ mi\ \varphi i^2} \tag{12}$$

where:

m*- mass of the equivalent system

m_i – mass per floor

φ_i – displacement of floor (i)

The software used on the case study (Tremuri®) determines the transformation coefficients which are represented on Table 19 for the case study:

Table 19 – Transformation coefficients obtained from Tremuri

Transfomation Coefficients (Γ)	
Γx	Γy
1,25	1,37

In view of doing a global analysis to the structure, following the annex B of NP EN 1998-1:2010 for severe damage (SD), the target displacement should not be higher than 3/4 of the ultimate displacement (d_u) allowed by the structure. On Figure 51, it is shown a graph with the different limit states and its relation to the capacity curve and its ultimate displacement (d_u). Represented in Figure 51 are three limit states, Damage limitation (DL), Severe damage (SD) and Imminent collapse (NC), where there is a permissible displacement for each. "Dy" is the displacement found at yielding point, where "Du" is the ultimate displacement.

Figure 1

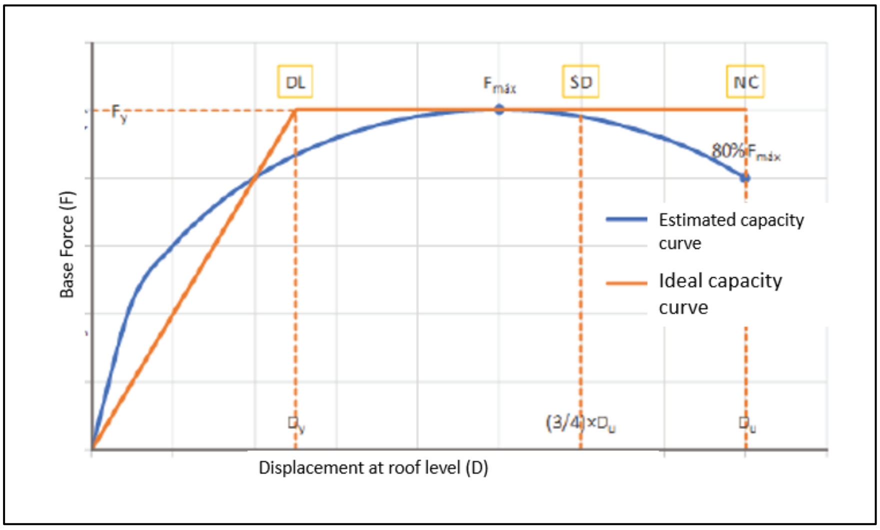


Figure 51 – Global capacity curve showing the different limit states adapted from (Candeias et al., 2020a)

The target displacement will change accordingly, as the Period (T*) obtained from each incremental analysis. Therefore, this period (T*) will depend on the Force (F*) applied on each analysis and is given by the following:

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} \qquad (13)$$

where:

d_y^*- or Du is the displacement at yielding point

F_y^* – Yielding force ($F_y^* = F_b/\Gamma$), F_b is the base force.

The target displacement of a structure with a Period (T*) and non-limited elastic behaviour is given by the following:

$$d_{et}^{*} = S_e(T^*)\left(\frac{T^*}{2\pi}\right)^2 \tag{14}$$

where:

$S_e(T^*)$ is the elastic spectrum response of the acceleration for a period T*

To determine the target displacement (d^*_t), for the structures with low periods and for these with medium to high periods the following expressions must be used:

Low periods where T*<Tc:

If $F_y^*/m^* \geq Se(T^*)$, it means there is an elastic response, therefore:

$$d_t^* = d_{et}^* \tag{15}$$

If $F_y^*/m^* < Se(T^*)$, it means the response is nonlinear, therefore:

$$d_t^* = \frac{d_{et}^*}{q_u}\left(1 + (q_u - 1)\frac{Tc}{T*}\right) \geq d_{et}^* \tag{16}$$

Where q_u is the relation between the acceleration in the structure with non-limit on elastic behaviour $S_e(T^*)$, and the structure with limited resistance F_y^*/m^*:

$$qu = \frac{Se(T^*)m^*}{F_y^*} \tag{17}$$

Medium to high Periods where T*>Tc:

$$d_t^* = d_{et}^* \tag{18}$$

The difference between low and medium/high Periods, is shown on Figure 52 for low Periods and on Figure 53 for medium/high Periods.

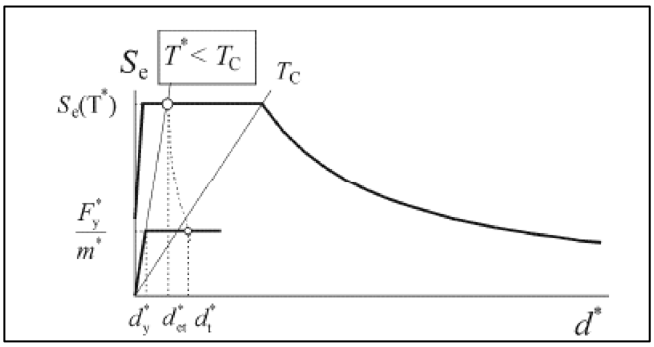


Figure 52 – Short Period representation (adapted from NP EN 1998-1:2010)

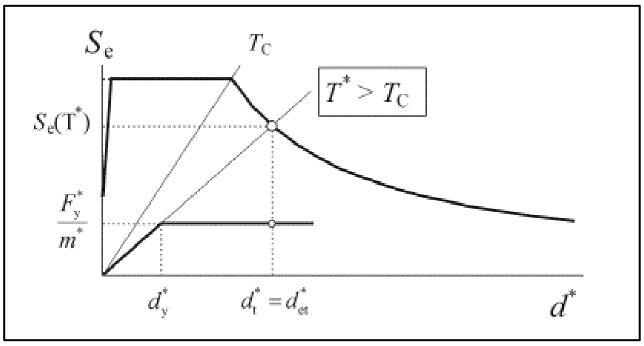


Figure 53 – Medium/Long Periods representation (adapted from NP EN 1998-1:2010)

Results

The Tremuri® software computes 24 capacity curves for each building, representing 12 curves in each direction X and Y. As mentioned above on the previous chapter and after analysing Graph 2 and Graph 3, the static analysis is the one that shows lower values on its yielding points, therefore the ones that show the highest vulnerability of the structure. Below are representations of each step that are represented in line with each capacity curve computed (Figures XX to YY).

As mentioned on the previous chapter, the analysis was be carried out for earthquake (EQ) type 1 and earthquake type 2. The analysis has been carried for X direction positive and Y direction negative either for the EQ type 1 or 2. These directions are the ones that show the most vulnerability of the building.

For EQ type 1 on X direction, the 3 capacity curves (on a static analysis) have been converted to an ideal capacity curve, which has been compared to the response spectrum and the analysis been carried. Figure 54 and Figure 56 represent the table of results from Tremuri on step 4 and 16 using for the calculation of the target displacement equation (16) above, as it is a Low Period case (T*<Tc). Both step 4 and 16 are showing no compliance according to NP EN 1998:3-2017, as shown on Figure 51 the target displacement should not be higher than ¾ of the ultimate displacement allowed by the structure. On Figure 55, it is the representation of step 15, however at this time the calculation of the target displacement has been done using equation (18) and (14), as it is a medium/long Period case. Regardless, no compliance has been verified, showing results above the ¾ of the ultimate displacement allowed by the structure.

For the EQ Type 1 Y direction and EQ type 2 X and Y directions, the same principle has been followed, which can be verified on Figure 57 to Figure 65, where all of the results show compliance according to NP EN 1998:3-2017, as the target displacement is not higher than ¾ of the ultimate displacement allowed by the structure on the control node.

In the Annex C, the different results obtained on the structure on each step are represented, where the damage caused on the structure can be verified, allowing to have a better understanding where the structure will require reinforcement.

	value	units
d*m =3/4 Du	31,89	mm
F*y =Fy	1061	kN
d*y=Dy	6,93	mm
m*	1396060,836	kg
T*	0,6	s
Se(T*)	4,04	m/s2
Fy*/m*	0,76	m/s2
q_u	5,32	
d^*_{et}	36,84	mm
d_t^*	36,84	mm

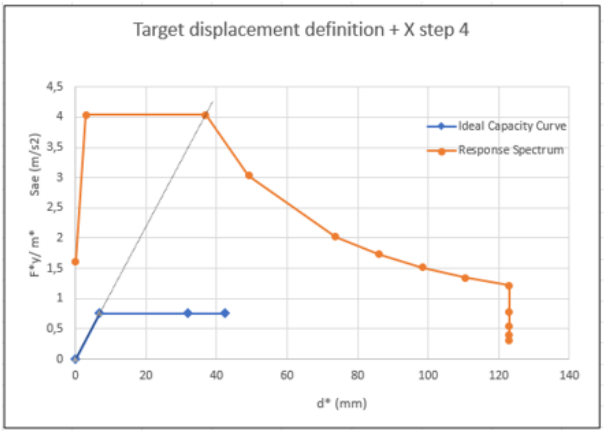


Figure 54 – Low Period on EQ type 1, X positive direction step 4

	value	units
d*m =3/4 Du	35,08	mm
F*y =Fy	1072	kN
d*y=Dy	7,41	mm
m*	1396060,836	kg
T*	0,617	s
Se(T*)	3,93	m/s2
Fy*/m*	0,77	m/s2
q_u	n/a	
d^*_{et}	37,90	mm

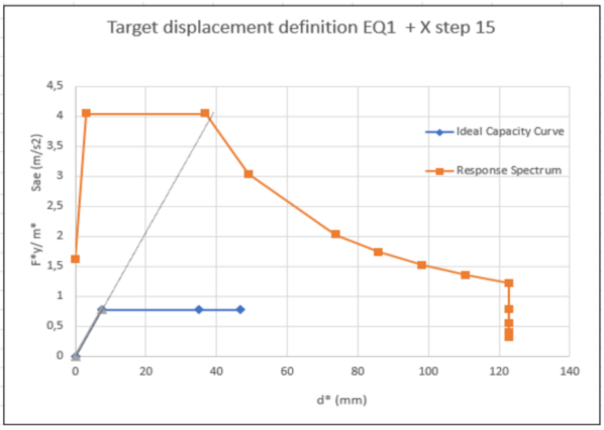


Figure 55 – Medium/Long Period on EQ type 1, X positive direction step 15

	value	units
d*m =3/4 Du	33,49	mm
F*y =Fy	1030	kN
d*y=Dy	6,26	mm
m*	1396060,836	kg
T*	0,579	s
Se(T*)	4,04	m/s2
Fy*/m*	0,74	m/s2
q_u	5,48	
d^*_{et}	34,31	mm
d_t^*	35,32	mm

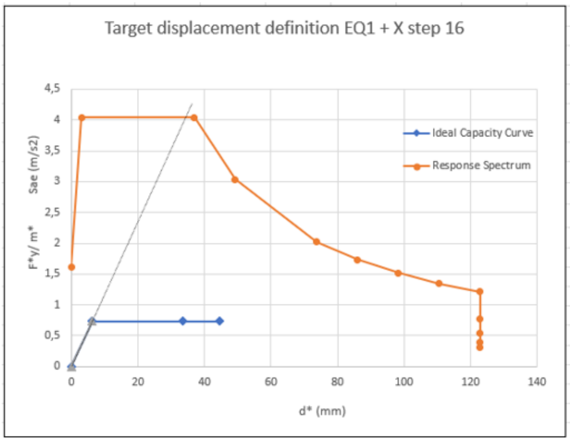


Figure 56 - Low Period on EQ type 1, X positive direction step 16

	value	units
d*m =3/4 Du	20,39	mm
F*y =Fy	3796	kN
d*y=Dy	2,8	mm
m*	1519147,64	kg
T*	0,21	s
Se(T*)	4,04	m/s2
Fy*/m*	2,50	m/s2
q_u	1,62	
d^*_{et}	4,51	mm
d_t^*	7,71	mm

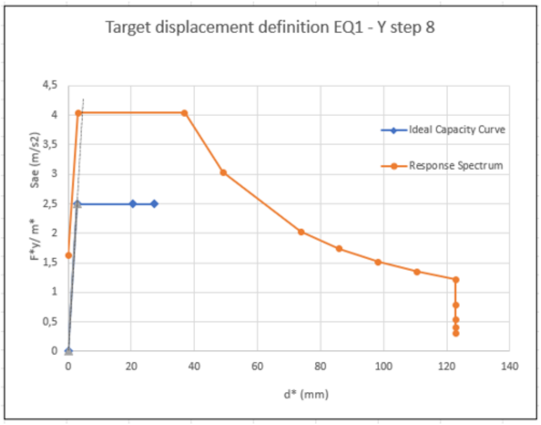


Figure 57 – Low Period on EQ type 1, Y negative direction step 8

	value	units
d*m =3/4 Du	35,69	mm
F*y =Fy	3660	kN
d*y=Dy	2,96	mm
m*	1519147,64	kg
T*	0,22	s
Se(T*)	4,04	m/s2
Fy*/m*	2,41	m/s2
q_u	1,68	
d^*_{et}	4,95	mm
d_t^*	8,41	mm

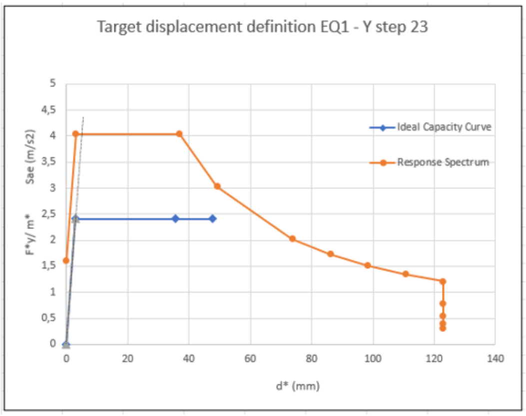


Figure 58 – Low Period on EQ type 1, Y negative direction step 23

	value	units
d*m =3/4 Du	14,57	mm
F*y =Fy	3832	kN
d*y=Dy	2,57	mm
m*	1519147,64	kg
T*	0,201	s
Se(T*)	4,04	m/s2
Fy*/m*	2,52	m/s2
q_u	1,60	
d^*_{et}	4,13	mm
d_t^*	7,22	mm

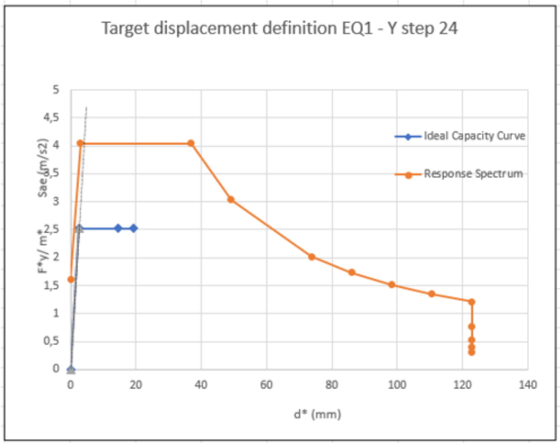


Figure 59 – Low Period on EQ type 1, Y negative direction step 24

	value	units
d•m =3/4 Du	31,89	mm
F•y =Fy	1061	kN
d•y=Dy	6,93	mm
m•	1396060,836	kg
T•	0,6	s
Se(T•)	2,07	m/s2
Fy•/m•	0,76	m/s2
q_u	n/a	
$d^{\bullet}_{et}$	18,88	mm

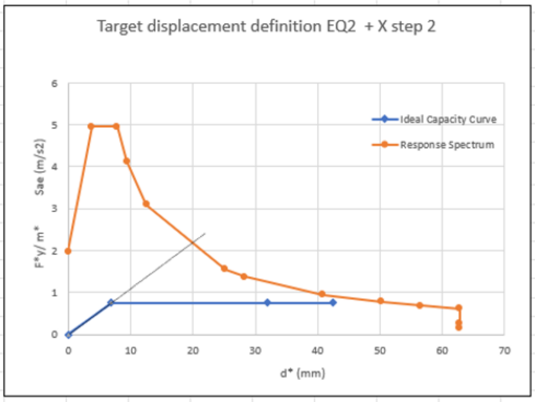


Figure 60 – Medium/Long Period on EQ type 2, X positive direction step 2

	value	units
d•m =3/4 Du	35,08	mm
F•y =Fy	1072	kN
d•y=Dy	7,41	mm
m•	1396060,836	kg
T•	0,617	s
Se(T•)	2,01	m/s2
Fy•/m•	0,77	m/s2
q_u	n/a	
$d^{\bullet}_{et}$	19,38	mm

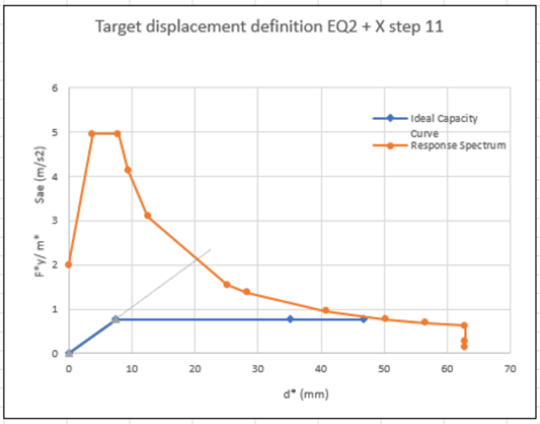


Figure 61 – Medium/Long Period on EQ type 2, X positive direction step 11

	value	units
d•m =3/4 Du	33,49	mm
F•y =Fy	1030	kN
d•y=Dy	6,26	mm
m•	1396060,836	kg
T•	0,579	s
Se(T•)	2,14	m/s2
Fy•/m•	0,74	m/s2
q_u	n/a	
$d^{\bullet}_{et}$	18,17	mm

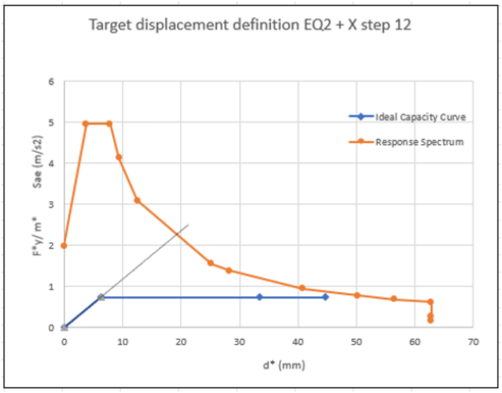


Figure 62 – Medium/Long Period on EQ type 2, X positive direction step 12

	value	units
d*m =3/4 Du	20,39	mm
F*y =Fy	3796	kN
d*y=Dy	2,8	mm
m*	1519147,64	kg
T*	0,21	s
Se(T*)	4,96	m/s2
Fy*/m*	2,50	m/s2
q_u	1,98	
d^*_{et}	5,54	mm
d_t^*	6,06	mm

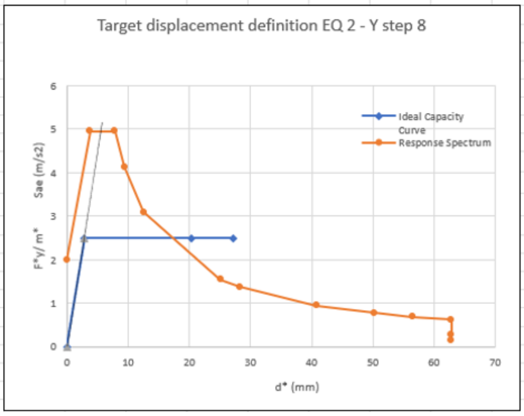


Figure 63 – Low Period on EQ type 2, Y negative direction step 8

	value	units
d*m =3/4 Du	35,69	mm
F*y =Fy	3660	kN
d*y=Dy	2,96	mm
m*	1519147,64	kg
T*	0,22	s
Se(T*)	4,96	m/s2
Fy*/m*	2,41	m/s2
q_u	2,06	
d^*_{et}	6,08	mm
d_t^*	6,51	mm

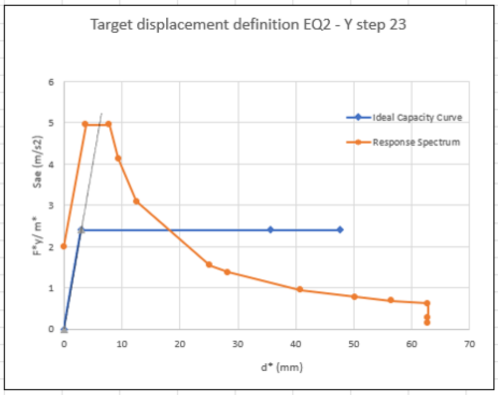


Figure 64 – Low Period on EQ type 2, Y negative direction step 23

	value	units
d*m =3/4 Du	14,57	mm
F*y =Fy	3832	kN
d*y=Dy	2,57	mm
m*	1519147,64	kg
T*	0,201	s
Se(T*)	4,96	m/s2
Fy*/m*	2,52	m/s2
q_u	1,97	
d^*_{et}	5,08	mm
d_t^*	5,68	mm

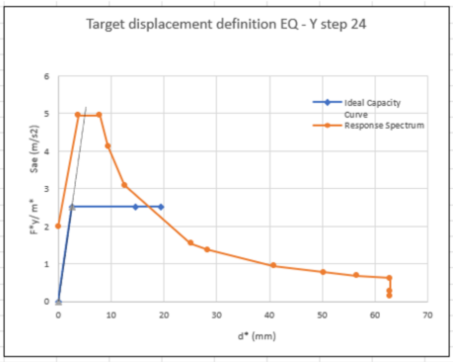


Figure 65 – Low Period on EQ type 2, Y negative direction step 24

6. CONCLUSIONS AND FURTHER DEVELOPMENTS

Pombalino buildings, are a representation of the patrimony built after the 1755 earthquake, as a result of a major improvement of the city to bring robustness and be more resilient to earthquakes. This buildings are mainly made out of masonry, stones and wood. As many years have gone through this buildings, some Pombalino buildings may have been well looked after than others and some have been subject to unexpected events that unfortunately could have damaged its structure. Sometimes, is unclear what changes one of these historical buildings have gone through, therefore and to preserve the origins, is important (when possible) to carry a site survey as in detail as possible. The more information is held the better understanding and results to the building are obtained.

The verification on the stability of an existing masonry building is carried to a global level, which must be compared to the demands and the capacities of the structural elements, aiming to identify the elements that require reinforcement. To do this, the capacity curves to be used must be similar to the ones shown in Figure 51, however using the average values of the properties of these materials. (Candeias et al., 2020)

The current regulations states if a building does not comply with the safety requirements referring to 90% of the seismic action defined on NP EN 1998-3:2017, it is mandatory to produce a seismic reinforcement project.

It is important to discuss analysis using macro elements, according to the study carried by Siano (Siano et al., 2018), where Siano stated that care must be taken on the assessment of existing structures, mainly historical buildings. The geometrical inconsistency of masonry walls are one o the main concerns of Siano, as these are seen as equivalent elements by assuming the masonry walls as, as well as the proposal that the structure will have a box behaviour, neglecting the collapse mechanisms out of plane. Siano, concluded that will be difficult to identify the equivalent elements when the distribution of openings are irregular. Lastly, the difficulties found when modelling the link between masonry wall and beam in confined structures or filled in.

The results obtained show that the structure in the X direction shows its highest vulnerability, as this is the direction where the analysis results are higher than the ¾ of the ultimate displacement allowed by the structure. As a result, a seismic reinforcement project is required, which means that a more meticulous analysis must be carried out for the elements that are part of the structure. The building itself shows the majority of the damages on main and rear elevations, perhaps justified by the amount of openings contained in these facades, as well the difference between the height of its stories which vary as it goes up.

The party walls show a lot more robustness and therefore do not bring as much damages as the main elevations; however, it is important to take in consideration the analysis has been done considering the building would stand on its own, which is known for a fact, that this is not the case, and the influence of the buildings either side have not been considered.

The building in its original state is very susceptible. It is thought to be because the floors are flexible. Simply by stiffening the floors, will bring an improved structure. Additional retrofitting of the structure is possible and advisable to increase its resistance towards earthquakes. The most profitable solution (which is also the most complicated in terms of implementation) is the inclusion of tie-rods in the lintels at front and back façades. The solution of inclusion of shear walls is advisable also but has architectural drawbacks; the solution of the inclusion of steel frames to all floors is advisable and has not much architectural drawbacks. Nevertheless, is important to highlight that there is out there much bibliography about the theme reinforcement, but is also important to always remember that any intervention must be compliant with the current regulations at the same time, as least intrusive as possible, reversible, durable and compatible with the existing materials.

Finally in this dissertation I would like to congratulate the RESIST PROGRAM, which I believe that should be expanded throughout the country as a method to adopt, and to bring the seismic resilience that Portugal needs which for several years seems to be forgotten. ReSist is thus based on five strategic objectives that are implemented in 48 actions to be implemented, with a view to standardizing technical standards and methodologies of seismic vulnerability assessment, including:

(i) the standardization of technical standards and methodologies for the evaluation of seismic vulnerability of the City;

(ii) the development of operational actions with a view to effectively improving the seismic resistance, achieved through inspection campaigns, projects and structural reinforcement works;

iii) carrying out awareness and dissemination campaigns to involve the society and empowerment of the population;

iv) the development of information management systems that streamline tasks knowledge sharing and programme implementation;

v) the definition and implementation of criteria and alerts

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Annexes

Annex A – Floor plans showing thickness an type of walls

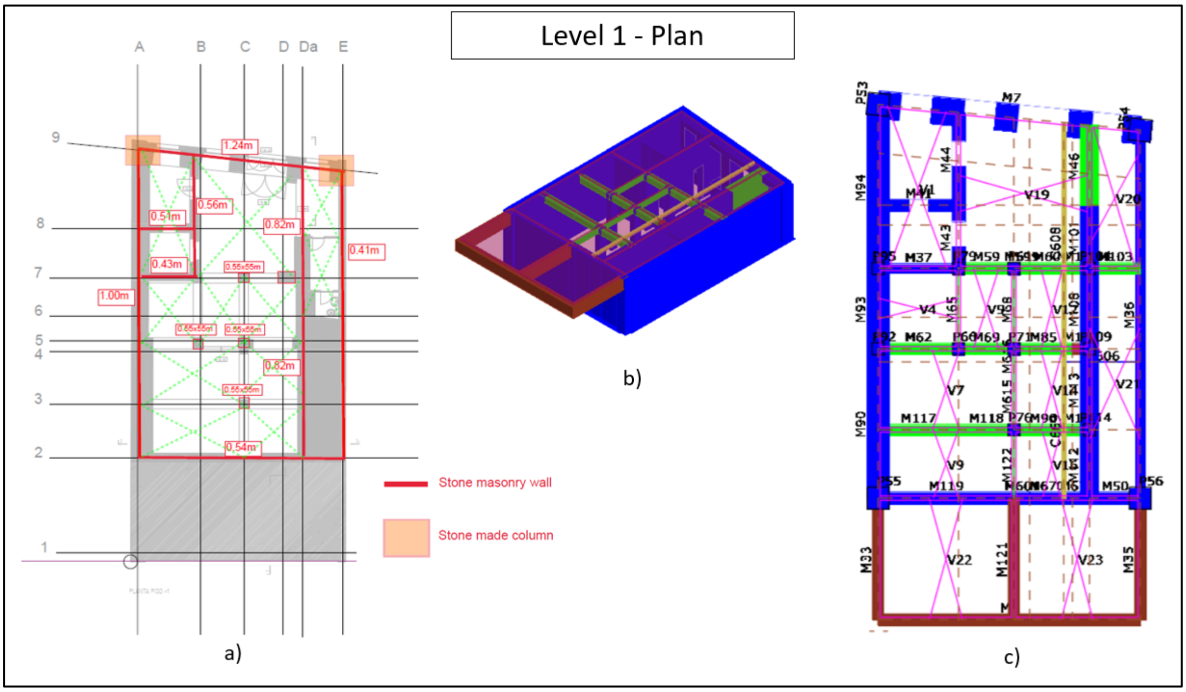


Figure 66 – Level 1, a) floor plan previously marked up following the site survey , b) 3D model in 3Muri, c) floor plan in 3Muri showing the different type of materials as per a)

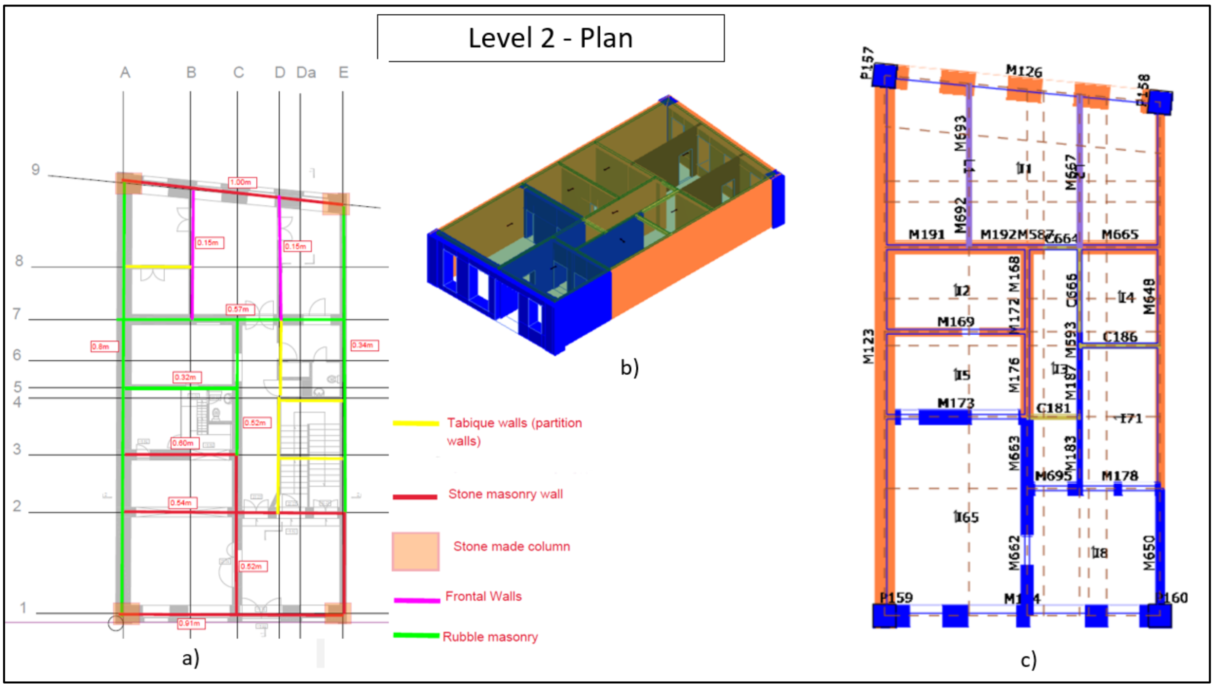


Figure 67 – Level 2, a) floor plan previously marked up following the site survey , b) 3D model in 3Muri, c) floor plan in 3Muri showing the different type of materials as per a)

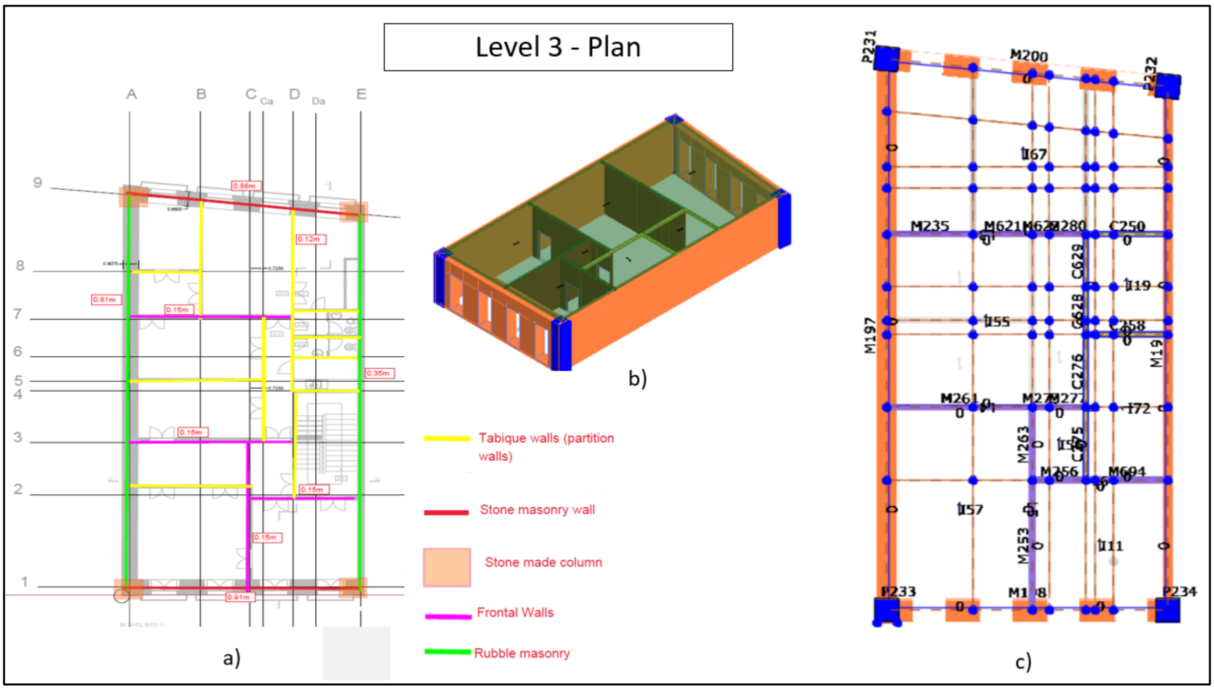


Figure 68 – Level 3, a) floor plan previously marked up following the site survey , b) 3D model in 3Muri, c) floor plan in 3Muri showing the different type of materials as per a)

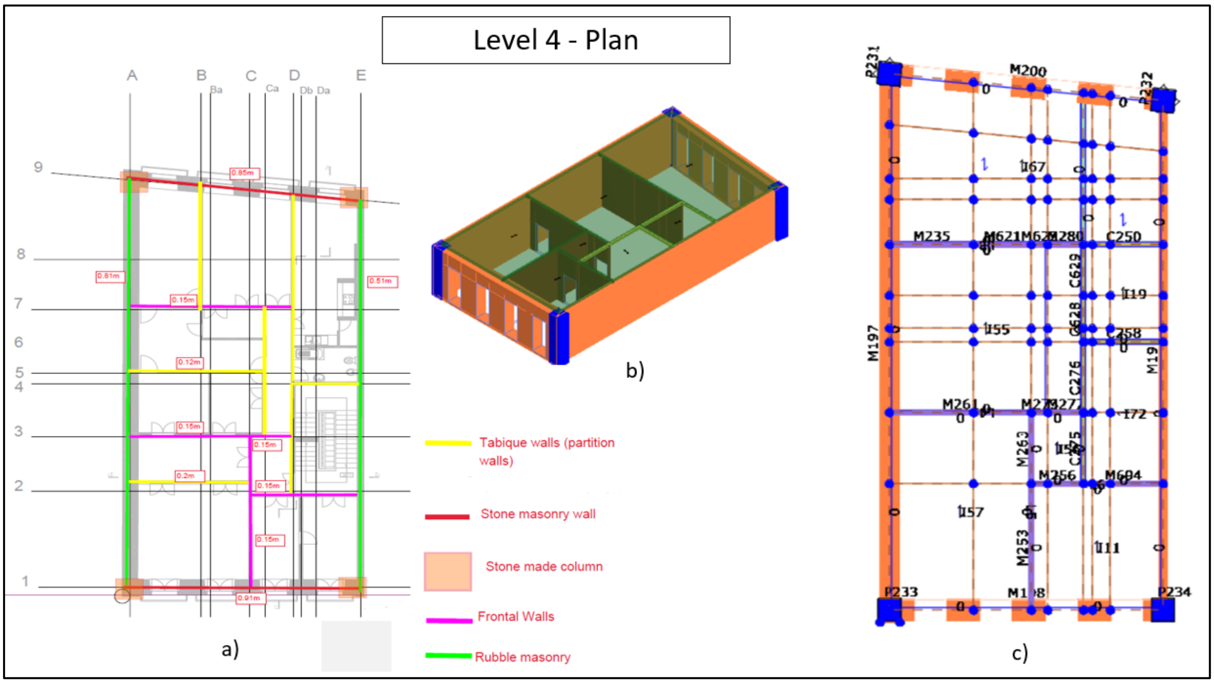


Figure 69 - Level 4, a) floor plan previously marked up following the site survey , b) 3D model in 3Muri, c) floor plan in 3Muri showing the different type of materials as per a)

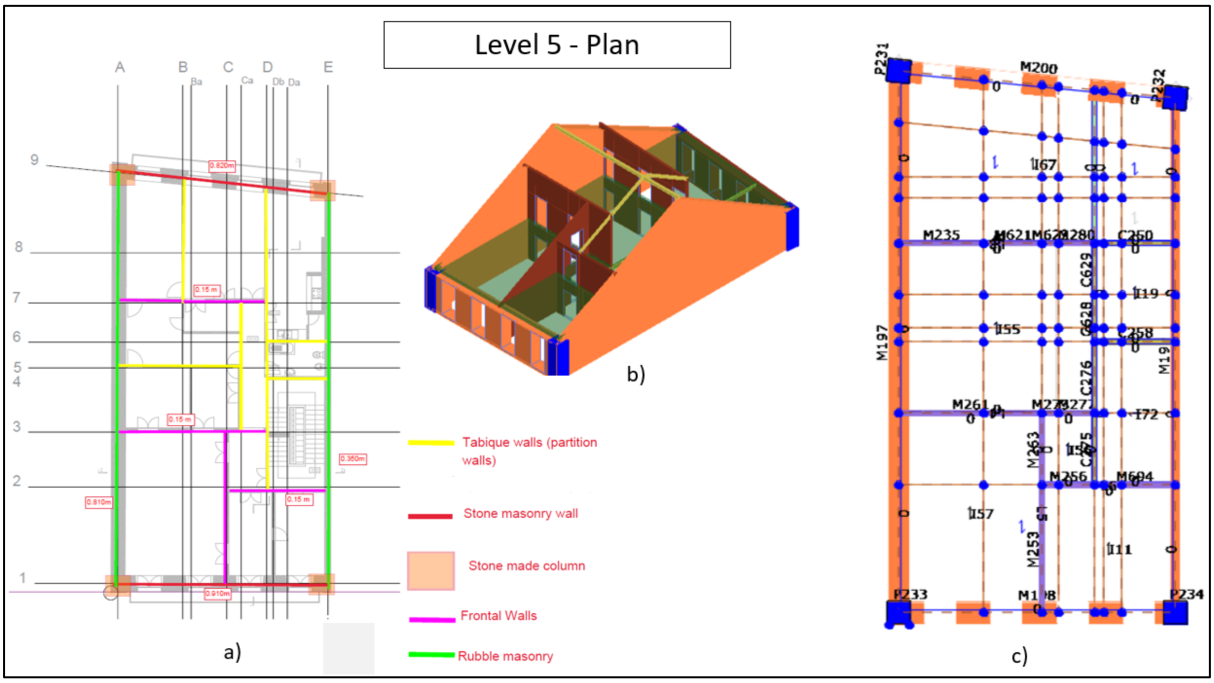


Figure 70 - Level 5, a) floor plan previously marked up following the site survey , b) 3D model in 3Muri, c) floor plan in 3Muri showing the different type of materials as per a)

Annex B – Table IV of LNEC M2 file, Structural maritime pine sawn timber

Propriedades Mecânicas		Classe de Qualidade	
		EE	E
(Valores característicos)			
Flexão estática (N/mm^2)	$f_{m,k}$	35	18
Tracção paralela às fibras (N/mm^2)	$f_{t,0,k}$	21	10,8
Tracção perpendicular às fibras (N/mm^2)	$f_{t,90,k}$	0,49	0,46
Compressão paralela às fibras (N/mm^2)	$f_{c,0,k}$	24,7	18
Compressão perpendicular às fibras (N/mm^2)	$f_{c,90,k}$	7,3	6,9
Corte (N/mm^2)	$f_{v,k}$	3,4	2,0
Módulo de elasticidade (kN/mm^2)			
Paralelo às fibras			
(Valor médio)	E_{mean}	14	12
(Valor característico)	$E_{0,05}$	9,38	8,0
Perpendicular às fibras (Valor médio)	E_{mean}	0,46	0,40
Módulo de distorção (kN/mm^2)	G_{mean}	0,87	0,75
Massa volúmica (kg/m^3)			
(Valor médio)	ρ_{mean}	610	580
(Valor característico)	ρ_k	490	460

Annex C – Graphic Tremuri Results

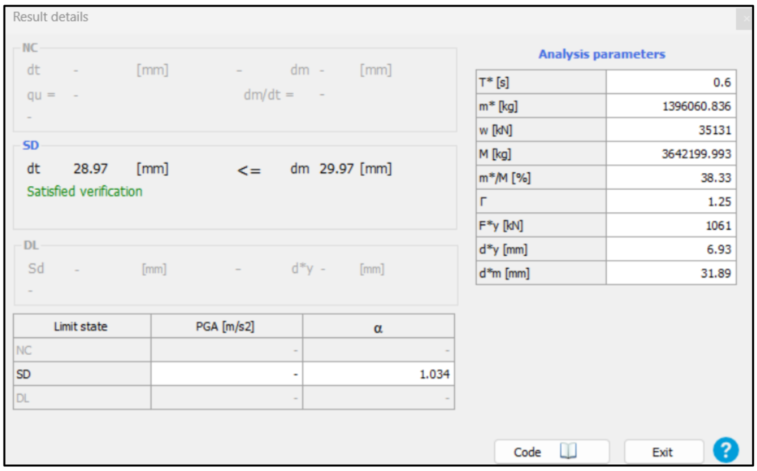


Figure 71 – Table of results EQ T2 step 2 | Static Forces

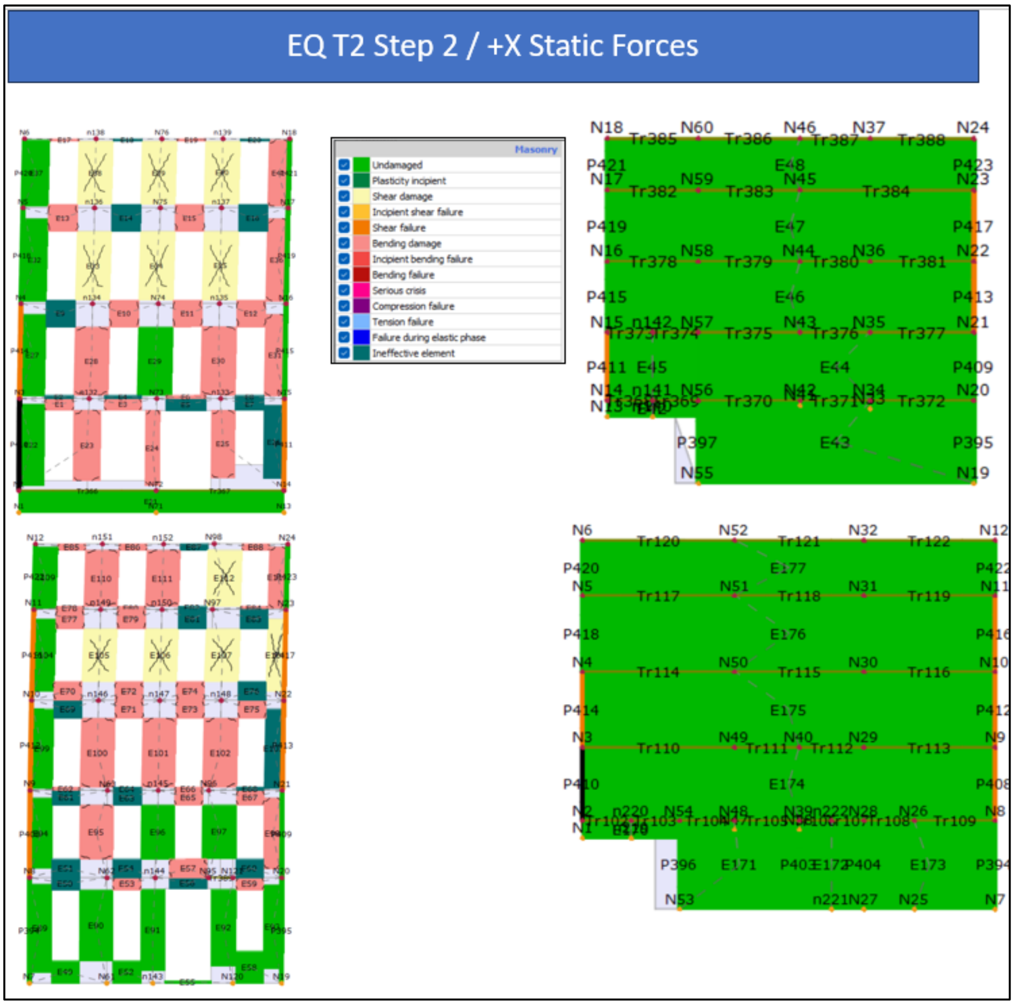


Figure 72 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

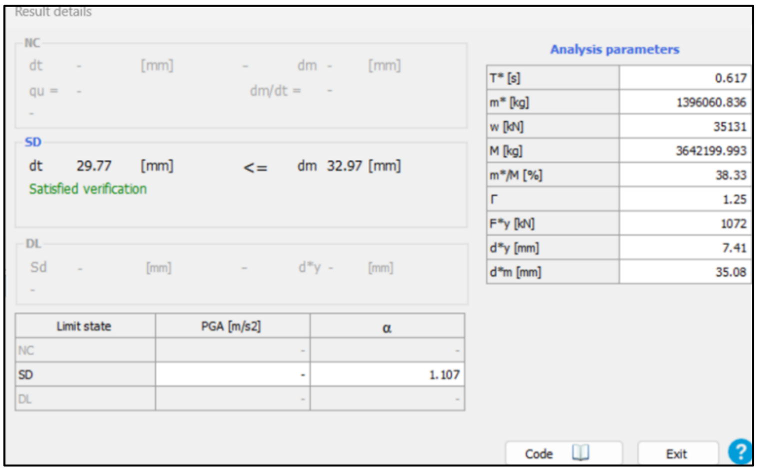


Figure 73 – Table of results EQ T2 step 11 | Static Forces

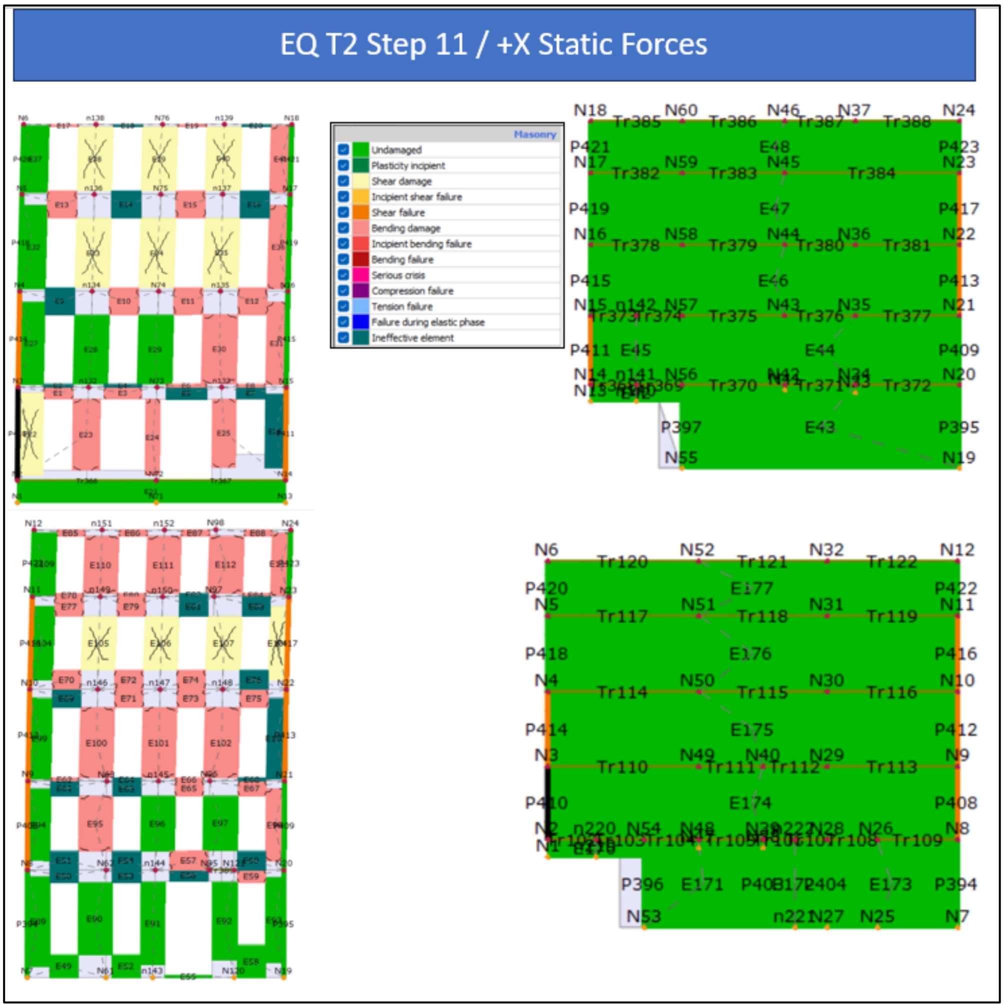


Figure 74 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

Figure 75 – Table of results EQ T2 step 8 | Static Forces

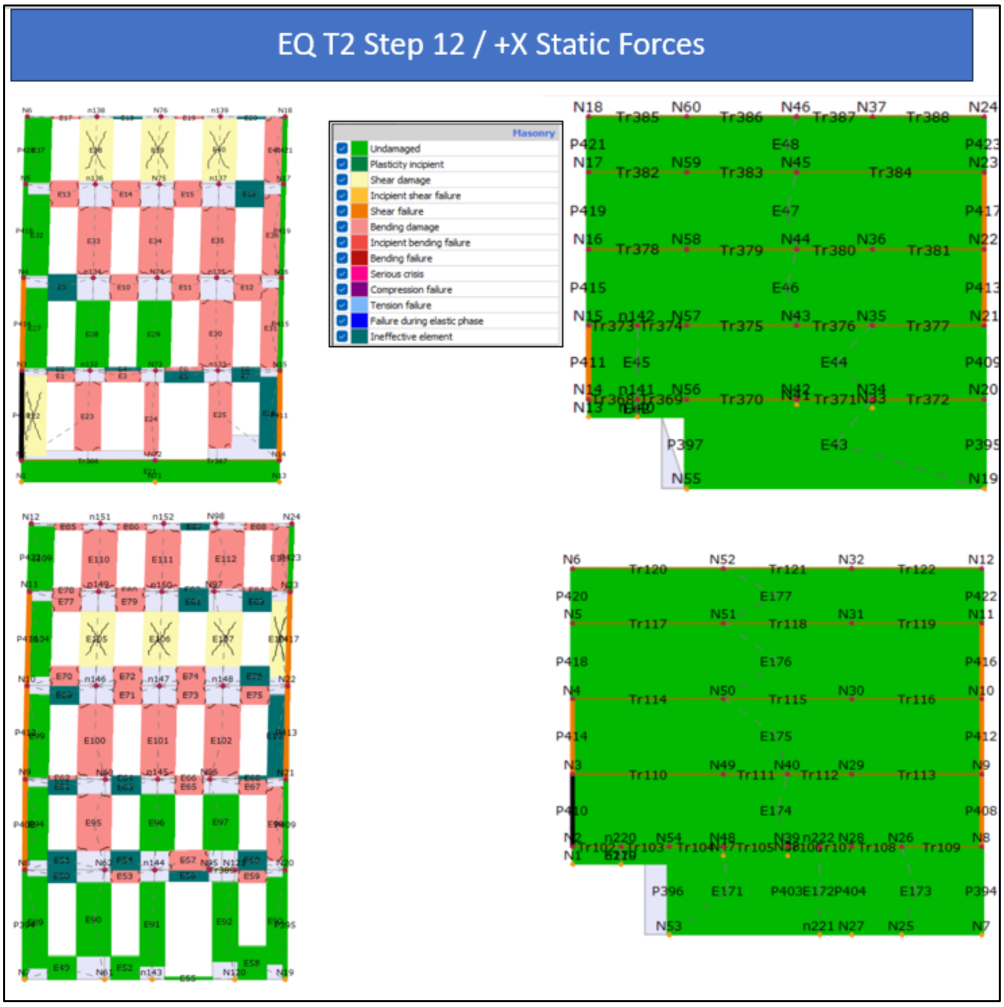


Figure 76 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

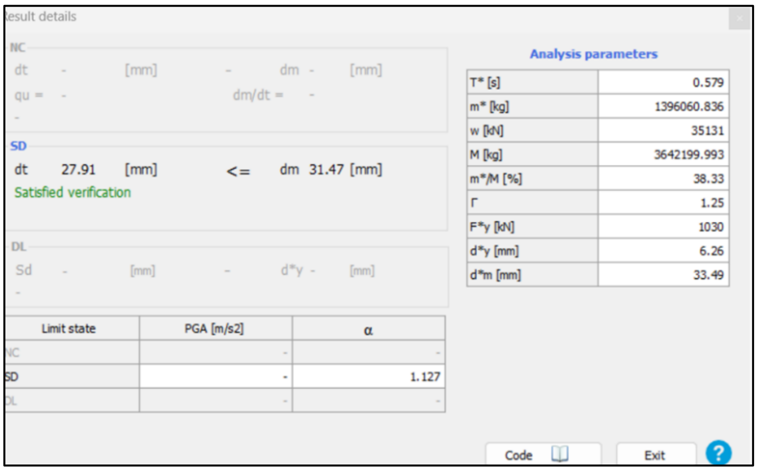


Figure 77 – Table of results EQ T2 step 8 | Static Forces

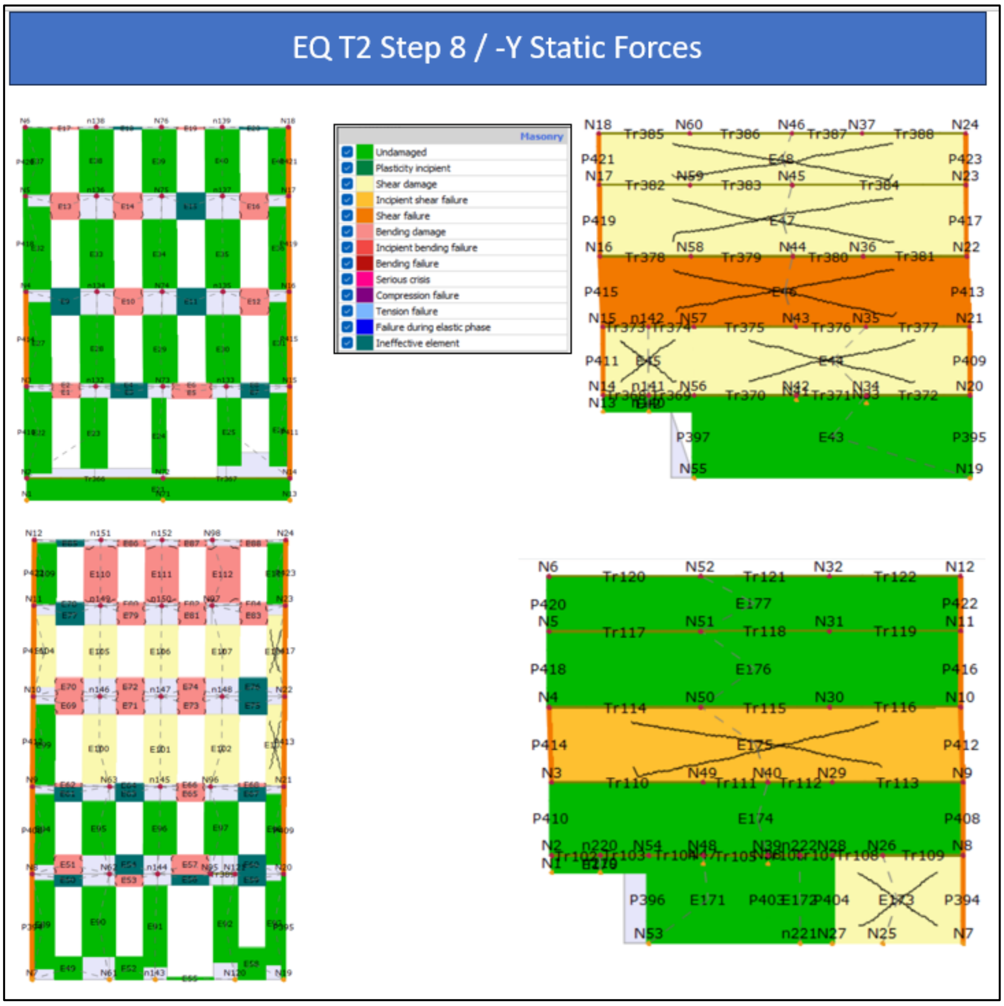
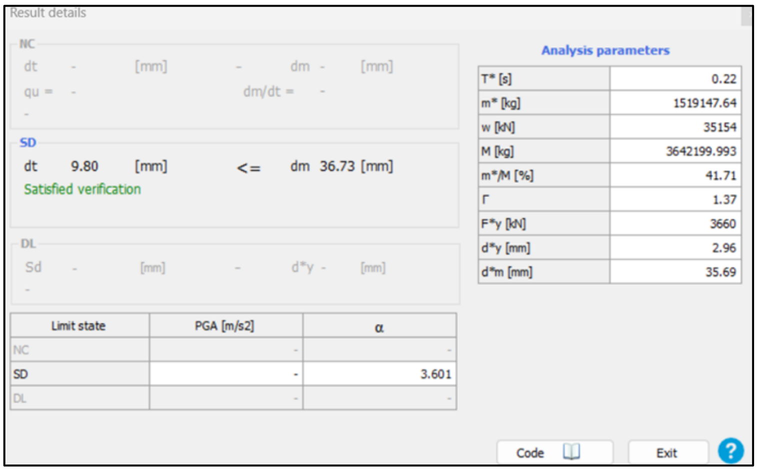


Figure 78 – Graphical damage showing the damage on the structure (Adapted from Tremuri)



Limit state	PGA [m/s2]	α
NC	-	-
SD	-	3.601
DL	-	-

Figure 79 – Table of results EQ T2 step 23 | Static Forces

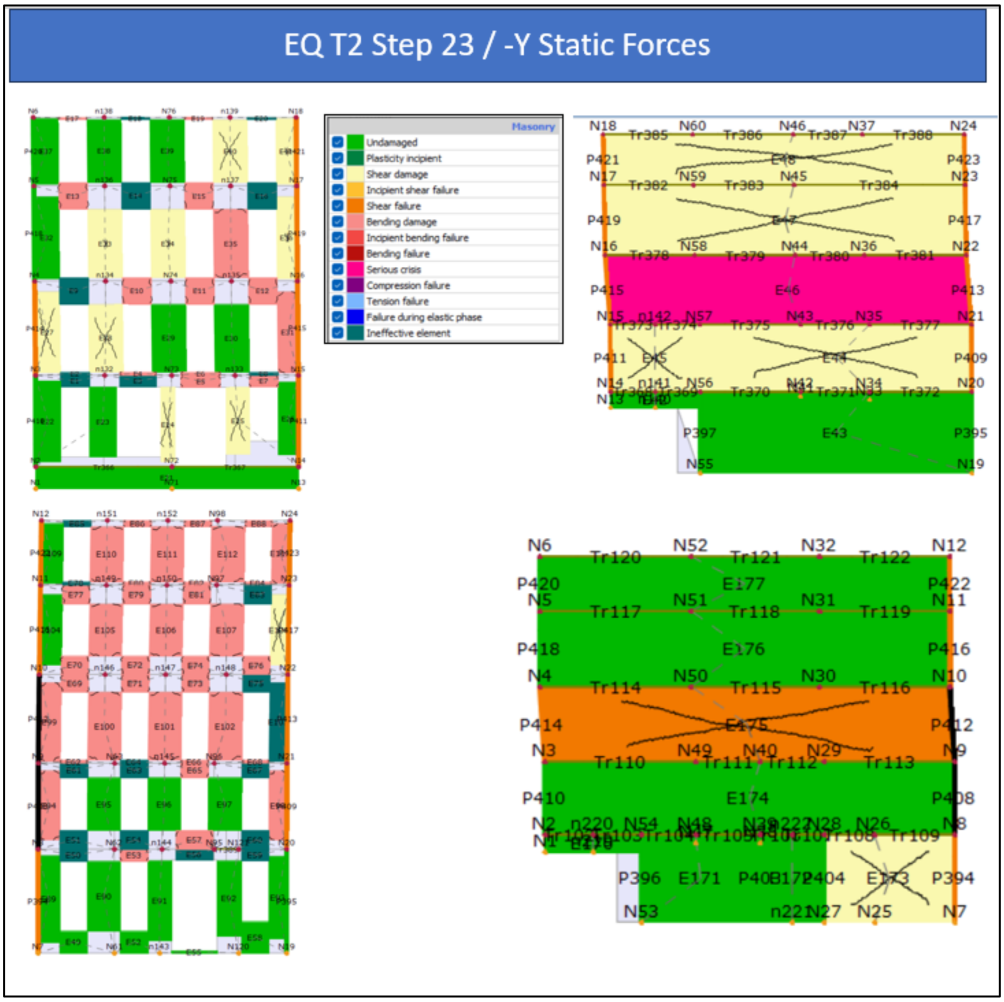
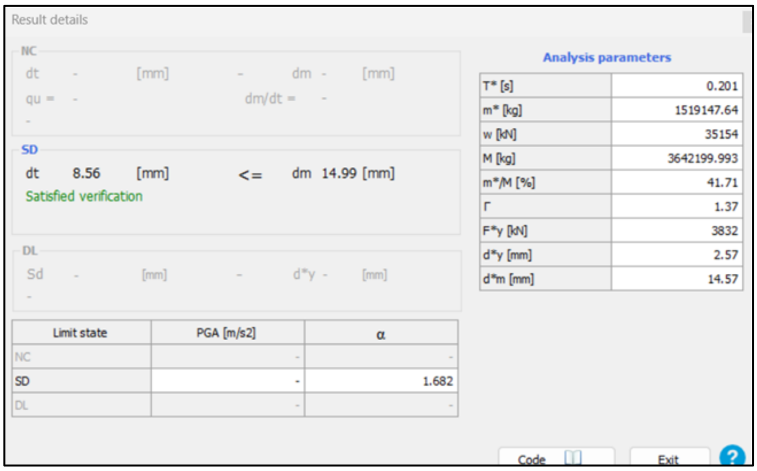


Figure 80 – Graphical damage showing the damage on the structure (Adapted from Tremuri)



Result details

NC
dt - [mm] - dm - [mm]
qu = - dm/dt = -
-

SD
dt 8.56 [mm] <= dm 14.99 [mm]
Satisfied verification

DL
Sd - [mm] - d*y - [mm]
-

Analysis parameters

T* [s]	0.201
m* [kg]	1519147.64
w [kN]	35154
M [kg]	3642199.993
m*/M [%]	41.71
Γ	1.37
F*y [kN]	3832
d*y [mm]	2.57
d*m [mm]	14.57

Limit state	PGA [m/s2]	α
NC	-	-
SD	-	1.682
DL	-	-

Code Exit

Figure 81 – Table of results EQ T2 step 24 | Static Forces

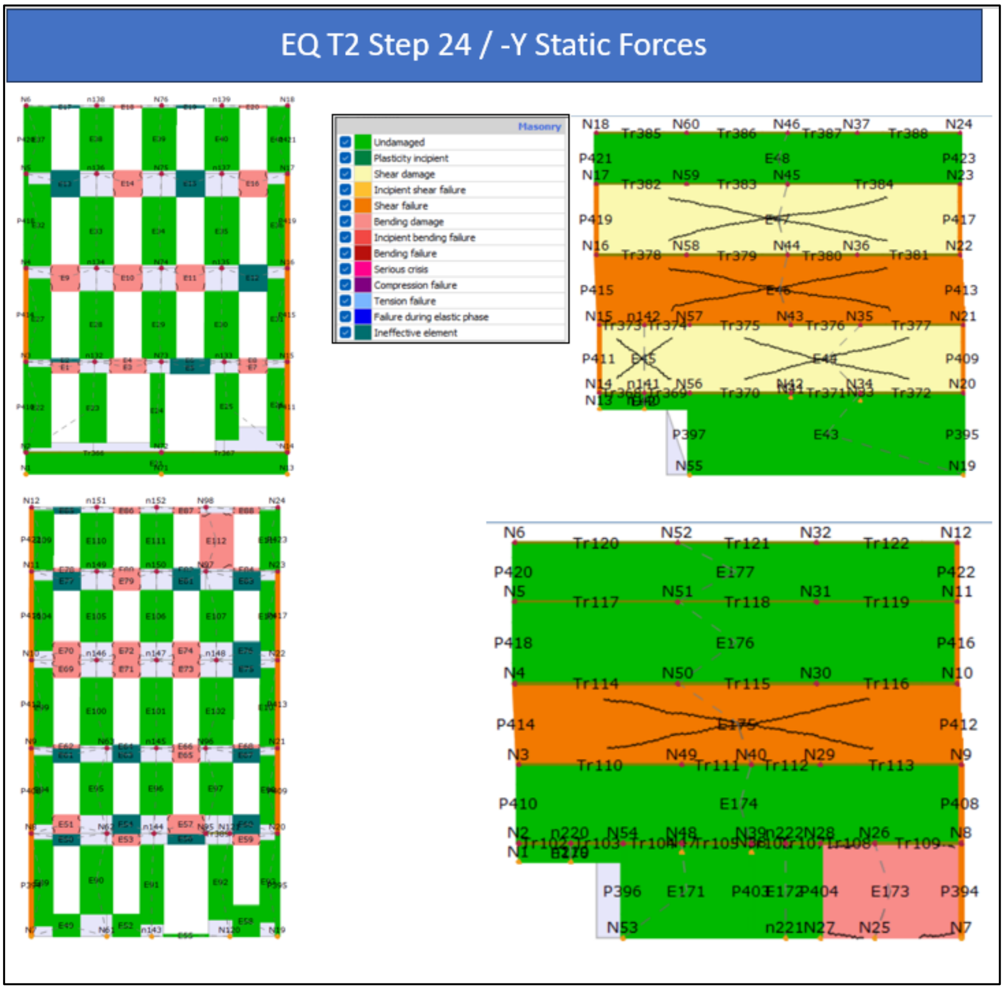


Figure 82 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

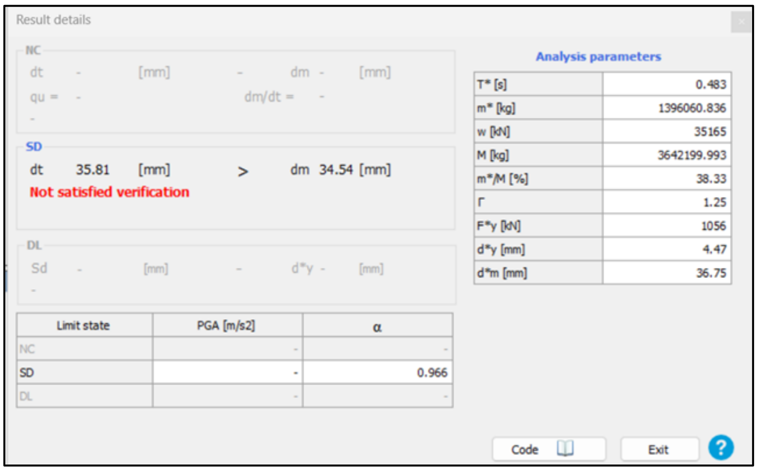


Figure 83 – Table of results EQ T1 step 4 | Static Forces

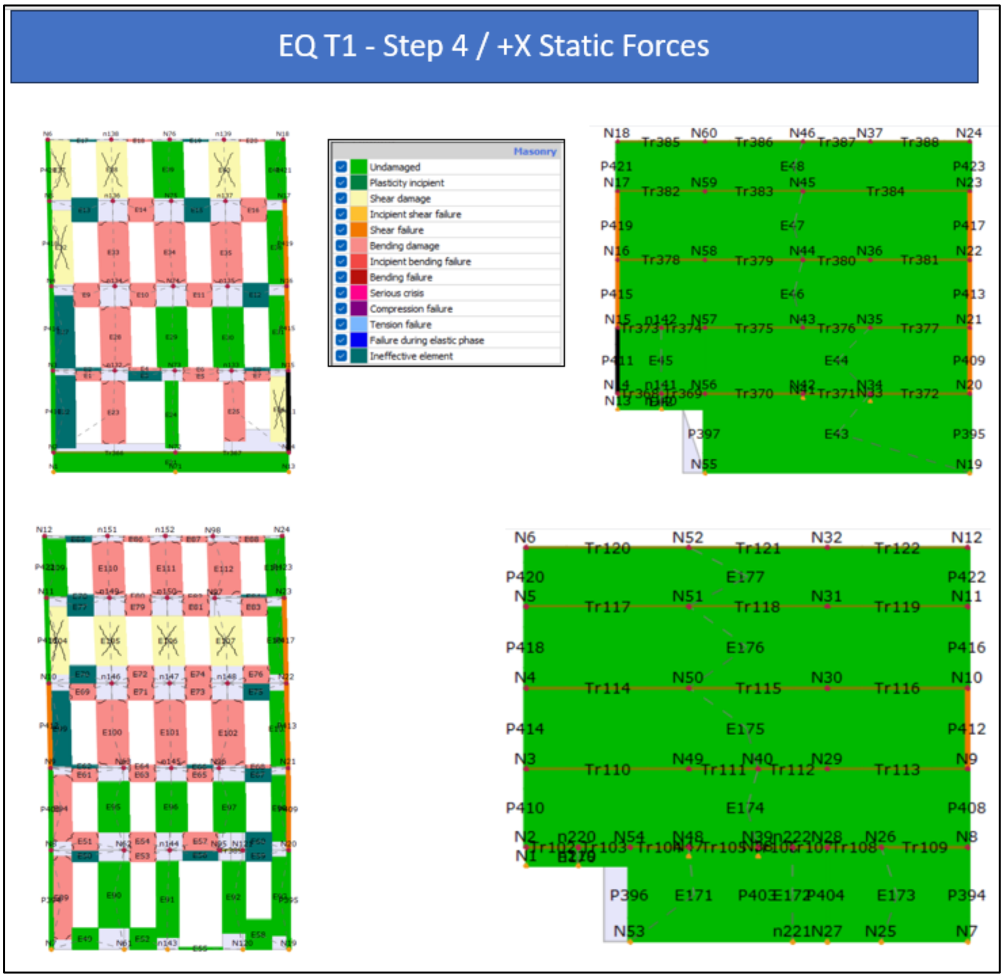


Figure 84 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

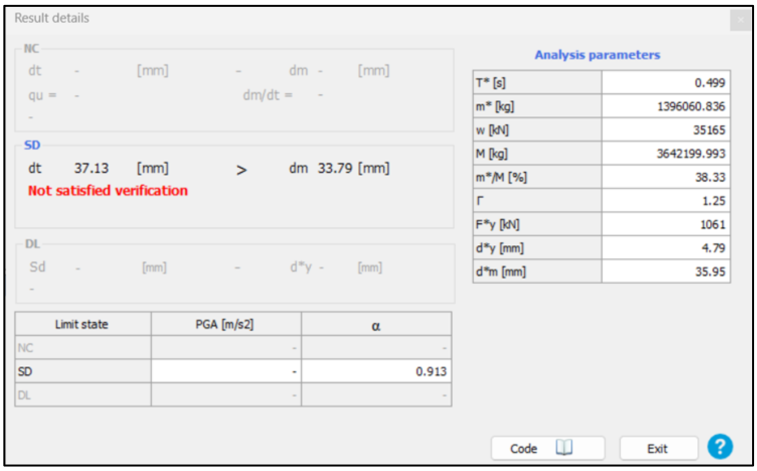


Figure 85 – Table of results EQ T1 step 15 | Static Forces

Figure 86 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

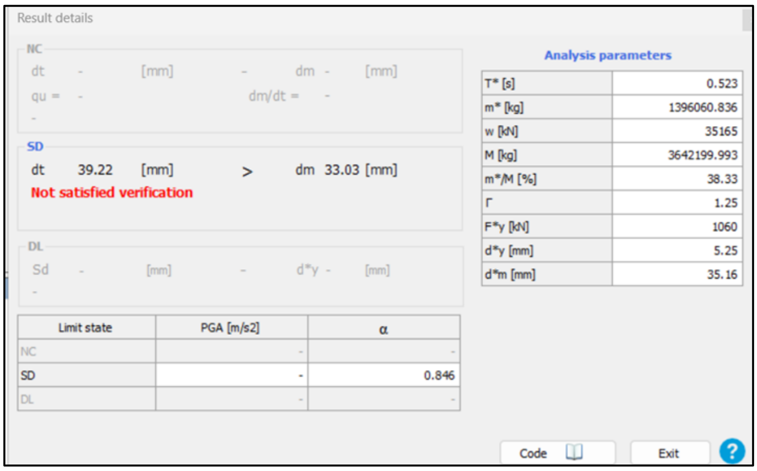


Figure 87 – Table of results EQ T1 step 16 | Static Forces

Figure 88 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

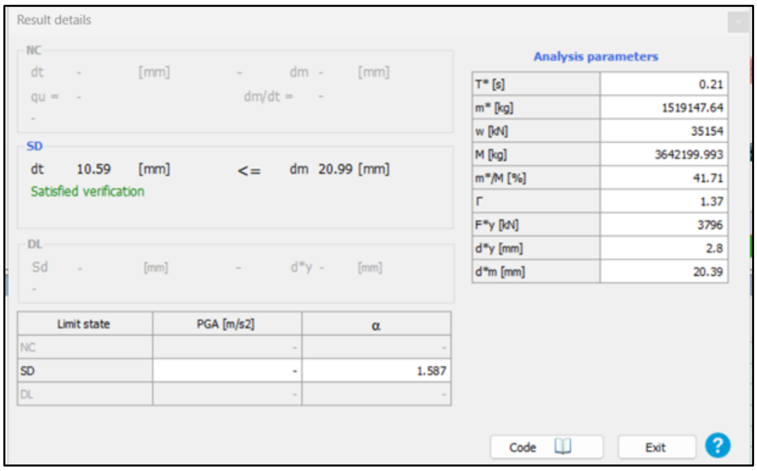


Figure 89 – Table of results EQ T1 step 8 | Static Forces

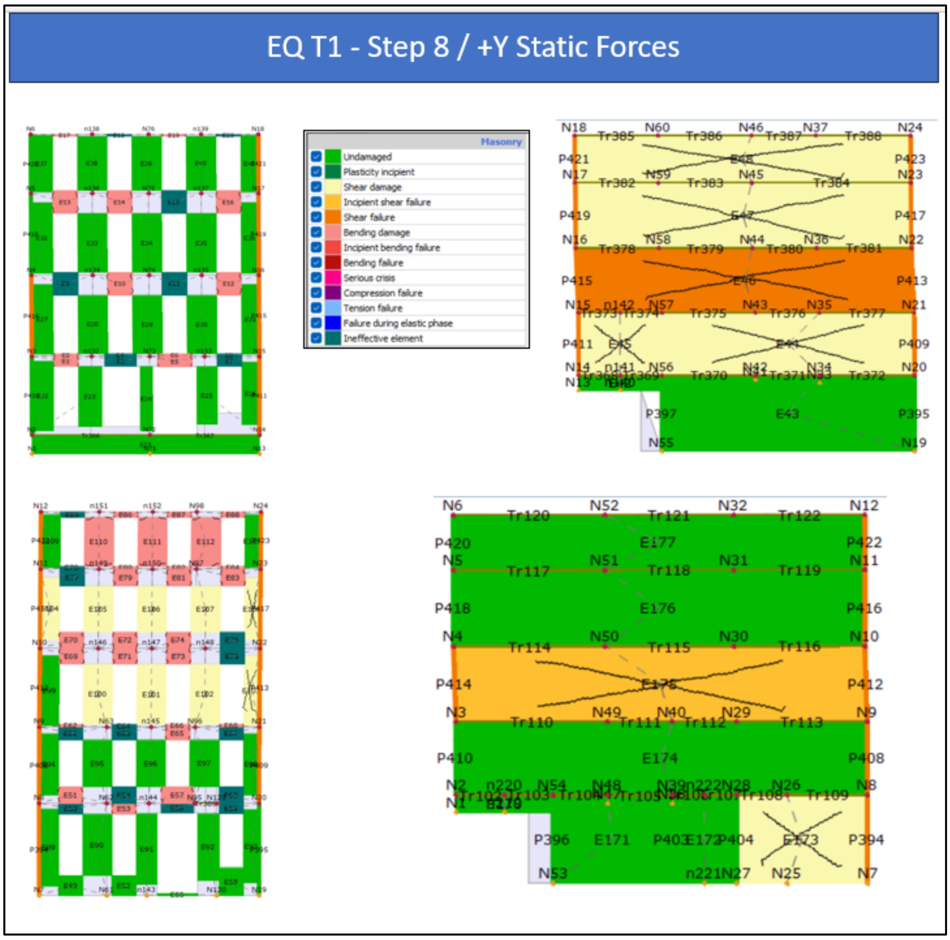


Figure 90 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

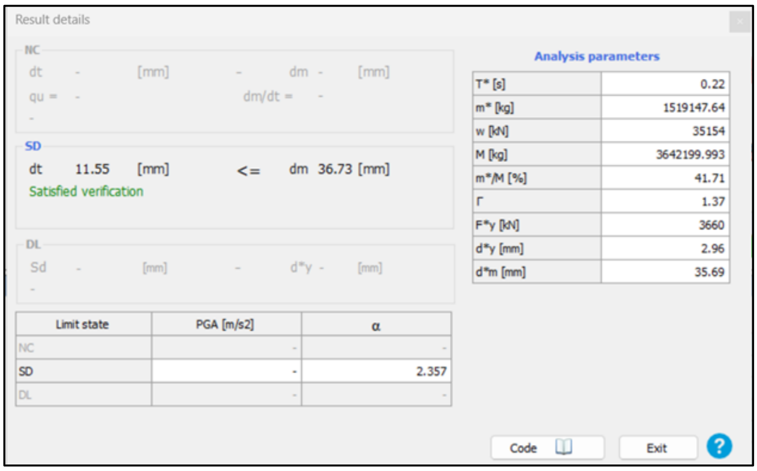


Figure 91 – Table of results EQ T1 step 23 | Static Forces

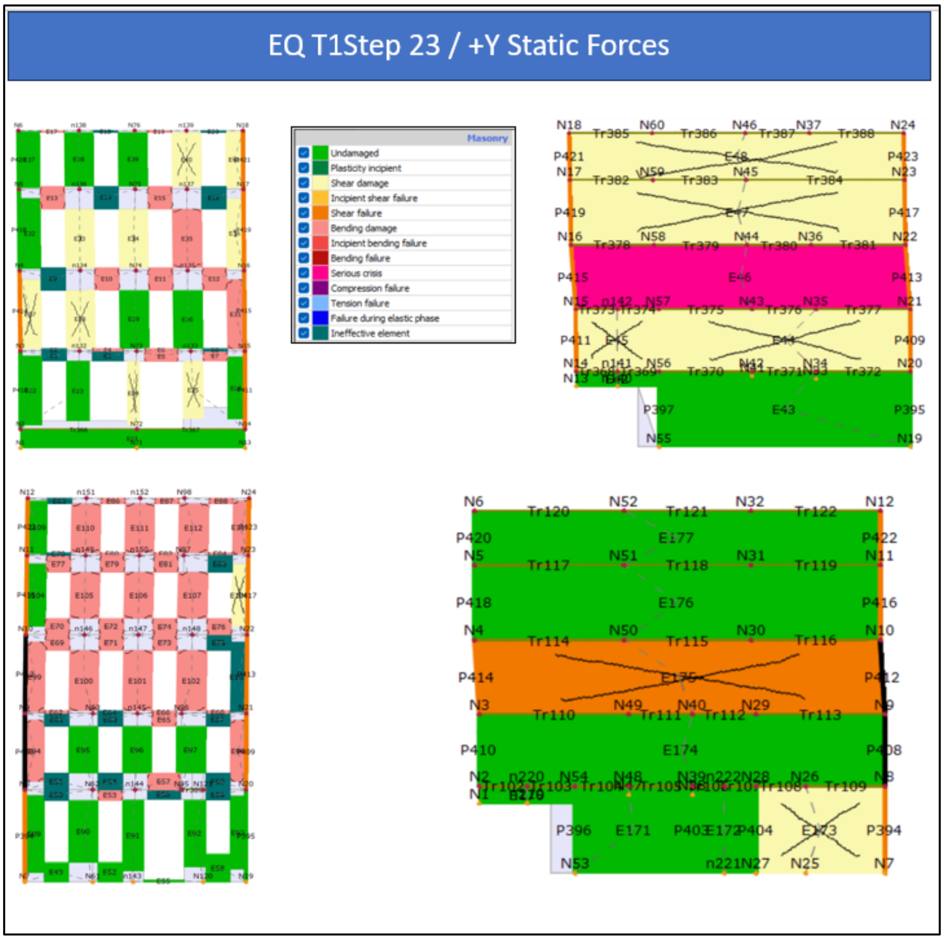


Figure 92 – Graphical damage showing the damage on the structure (Adapted from Tremuri)

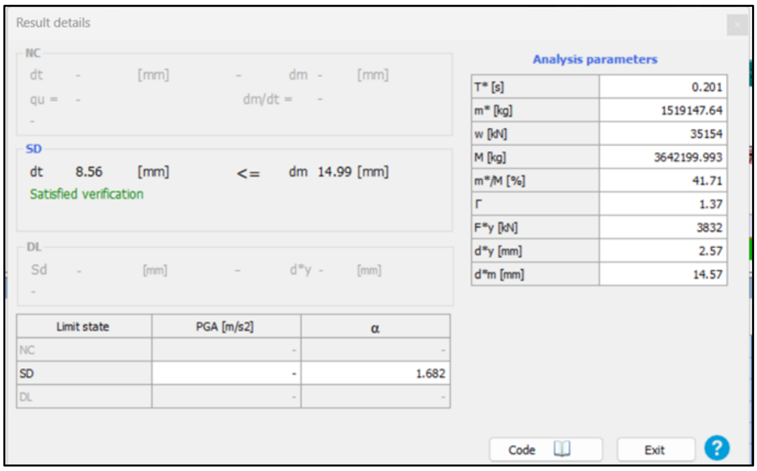


Figure 93 – Table of results EQ T1 step 24 | Static Forces

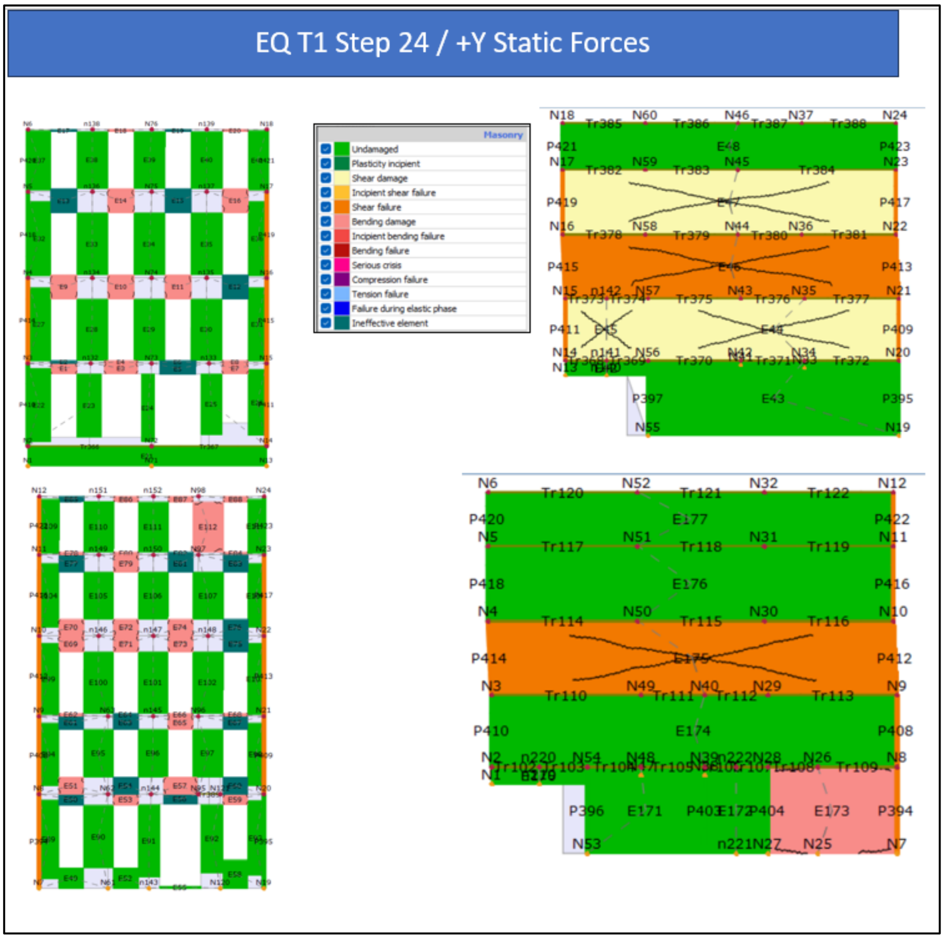


Figure 94 – Graphical damage showing the damage on the structure (Adapted from Tremuri)