Seismic vulnerability of timber framed buildings

Pombalino versus Baraccata

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Dissertation for the degree of Master of Science in

Civil Engineering

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Abstract

The constructive bases on the origin of the timber framed masonry buildings were lost through the centuries and different national or regional variations can be found. Only in the XVIII century, these traditional systems found some sort of codification, in Portugal before and afterwards in Southern Italy, in both cases as part of the massive reconstruction process that followed catastrophic earthquakes that occurred in those countries at that time. This constructive technology, that mixes the mechanical properties of masonry and timber frames, is reasonably described in literature by different authors, but nonetheless the researches about their actual seismic behaviour are still scarce and lack integration. The main goal of the thesis is to investigate and compare their seismic performance. Two case studies, one of the regional variation of the "Pombalino" system that occurs in Vila Real de Santo António (Algarve, Portugal), and other of the Calabrian "Baraccata" system were chosen. After a technical and architectural survey, two structural models were developed in 3Muri software. The seismic vulnerability assessment was conducted according to the N2-method of the Eurocode 8. The work about the Baraccata case study was developed at Università della Calabria. Subsequently the results obtained for both buildings were compared each other and were presented in the complete version of the work identifying the possible retrofitting or strengthening techniques that could overcome inherent flaws and/or the effects of the deterioration that some of these buildings exhibit in the present days.

Keywords: Timber framed masonry buildings, nonlinear static analysis, mixed structures, Pombalino building, Casa baraccata

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Index

Index of the figures

Index of the tables

Index of the acronyms

PPSNP………………………………………….. Plano de Pormenor e Salvaguarda do Núcleo Pombalino
SGU………………………………………………………………………….. Sociedade de Gestão Urbana
VRSA…………………………………………………………………………. Vila Real de Santo António
INGV………………………………………………………….. Istituto Nazionale di Geofisica e Vulcanologia
MCS…………………………………………………………..…………… Scala Mercalli-Cancani-Sieberg
H.Ea.R.T…………………………………………………………………………………………….. Historic Earthquake-Resistant Timber Frames in the Mediterranean Area (International conference)
DIGC……………… Departamento de Informação Geográfica e Cadastro (Càmara Municipal de Lisboa)
CPTI04 ………………………………………Catalogo Parametrico dei Terremoti Italiani (maggio 2004)
PGA ………………………..………………………………….. Peak horizontal Ground Acceleration

Index of the symbols

S_{De} (T) elastic displacement response spectrum

S_e (T) elastic horizontal ground acceleration response spectrum also called "elastic response spectrum". At *T*=0, the spectral acceleration given by this spectrum equals the design ground acceleration on type A ground multiplied by the soil factor *S*;

S soil factor

a_g design ground acceleration on type A ground

η damping correction factor

γ_I importance factor

d displacement

T_C corner period at the upper limit of the constant acceleration region of the elastic spectrum;

F_b base shear force

T_{NCR} reference return period

C_u importance factor

E Average Young's Modulus

G Average Shear's modulus

FC confidence factor

w weight

f_m average compressive strength

τ_0 average shear strength

$\overline{F_i}$ normalized lateral forces

Φ_i normalized displacements

m_i mass in the *i*-th storey

m^* mass of an equivalent SDOF system

Γ Modal transformation factor

F^* force of the equivalent SDOF system

d* displacement of the equivalent SDOF system

T^* period of the idealized equivalent SDOF system

d_{et}^* target displacement for the equivalent SDOF system

F_y^* yield force

d_y^* yield displacement of the idealised SDOF system

d_t^* target displacement of the SDOF system

d_u^* ultimate displacement of the SDOF system

d_t target displacement of the MDOF system

μ ductility factor

R_μ reduction factor

d_u ultimate displacement

d_y yield displacement

1. INTRODUCTION

1.1 INITIAL CONSIDERATIONS

The territories of Southern Italy and Portugal are important scenery to investigate the possible seismic behaviour of the timber framed masonry buildings, a constructive technology, mixing the mechanical properties of masonry and timber frames. It is adopted as an Earthquake Resistant solution in some earthquake prone countries.

The origin of these mixed structures were lost through the centuries and different national or regional variations can be found. Only in the XVIII century, these structures found some sort of codification, firstly in Portugal before and afterwards in Southern Italy, in both cases as part of the massive reconstruction process that followed catastrophic earthquakes that occurred in those countries at that time.

This constructive technology is reasonably described in literature by different authors, but nonetheless the researches about their actual seismic behaviour are still scarce and lack integration. Investigate their seismic resistance to assess their nonlinear seismic response is the main purpose of this thesis. It is also important to identify their critical parts in order to adopt retrofit measures.

The present work shown a comparison between the Portuguese "Pombalino" system, namely the regional variation that occurs in Vila Real de Santo António (Algarve, southern Portugal), and the Calabrian "Baraccata" system. First was conducted an historical and geographical investigation about the development and the main constructive features of these systems.

A Pombalino case study was chosen to develop the research, and after a technical and architectural on site survey, was developed a structural model in 3Muri software. The model was used to derive the capacity curves. To conduct the seismic vulnerability assessment of the building was applied the N2 method, according to the Eurocode 8.

In the last part of the work was investigated the expected seismic performance of the structure and was compared the results obtained, regard the x and y direction of the seismic load considered.

1.2 ORGANIZATION OF THE THESIS

The thesis is divided into four chapters. In the first chapter, of introduction, some initial consideration and objectives of the thesis to explain the develop of the work are reported.

In the second chapter are carefully investigated the historical and geographical background, about the two countries subject of the study. Particular attention was dedicated at the seismicity of the areas and at the development of the constructive techniques after the massive reconstruction, as a result of the catastrophic earthquakes of 1755 in Portugal, and 1783 in the Calabrian area. An important part of the second chapter is constituted by the analysis about the constructive characteristics of the “Pombalino buildings”, in the regional variation that occurs in Vila Real de Santo António, and of the “casa baraccata”, comparing their each other and highlighting the principal similarities and strengths.

The third chapter is focused on the Portugal case study, a two-storeys building that is part of the Pombalino core in Vila Real de Santo António. The chapter presents the geometrical relief, the characterization of the soil and all aspects about the materials of the structure. Secondly, is represented the structural model, developed in 3Muri. The model is important to investigate the seismic behaviour of the building and to derive the capacity curves. Finally is presented the nonlinear static analyses with the N2 method for the existing building.

The last part of the third chapter presents all the conclusions obtained. Lastly, it is reasonable to ask if these traditional systems can still be used today. The final considerations about the comparison of these systems were included in the complete thesis work presented and discussed at Università della Calabria (Italy).

2. HISTORICAL AND GEOGRAPHICAL BACKGROUND

Il terremoto è "un genere di male contro cui si è autorizzati a servirsi della precauzione"

E. Kant (riferendosi al catastrofico terremoto di Lisbona del 1755)

The seismic-hazard map is the foundation of the building code. It proves that, being based the distribution of the earthquake events in the Past, is a valid tool to mitigate seismic risk. The seismic hazard map provide a good deal of information to understand the earthquake risk in many countries. This and the evaluation of the vulnerability of the entire built heritage, usually conducted by considering human lives and buildings exposed to this risk are valid instruments to reduce the consequences of these natural phenomena.

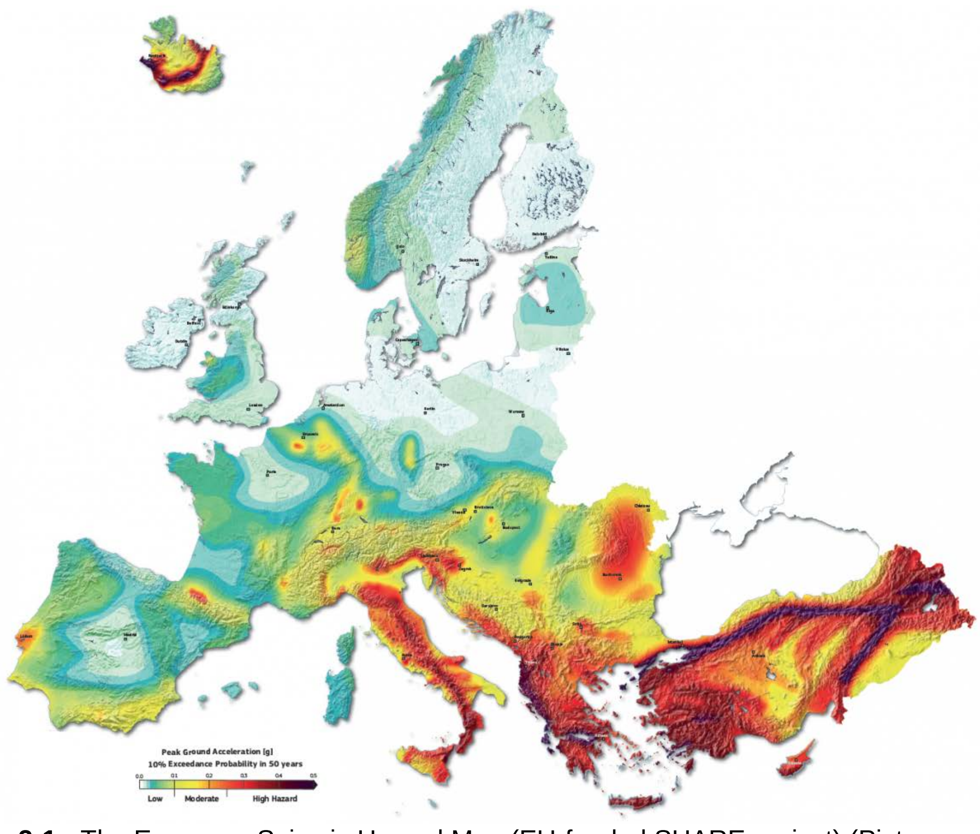


Figure 2-1 - The European Seismic Hazard Map (EU-funded SHARE project) (Picture retrieved from http://www.share-eu.org/)

2.1 EUROPEAN SEISMICITY

Europe has a long history of destructive earthquakes, for this way it is very important to learn from the past. The European Seismic Hazard Map (EU-funded SHARE project), in Figure 2-1, shows the Europe zones' which are most at risk from an earthquake, where low hazard areas (PGA ≤ 0.1 g) are in blue colours, moderate hazard areas are in yellow to orange colours, and high hazard areas (PGA > 0.25g) are in red colours. The PGA value represents the peak horizontal ground acceleration to be reached or exceeded with a 10% probability in 50 years, corresponding to the average recurrence of such ground motions every 475 years. As can be seen in the map, the Balkan and Mediterranean countries (Italy, Greece, Bulgaria, Romania), as well as Turkey, are all at higher risk of earthquakes than many other parts of Europe.

There are also earthquake hotspots, near Brussels (Belgium), in Lisbon (Portugal) and near Budapest (Hungary), because a number of earthquakes has happened in the past, and along the Pyrenees mountain range.

Portugal and Italy have two different historical-geographical settings, construction technologies and typologies, and a different urban planning, but have a common danger, the earthquake.

2.1.1 CALABRIA SEISMIC HAZARD

Calabria, in Southern Italy, is one of the most seismic area in the Mediterranean; According to S. Porfido et al., "the MessinaStrait is strictly connected with the Siculo-Calabrian rift zone, one of the most seismically active areas of the Italian peninsula". Its economic development is still hindered by also frequent earthquakes.

Calabria Region has been struck in the past by the most catastrophic earthquakes ever occurred in Italy. "The Siculo-Calabrian seismic belt includes the largest earthquakes which have occurred in southern Italy in the last six centuries as the 1693 earthquakes, the 1783 seismic sequence, the 1905 Calabria earthquake and the 1908 Messina - Reggio Calabria earthquake." According to Porfido et al., "These

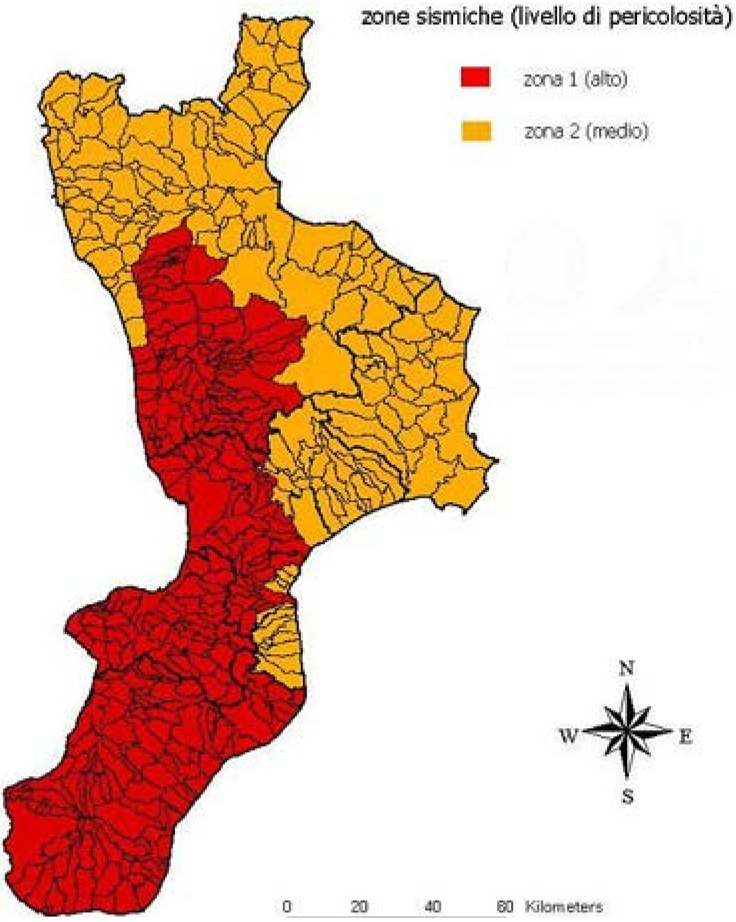


Figure 2-2 – Hazard seismic map of Calabria Region (INGV) (Picture retrieved from http://www.protezionecivilecalabria.it/index.php/i-rischi-in-calabria/il-rischio-sismico) earthquakes produced wide damage on the urban design and triggered spectacular coseismic environmental effects along the coastal area".

Almost all the Calabrian recorded events are concentrated in less than three centuries, between 1638 and 1908, and they often occurred within a few months or years of one another (e.g., March and June 1638, February–March 1783, and 1905–1908) or within one century (southward migrating sequence of Crati Valley of 1767, 1835, 1854, and 1870). Differently from southern Calabria, where all major earthquakes have been related to primary NE-SW normal faults, the northern Calabria seismogenetic framework is still poorly constrained and debated (see Tortorici et al., 1995 [1]; Jacques et al., 2001 [2]; Galli and Bosi, 2002 [3]).

The seismicity that occurred in this area in the last 2000 years, is well documented in the Table 2-1. The Table 2-1 "shows a very high-recurrence of large events with at least 34 earthquakes with VII ≤I≤XI on the MCS scale; nine events with I = X-XI in the last 2000 years, five of which in the last 225 years and two in XX century", according to S. Porfido et al. [4] description.

Some of several factors, which characterize the critical situation of the Italian territory for the seismic risk, are the obsolescence of many buildings, late seismic classification of the territory, the high seismic vulnerability of the historical centres and of the huge Italian cultural heritage.

Year	Month	Day	Epicentral area	Io	Maw
-91			Southern Calabria	9.5	6.3
374			Southern Calabria	9.5	6.3
853	8	31	Messina	9.5	6.3
1169	2	4	Eastern Sicily	10	6.6
1184	5	24	Crati Valley	9	6
1509	2	25	Southern Calabria	8	5.57
1626	4	4	Girifalco	9	6.08
1638	3	27	Calabria	11	7
1638	6	8	Crotonese	9.5	6.6
1659	11	5	Central Calabria	10	6.5
1693	1	11	Eastern Sicily	11	7.41
1743	2	20	Southern Ionio	9.5	6.9
1767	7	14	Cosentino	8.5	5.83
1783	2	5	Calabria	11	6.91
1783	2	6	Southern Calabria	8.5	5.94
1783	2	7	Calabria	10.5	6.59
1783	3	1	Central Calabria	9	5.92
1783	3	28	Calabria	10	6.94
1786	3	10	North-East. Sicily	9	6.02
1791	10	13	Central Calabria	9	5.92
1818	2	20	Catanese	9	6
1823	3	5	Northern Sicily	8.5	5.87
1832	3	8	Crotonese	9.5	6.48
1835	10	12	Cosentino	9	5.91
1836	4	25	Northern Calabria	9	6.16
1854	2	12	Cosentino	9.5	6.15
1870	10	4	Cosentino	9.5	6.16
1894	11	16	Southern Calabria	8.5	6.05
1905	9	8	Calabria	11	7.06
1907	10	23	Southern Calabria	8.5	5.93
1908	12	28	Southern Calabria	11	7.24
1909	7	1	Calabro Messinese	8	5.55
1947	5	11	Central Calabria	8	5.71
1978	3	11	Southern Calabria	8	5.36
1978	4	15	Patti Gulf	9	6.06

Table 2-1- List of the largest earthquakes occurred in Southern Italy in the last 2000 years (I ≥ VIII MCS); (extracted from S. Porfido et al. [4])

2.1.2 THE 1783'S CALABRIAN EARTHQUAKE

The telluric event was characterized by a seismic sequence from the beginning of February to the end of March. There have been five peaks of maximum intensity happening on February 5th (M 6.9), 6th (M 6.2), 7th (M 6.4) and March 1st (M 5.6) and 28th (M 7.0) and several hundred lesser tremors, lower in force. The cumulative effect of this seismic sequence have been catastrophic, destroying thousands of square kilometres of territory. How is described by S. Porfido et al. [4] “More than 30,000 lives were lost and 200localities were completely destroyed by the February 5 main shock; the epicentral area (I0= XI MCS) was located on the Gioia Tauro plain, at the western foot of the northern Aspromonte mountain.” The descriptions by Porfido et al. of this catastrophic event were reported below: “The shock produced spectacular ground effects, both primary and secondary, such as tectonic deformations, ground fractures, liquefactions phenomena, tsunami, hydrological changes and diffuse landslides of large size, which in most cases dammed the rivers creating more than 200 new temporary lakes”. (see S. Porfido et al. [4])

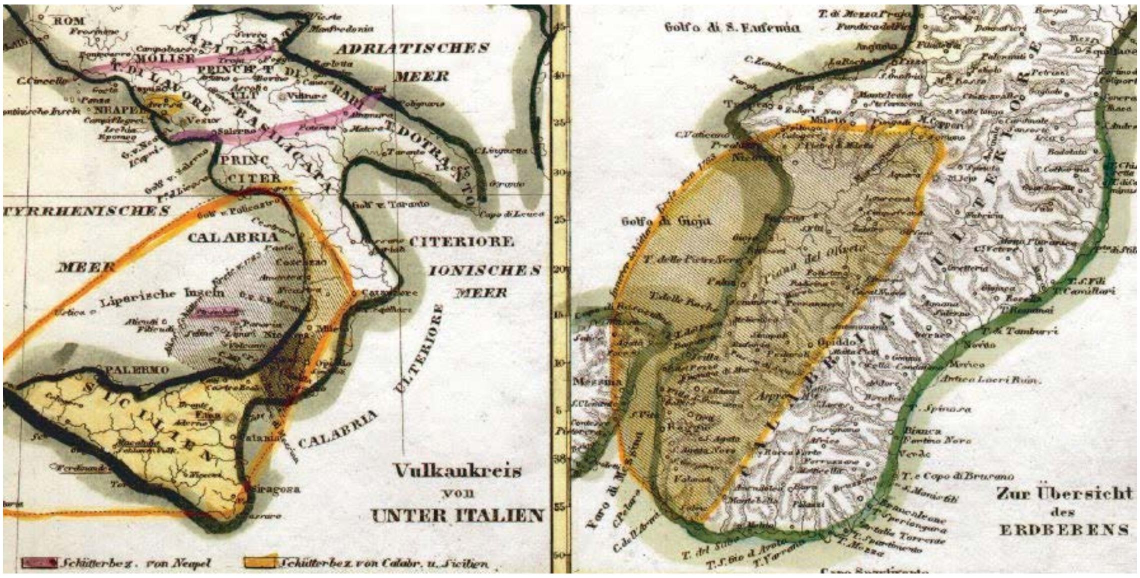
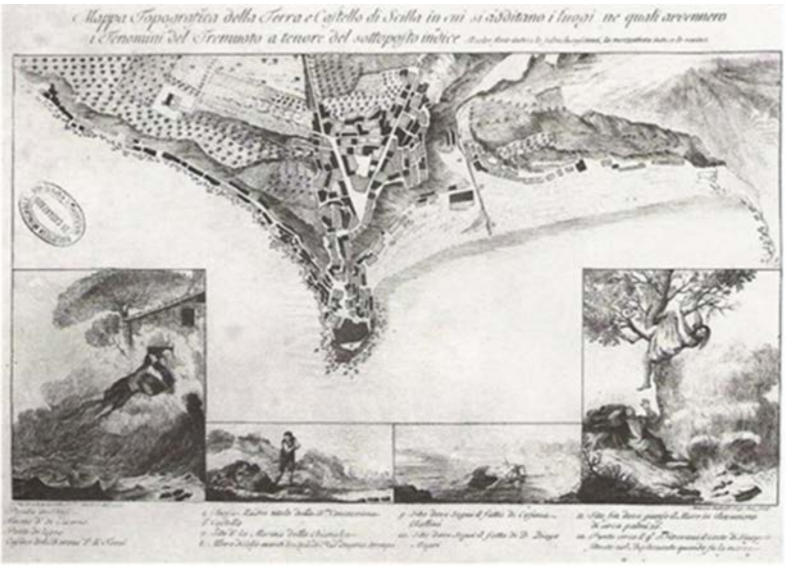


Figure 2-3 - An early map of the 1783 Calabria volcano and earthquake-areas plotted in the mid-19th century (Picture extracted from BERGHAUS [9], 1845-1848)



Mappa topografica della terra e castello di Scilla in cui si additano i luoghi ne quali avvennero i fenomeni del tremuoto... (Schiantarelli, 1784)

Figure 2-4 – Historical-drawing illustrating the tsunami triggered by the February 6, 1783 event along the Scilla coast (Schiantarelli, 1784) (Picture extracted from S. Porfido et al. [4])

“The second shock, occurred on February 6, struck the coast between Scilla and Palmi and induced the large Monte Pacì-Campalla rock avalanche from the sea-cliff west of Scilla, generated a disastrous tsunami, run-up of 16 m (Figure 2-4). This event has affected the coast for a total length of 40 km, from Bagnara to Villa San Giovanni and from Torre Faro to Messina, causing more than 1500 casualties in Scilla” (see S. Porfido et al. [4])

“From February 7 to March 28, three main shocks took place with epicentres migrating northwards from Mesima Valley to Catanzaro” (see S. Porfido et al. [4]).

“The last one caused severe damage along both the Tyrrhenian and Ionian coasts. The cumulative effects of all these earthquakes was devastating, more than 380 villages were damaged, 180 of these were totally destroyed” (see S. Porfido et al. [4]).

The rough epicentral locations of the five main shocks are reported in Table 2-2, together with the Io (MCS epicentral intensity), the estimated magnitude Me (the equivalent magnitude evaluated based on the real distribution of the intensities) and Ms (the magnitude evaluated using Io, [$Ms = 0{,}56 * Io + 0{,}94$]):

Month	Day	Latitude deg	Longitude, deg	Io(MCS)	*Me*	*Ms*
Feb.	5	38.30	15.97	11.0	6.88	7.1
Feb.	6	38.22	15.63	8.5	6.34	5.8
Feb.	7	38.58	16.20	10.5	6.56	6.8
March	1	38.77	16.30	9.0	5.85	6.0
March	28	38.78	16.47	11.0	6.98	6.6

Table 2-2 – List of the Five Most Severe Events of the 1783 Earthquake Sequence, Working Group CPTI (1999) (P. Galli, V. Bosi et al. [4])

The impact of the earthquake has been devastating, marking an epoch in the region’s economic, social and cultural life, shaking an already precarious social order.

For the first time, were mapped and drawn scientific documentation, describing the spectacular impact on the natural environment, such as landslides and slips, fissures and cleavages, newly formed lakes and liquefaction phenomena [Schiantarelli et al.[1]].

As clearly described by M. Folin et al. [8], several new lakes were created, due to the blocked waterways by the landslides. Because of one of these, that was crashed into the Tyrrhenian Sea, near Scilla, on the night between the 5 and the 6 February 1783, waves, 6-8 metres high, have swept inland more than 160 metres and have brought another 1.300 deaths, terror and havoc in the Calabrian Coast.

The extent of the area destroyed from the earthquake prompted the central Neapolitan government to intervene. In fact, were involved not just small villages, but also important economic and military urban centres of the Kingdom of Naples and Sicily, like Reggio Calabria, Catanzaro and Messina.

Despite the rescue operations and subsequent reconstruction projects enacted by the Bourbon government, the “Grande flagella” (great disaster as termed by the 18th century authors) has prostrated and conditioned the economy and development of the region, perhaps until the 20th century.

In this context, certainly influenced by the experience of Portugal, and in particular of the reforms used in Lisbon to rebuild in the wake of 1755 earthquake, was developed the Bourbon constructive system, the first anti-seismic code in Europe. This code, which included many important design criteria, still present in modern codes, among the other rules, prescribed the buildings' reconstruction with the reinforcement of a skeleton of timber elements.

[1] M. Sarconi. Istoria de’ fenomeni del tremuoto avvenuto nelle Calabrie e nel Valdemone nell’anno 1783. 1784, Reference from S. Porfido et al. [4]

2.1.3 PORTUGAL SEISMICITY

According to what has been written in FCT research[2] "Concerning the number of earthquakes registered in Portugal, most of the occurrences happened on the XVIII and XIX centuries. However, this great difference in numbers may be related with a greater availability of register sources, which, before this period, was scarce." In FCT research, it is also reported: "There were some violent earthquakes BC, but they were not properly documented. In the fourth century AD, violent earthquakes occurred, with magnitudes probably around 9.0 MS. These were followed by the eighteenth century earthquakes, in which the 1755 earthquake was recorded with a high of 8.0 MS."

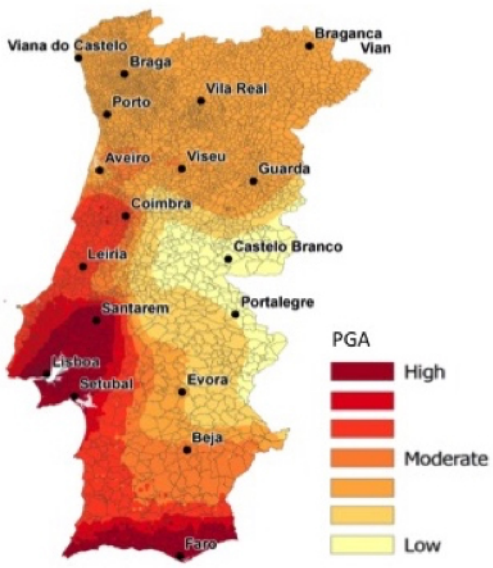


Figure 2-5 - Seismic hazard map for 475 years for Portugal. (Picture retrieved from [https://gemrisk.wordpress.com/2016/02/11/bridging-the-gap-between-physical-and-social-earthquake-risk/)](https://gemrisk.wordpress.com/2016/02/11/bridging-the-gap-between-physical-and-social-earthquake-risk/)

The interest regarding this type of phenomena started after the occurrence of this violent earthquake. In fact, for the increasing interest and the fear, we have today, the documentation also of the smaller tremors, which would probably otherwise, do not be registered. In the history of Portugal, thanks of the quantity and quality of the available documentation, "the1755's Lisbon earthquake, the 1909's Benavente earthquake, the 1969's Algarve earthquake and the 1980's Azores earthquake, despite their different circumstances, are considered the most relevant occurrences" as it is well observed in FCT research[2].

The political authorities, after these events, in order to regulate the reconstruction process, often used a set of rules and a plan. The Pombal reconstruction was the unique effectively and fully implemented plan. It cannot tell the same of the other plans respectively after the Benavente earthquake (1909), and the 1969 Algarve earthquake. In these cases, in fact, without substantial innovation, the same construction techniques practiced before the quake were reproduced for the reconstruction phase. The delay of official aid was one of the cause and then, as often happened the not consciously choice to leave everything to the private initiative.

However, the builders, neglecting the importance to reinforce the buildings, did not prevent damages due to the future quakes, rising the risk of the population. It happened also that after some time from the earthquake's occurrence, the rules, established in the reconstruction plans, were no longer followed and the successful anti-seismic techniques were lost in the century. Due to the devastating earthquakes that occurred in the past, Lisbon can be considered one of Portugal's high-risk seismic areas. According with

[2] FCT (Fundação para a Ciência e a Tecnologia) research [12] retrieved from https://esg.pt/seismic-v/portuguese-historical-seismicity/

the AIR Source[3] [G. Franco, B. Shen-Tu], in the last few decades, reinforced concrete is the predominant type of construction in Portugal, accounting for roughly 50% of the current building inventory, as is typical of many European countries.

Portugal's masonry constructions, present in historic districts and city centres, accounts for some 36% of the building stock, displaying a wide range of building techniques and construction quality. There has been a variety of typologies of masonry buildings in Lisbon, ranging from the traditional rubble-stone double-wythe two or three storeys building to the more sophisticated "Pombalino" structures, characterized by a braced timber frame with masonry infill.

The AIR Source reports also that one study[4] found that the poor maintenance of aged masonry structures in Portugal—some over 200 years old—may have a significant impact on their vulnerability. Mortar decays over time, and failure to monitor and maintain this bonding layer in masonry walls can increase the building's vulnerability. Therefore, the masonry buildings in Lisbon's historic districts might therefore be somewhat more vulnerable than buildings of a similar age in other parts of Europe.

In the conclusion of the AIR Source is reported that, if a similar event were occurred today, modern tsunami warning systems and disaster response practices, as well as superior building construction, would moderate the scale of damage and casualty. Even though seismologists believe that, the recurrence potential is relatively low. The more critical source of concern is the Lower Tagus Valley region, in proximity to Lisbon, which could produce a magnitude 6 to 7 earthquake with a return period as short as 150 to 200 years. In this seismic zone, the presence numerous old masonry buildings and a fraction of reinforced concrete frames, designed with limited lateral resistance, represents the most significant potential for large loss earthquakes in Portugal.

2.1.4 THE 1755'S LISBON EARTHQUAKE

The main telluric event, that has interested Portugal, was so destructive that then became a chronological landmark, dated, 1st November 1755.

The lower part of Lisbon was reduced to rubble. It was an unprecedented impact on Portuguese territory, which followed also the process that was characterized the Lisbon's downtown reconstruction.

A violent impact to the city were generated by the action of the quakes, with fires' propagate and the tsunami that reached the city.

It is shown, in the Figure 2-6, the limits of the fire in dashed lines and the most affected areas struck by the earthquake in orange. After the event occurred, it has been necessary to rebuild the heart of the city. Many was the areas struck brought about by the earthquake. The work was led by the Marquis of Pombal, the Prime Minister of the time, under the orders of the king Dom José I, and carried out by the

[3] The AIR Sources include a study by Rossetto and Elnashai (2003) that derived fragility curves for reinforced concrete structures in Europe based on damage reconnaissance data from 19 earthquakes, as well as research by Speranza et al. (2006) and Di Pasquale and Goretti (2001) that examined the behavior of reinforced concrete and

[4] D'Ayala et al. (1997)

more competent architects and engineers of the time. The proposals by Manuel da Maia, the Kingdom's official engineer for the reconstruction of the city comprised three major phases.

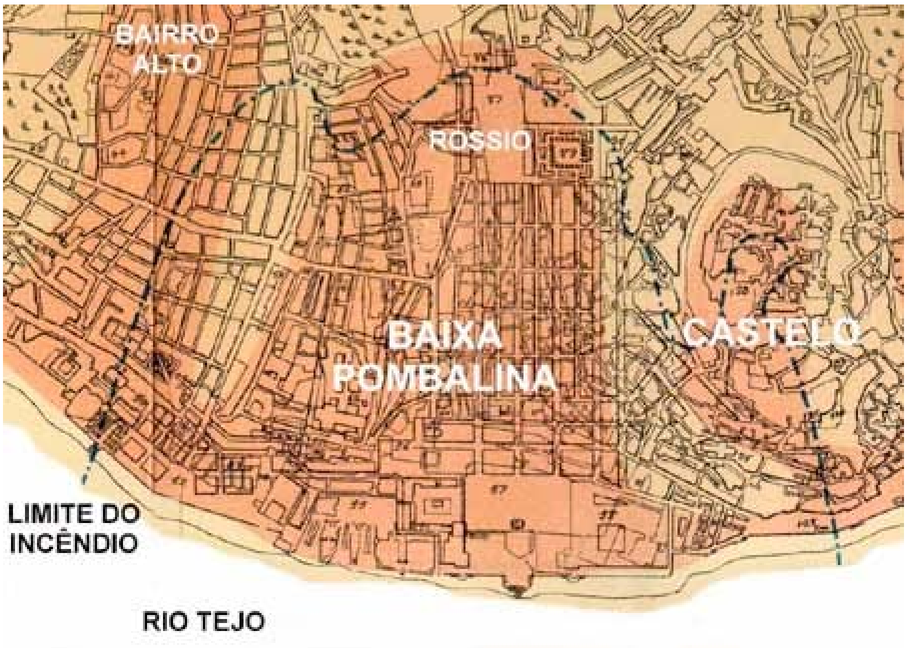


Figure 2-6 - Limits of the fire that struck the capital after the earthquake in dashed line and the most affected areas of the earthquake of 1755 in orange

The first phase, dated 4 December 1755, consisted of five possible approaches, well described by Mascarenhas [15]. After considering the proposals, the king decided to build the new royal palace at Belém, but as far as the city was concerned, he decided for the fourth approach, thus to demolish any remnants of the previous buildings and to draw, for the city centre, a completely new plan (Mascarenhas, 1996 [15]). For the second phase of the proposals, dated 16 February 1756, Manuel da Maia recommended, among his possible approches, to completely demolish Lisbon to the ground and rebuild it following a rational plan and this approach was the one selected (Mascarenhas, 1996 [15]).

Lastly, for the third last phase, dated 31 March 1756, three teams were constituted to prepare six drafts of plans for the reconstruction of the city. All the plans presented tried to preserve Rossio and Terreiro do Paço. The winning plan was the fifth, prepared by architect Eugénio dos Santos de Carvalho, known as plan number 5. This reconstruction plan, as can be seen in Figure 2-7, has an orthogonal layout, with the repetition of rectangular blocks. It shows eight streets perfectly parallel and rectilinear, oriented in the north-south direction and nine streets cross orthogonally in an east-west direction. The plan was also particularly successful in integrating two separate areas with different functions: Rossio, with its daily social functions and Praça do Comércio (Commerce Square) with its commerce and administrative functions (Mascarenhas, 1996 [15]).

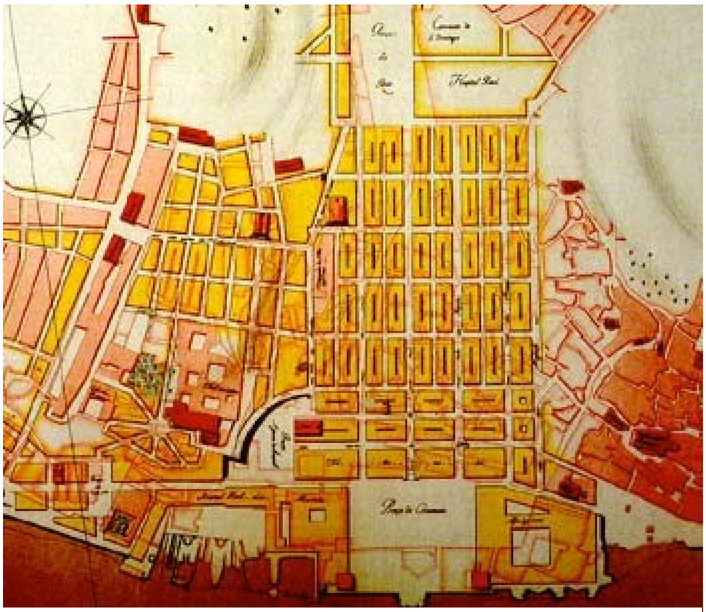


Figure 2-7 - The winning plan of the reconstruction

The rebuilding of the Baixa, so is called the downtown area, initiated in 1756, lasted many decades (about 150 years). The main choice for the reconstruction, due above all to the concern of avoiding future tragedies, was to provide more seismic resistance to the new buildings. For this reason, the rubble due of the demolition process, after the earthquake, was used to create a platform, raising the elevation above the river and extending the shoreline. The new urban planning action required also the use of a building 'model', with minor variations for its implementation. This 'building model' consisted in the use of wooden piling for the foundation system, of the vaulted ceilings on the ground floor, of the 'Gaiola' (cage) system, in all interior facings, and other rules to improve the construction's seismic behaviour.

2.2 CONSTRUCTIVE CHARACTERISTICS

Hazard maps, seismic code and seismic classification are relevant to the improvement of tools for seismic design but learning from traditional construction systems, knowledge of the territory and of the constructions' characteristics of the ancient building might be very important to mitigate the seismic risk.

2.2.1 BARACCATA SYSTEM - THE ANTI-SEISMIC PROTOTYPE

Following the 1783 Calabrian earthquake, under the Borbone Government, were promulgated several measures, to organize the reconstruction. The "*Istruzioni Reali*" [1784], emanated by the Bourbon Government, described a building, of neoclassical shapes, at two floors, that rests on a masonry foundation plate, formed by stacks of oaks well laid in the ground, some of which bigger rise up to the top. Numerous engineers from Naples were sent in the areas struck by earthquake. An important testimony of buildings' damages and victims' number can be seen in *"Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina"*, published in 1783 by Giovanni Vivenzio.

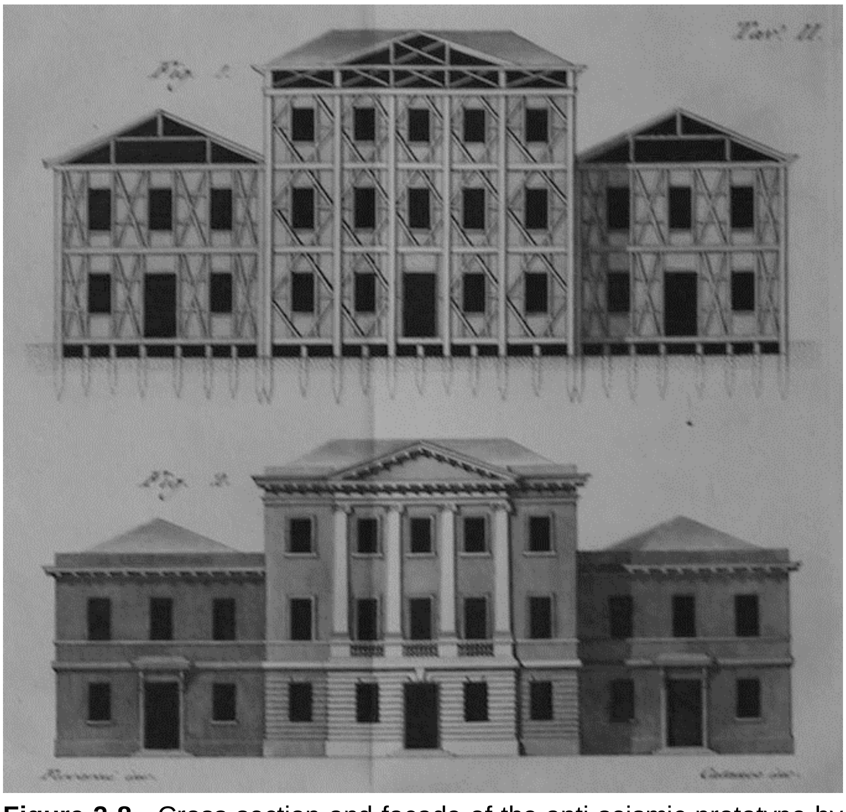


Figure 2-8 - Cross section and facade of the anti-seismic prototype by V. Ferraresi in "La casa tipo. Proposte tecniche", 1783. Tratto da "Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina", G. Vivenzio, 1783.

He was the doctor of the Royal Household, university teacher, erudite in science, in particularly seismology and volcanology. Along with the treatise, there are the Tables (Figure 2-8 and 2-9), with '*Spiegazione*' (explanations). At the end of this book, in the part named 'Case formate di legno', there is an explanation of an earthquake resistant prototype constituted by masonry reinforced with a double timber framing.

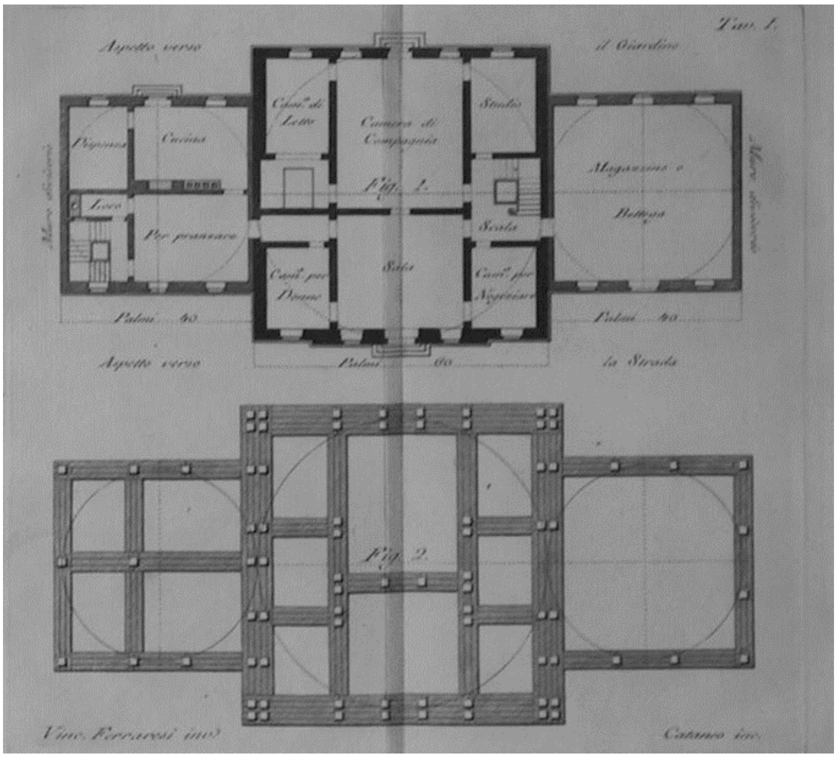


Figure 2-9 - Table 1, drawings of the anti-seismic prototype by V. Ferraresi that shows the ground floor plant and the foundation plate plant, in "La casa tipo. Proposte tecniche", 1783. Tratto da "*Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina",* G. Vivenzio, *1783*

These buildings, constituted by three storeys and two laterals constructions both characterized by two floors, are described through the ground floor plan, the foundation, the building elevation and some constructive details of the corner posts and of the rafter to tie joint of the king-post roof truss. The Vivenzio's "*Case formate di legno*" represents a constructive technique characterized by two wooden frames, a high amount of timber elements and an extreme carpenter difficulty in connections execution, prescribed for public buildings, given the difficulties of construction and the high cost. These features inevitably involve a spread of the use of the "reduced Bourbon system", so appointed by Baratta[5], with a single wooden frame, where usually the ground floor constituted entirely by masonry wall and the only second floor characterized by a timber framing load bearing. Mori[6], to whom reconstruction had been entrusted, in a report[7] sent in 1789 to the General Pignatelli, describes the construction method established by the Court, calling it "Sistema governativo", unlike that used in Reggio Calabria, where was used less timber elements.

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[5] Baratta provides a detailed description of the damage suffered by the buildings due to the earthquake of 1908, reporting the constructive features of the analysed buildings and highlighting the reduced Bourbon system. Highlights the reduced Bourbon system, a structural typology that is distinguished from the rules of the Bourbon Regulation by a masonry base, which extends for a whole level, reinforcing only the first floor by using wooden frames, with the awareness of decreasing the seismic mass with the height of the building.

[6] Engineer responsible for the reconstruction of Reggio Calabria after the 1908's earthquake.

[7] G. Mauri-Mori, Riedificazione di Reggio Calabria dopo i terremoti del 1783, in "Nuova Antologia", Fasc. 897, 1909, p. 89. Cfr. anche, sulla ricostruzione di Reggio Calabria dopo il terremoto del 1783: N. Aricò, O. Milella, Riedificare contro la storia. Una ricostruzione illuminista nella periferia del regno borbonico, Roma-Reggio Calabria 1984.

Furthermore, Mori also said that all the merit of the invention of the “Baraccata” system was to attribute to Francesco La Vega, in Italy. This because, under Bourbon Government, were assigned to him the activities of recovering and studying archaeological finds of Herculaneum and Pompeii, which influenced and inspired him to the conception of this system and its adoption in Calabria. In fact, the system have many similarities with the Roman "Opus Craticium".
The Bourbon system showed a ductile behaviour and therefore a seismic resistance even in its different versions, because create a box with the timber framings, connecting the perpendicular masonry panels, to reduce the building seismic mass.
It defines the most ancient of European anti-seismic rules, reporting the most important seismic design criteria, in use in the 18th and 19th centuries, which are still present in modern codes, such as shape regularity in plan and elevation.

Figure 2-10 - Bishop's Palace of Mileto (VV, Calabria, Italy), example of “casa baraccata”

The Mileto Bishop’s Palace, in Figure 2-10, built immediately after the catastrophic earthquake of 1783, is an example of load bearing system, executed exactly according to the Bourbon rules.
The reason why this constructive system is still being analysed is because it has shown good resistance to the telluric events that there have been.
As can be read in *The Report of the Italian Royal Committee of 1909 on the Messina-Reggio Earthquake*[8]:

“The Committee stated that it knows of houses built under these Bourbon rules that had resisted all the successive earthquakes. The system of wood-framed dwellings known as Baraccata, originated from these ordinances issued by the Bourbon Government, and … proved very successful so that the system is even today under certain circumstances highly commendable”.

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[8] Freeman, p. 569

Thus, after the 1908's telluric event, was also adopted the same construction technique, with only slight changes. It has been pointed that many buildings, with wooden skeleton, such as Casa Cammareri (Figure 2-12) and other buildings in Reggio Calabria, out slight damages, during the 1905 and 1908 earthquake.

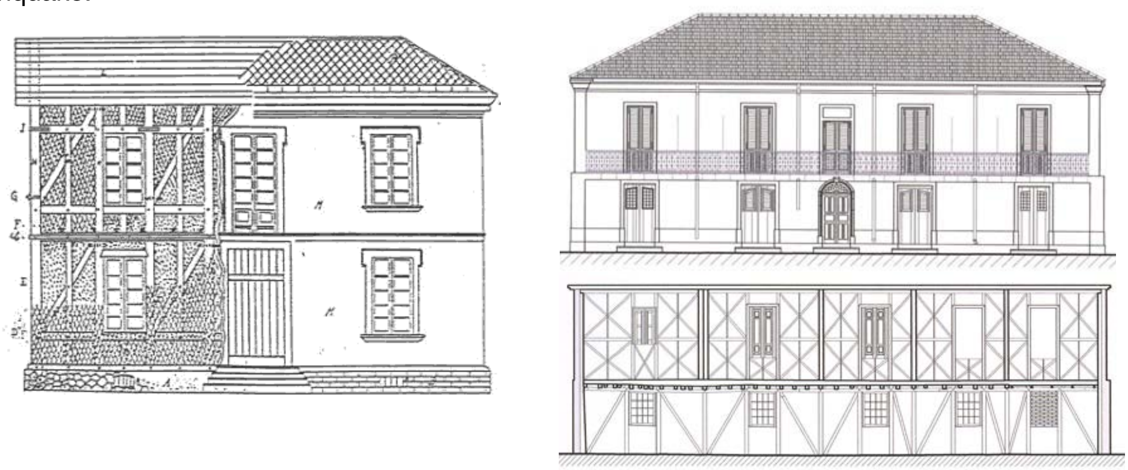


Figure 2-11 - (a) Patent wood walling for a-seismic house with wood frame and iron plates, L. Lanza (1909); **(b)** Facade and section of casa baraccata with timber bracings, built after the 1908's earthquake, in Gallico (RC), (Picture extracted from A. Bianco et al. [20])

In the description by Mori (p. 93), the "sistema governativo" did not foresee presence in the exterior walls, of timber bracings, or armor of wooden elements linked to each other like 'Saint Andrew crosses', filling the spaces with lime and plaster, probably because wooden elements could suffer from temperature changes and be attacked by insects. While these elements were admitted in the interior walls.

Wooden 'Saint Andrew crosses' are, in many framed constructions, absent. Even though, it is important to note which in some examples (Figure 2-11), following the reconstruction regulations after 1908's earthquake, were used these diagonal wooden elements.

In fact, in the 1909's prescriptions[9], can be clearly read the compulsory to stiffen 'case baraccate', already existing, to improve its behaviour under seismic actions, diagonal connections or timber bracings.

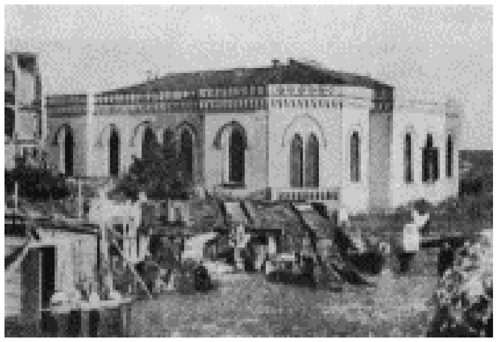


Figure 2-12 – Casa Cammareri, sistema baraccato (ME, Sicily, Italy)

Instead, in the Vivenzio's *Table II*, the diagonal wooden elements are present, arranged in the corners of the frame and bound to the two perpendicular rods, used to realize the doorways and windows, consisting of a more robust hooping.

The foundations used for the Bourbon system are of two types, in piles and surface type. In the description of Vivenzio, "Case formate di legno", wooden piles are well fixtures to a depth of 3 meters. For the Bourbon Code, it had to be used continuous foundations, on which stand out a masonry plinth,

[9] Regio Decreto 18 Aprile 1909, n. 193 [19], 3th article

maximum high five feet or a little more from the ground. This foundation characterizes the Mileto Bishop's Palace (VV).

Unlike Portuguese solution, especially in buildings built after 1908, there is the continuity of the vertical timber elements from the foundation to the roof, being anchored in the foundation (Figure 2-13). In Calabria, there are many structural variants because of the choice of timber typology, according to its ease of supply, the size and the arrangement of wooden elements and masonry filling.

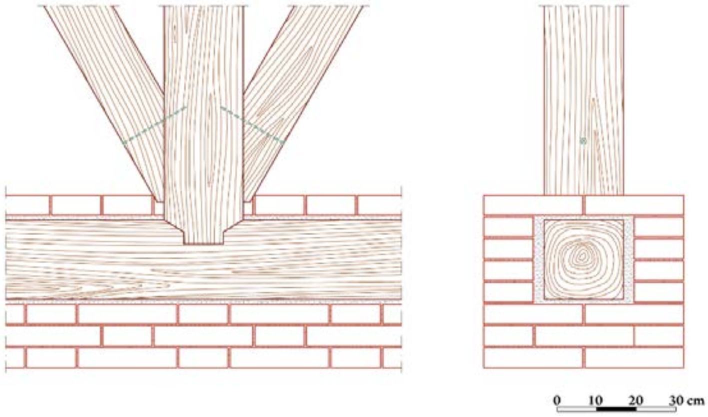


Figure 2-13 - Details of the foundation in the "casa baraccata", built after the 1908's earthquake (Picture extracted from A. Bianco et al. [21])

2.2.2 POMBALINO BUILDINGS IN LISBON

Figure 2-14 - Orthophoto of Lisbon, view of Baixa Pombalina, picture from DIGC (Càmara Municipal de Lisboa)

The "Gaiola" buildings were developed under the direction of the Marquis de Pombal (which is why it is also called "Pombalino construction"). Gaiola is the Portuguese word for 'cage', as consists on a three-dimensional wood truss that looks like a cage.

The *Pombalino buildings,* developed between parallel and orthogonal streets are usually grouped in rectangular and homogeneous blocks (about 70 x 25 m^2), as can be seen in Figure 2-15. Each block comprised approximately ten separate buildings and gable walls.

The block is not necessarily symmetrical and the buildings may not all have been constructed at the same time.

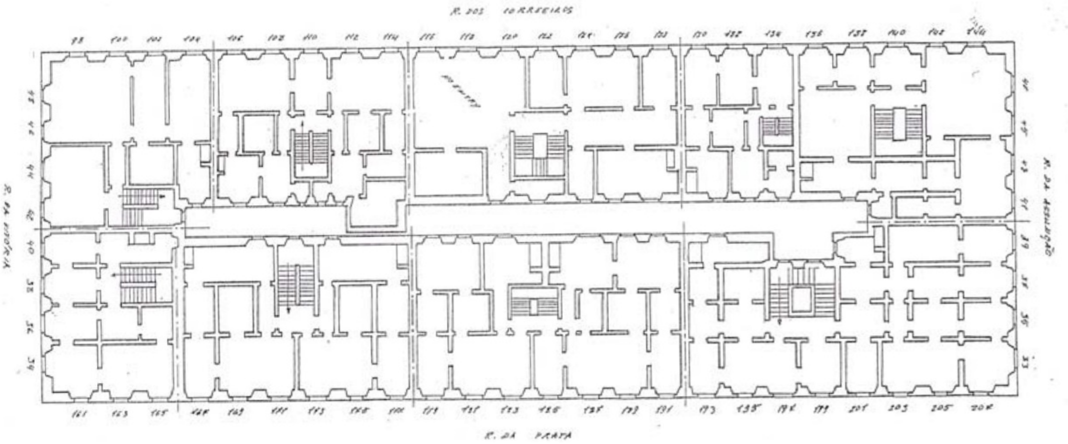


Figure 2-15 - The first floor plan of a given block (Mascarenhas, 1996, [15])

As shown in the Figure 2-16, the buildings were all the same height, with five storeys, comprising the ground floor, usually commercial, three residential upper floors and the attic. In this way the set of buildings of each quarter has similar dynamic characteristics, to improve the performance under seismic actions.

The attic storey, in the main streets and squares takes the form of a mansard roof, while elsewhere it takes the form of a hipped roof. There were also, 0.80 m above the roof, gable walls which rising to prevent fire from spreading between the adjacent buildings.

The stairs are resting on the "frontal" walls that are on the side of the stairs. As a solution to avoid the spread of fire, usually the first flight of stairs was of stone while the following flights of stairs were wooden.

The construction system is delimited toward the external of the building and the adjoining constructions by a masonry with thickness up to about 70 cm. This is an essential measure in order to compartmentalise the building and limit the development of fires. The thickness of the wall of the main and

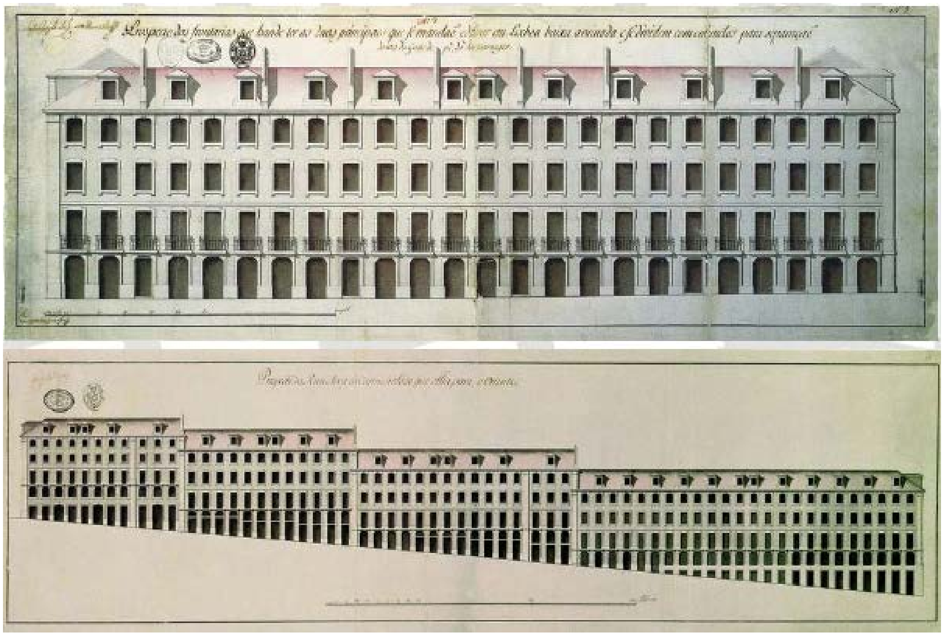


Figure 2-16 – Typical front view of Pombalino buildings (O cartolàrio Pombalino Câmara Municipal de Lisboa)

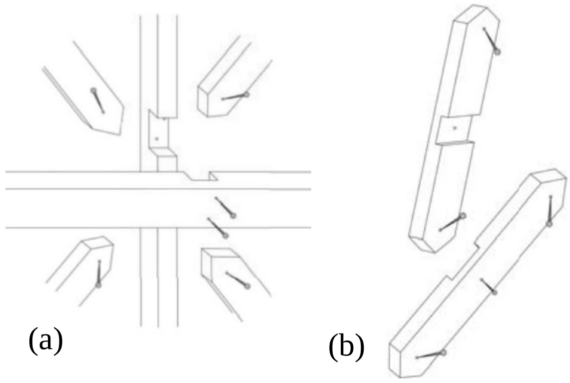
back façades decreases with height, starting from a thickness of 90 cm.
Their internal walls, the so-called “frontal” walls, are very important to understand their behaviour under seismic loading, because they have a structural function, in the behaviour of the entire buildings and are also deemed to resist seismic loads.
It consisted in a wooden framing braced by Saint Andrew’s crosses and masonry infill (heterogeneous material and the masonry was completely irregular). The choice of the triangles, to compose these wooden structure, is not random but is justified from the geometry of this figure, that cannot deform, without variation of the length of the sides. Therefore, each frontal wall only needs to mobilize the axial force of its bars to resist the forces in any direction in its own plan. These frontal walls, connected at the corners by vertical bars that belong to orthogonal frontal walls, with the timber floors formed a single system, like a cage (Figure 2-17 and 2-18). Therefore, the connection between orthogonal frontal walls by means of common vertical wood bars yields a three-dimensional truss capable of resisting forces applied in any direction. The Gaiola generally is not visible, because the surfaces of the frontal walls are covered with a finishing material and wood rips.

Figure 2-17 – Views of the Gaiola system after removal of cover and masonry, in a building recently demolished in downtown Lisbon

Figure 2-18 - Gaiola wall with the masonry filling

Different sizes of walls (heights and lengths) exist, such as a wall of 3x2, 3x3, or 3x4 modules. Three modules' height was used usually for the lower floors, while the two modules' height in the attic. The façades and gable walls (between adjacent buildings) are usually in rubble masonry. Even though, there are, mainly at corners and in some ground floor columns and walls, some exceptions of better quality masonry. According to the description of Mascarenhas [15], the section dimensions of the Gaiola members varied with specie of timber used and the location, with larger sections generally being used for lower floors.
There are also partition walls, in some examples, called ‘tabiques’, thinner and with much less resistance to horizontal loads than the frontal walls, made of one or two sets of boards or small wood bars. The ‘tabiques’, were used only in order to subdivide a space in a storey, do not belong to the Gaiola as they do not continue on the floor above and below. They are formed with planks nailed onto the struts then covered with laths and plastered.
About the connections, there are doubts about their quality, as well as of their widespread execution. It can be seen, for the middle central connection, in the figure 2-19a, that the vertical and horizontal struts

are cut at their mid-sections to attach them together, and in figure 2-19b how two diagonal elements are attached together.



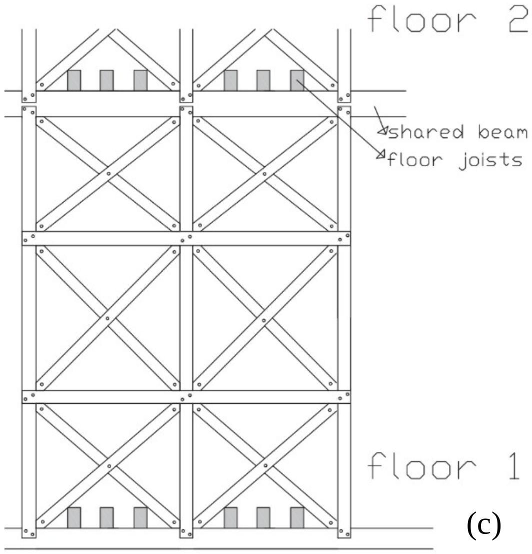


Figure 2-19 – Connections' details of the middle central connection **(a)**, of how two diagonal elements are attached together, cut at half their thicknesses **(b)**, and of "frontal" wall at floor 1 with upper floor 2 **(c)** (Pictures extracted from Helena Alves Meireles PhD Thesis in Civil Engineering (2012) [16])

The Figure 2-20 (b) shows that the horizontal element is not cut to half its thickness. Into the original practice, the façades and the frontal walls support the perpendicular wood joists (typical dimensions of these are 10x20cm^2), that support the floors composed by wood planks (typically 2 cm of thick). The frontal walls were anchored by means of iron anchors embedded in the masonry of the façades, as also the horizontal wood joists of the floors. While, iron nails and cuts on the wood bars were used to connect the different wood bars are done.

In the 1st floor (that is the ground floor's ceiling), was important to create a barrier to fire, in order that possible fires, on the ground floor, did not spread to the upper floors. That is why were used, for the ground floor level, arches and vaults in masonry, supported in the interior by masonry piers and on the exterior by façades and gable walls.

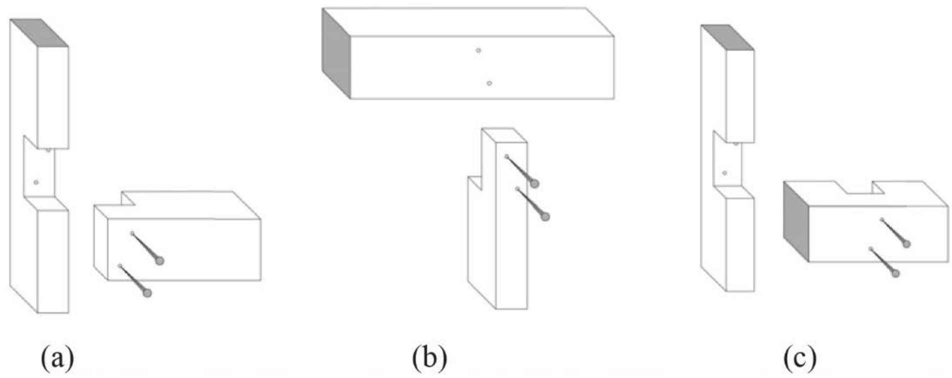


Figure 2-20 - Details of the "frontal" wall connections at left and middle **(a)**, at the upper middle **(b)** and at the middle central connection **(c)**.
(Pictures extracted from Helena Alves Meireles PhD Thesis in Civil Engineering (2012) [16])

This solution had to avoid also that the soil humidity will reach the Gaiola wood structure above the first floor.[10]

According to the description of architect Mascarenhas [15], on a large embankment, constituted by the debris of the buildings destroyed by the 1755 earthquake, were built the foundations of the Pombalino buildings. They are composed by three-dimensional grid, constituted vertical wooden pilings, compacted on the top of cross-platforms of wood bars (of around 1.5 m of length and 15 cm of diameter). The embankment receives the loads, from the structure, through by the wood grid, and each vertical wooden piling, with its substantial depth, distributes them by a larger area of the underlying soil, to reduce the stresses at this level.

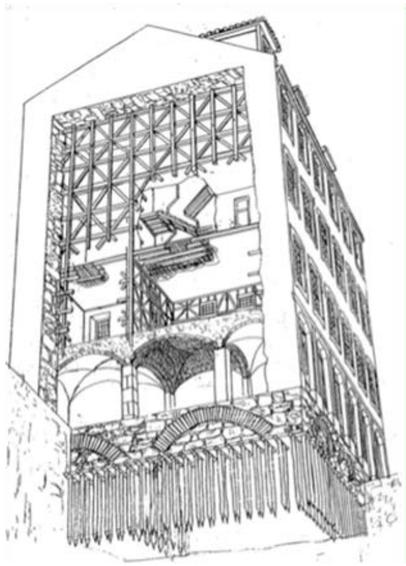


Figure 2-21 - Example of a Pombalino building (Mascarenhas, 1996, [15])

2.2.3 POMBALINO BUILDINGS IN VILA REAL DE SANTO ANTONIO

The use of Pombalino buildings was not limited only to Lisbon (Figure 2-22 (a)). In fact, spread around all Portugal as a model of earthquake resistant construction, becoming part of the Portuguese Local Seismic Culture. In particular, in the South of Portugal, in the region of Algarve, is located Vila Real de Santo António, an area with high seismic activity and many examples of this construction system.

The city, on the border with Spain, was an important example of reconstruction, due the Marquis of Pombal, after the 1755 earthquake. The territory was practically abandoned at the time, because the effect of the quake was very devastating, like for Lisbon. Thanks for the considerable economic potential of the place for its strategic position and for the control of port transactions, the Marquis of Pombal, between 1760s and 1770s enacted an official recovery program to setting-up of a

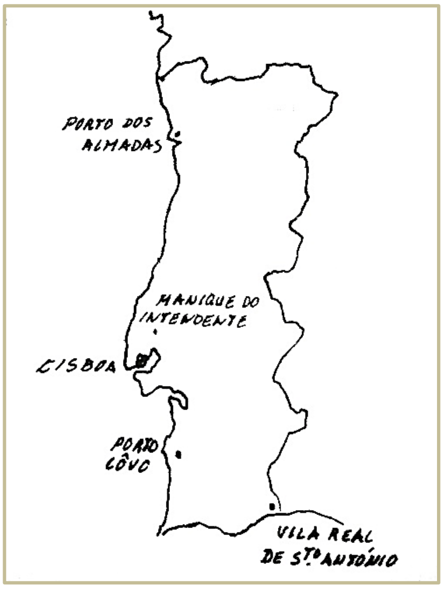


Figure 2-22 (a) - Distribution of Pombal core in Portugal, (Picture extracted from Mascarenhas [26])

[10] Author of a series of fundamental books that trained generations of engineers and architects, Mascarenhas, is a Portuguese reference in the field of the construction details of the Pombalino buildings. He had the chance to observe directly the constituent elements of this construction system from many buildings that were being completely or partially demolished some decades ago. Therefore, the descriptions presented in his PhD thesis work in 1996, is really important to understand the details of these buildings.

completely new planned town. The city, built at the end of the 18th century, contemporary to Lisbon, follow similar urban and architectural construction's solutions.

All the territory where Vila Real de Santo Antonio stands is made up of dunes, a sandy area, protected for boats, where the Guadiana River creates a delta.

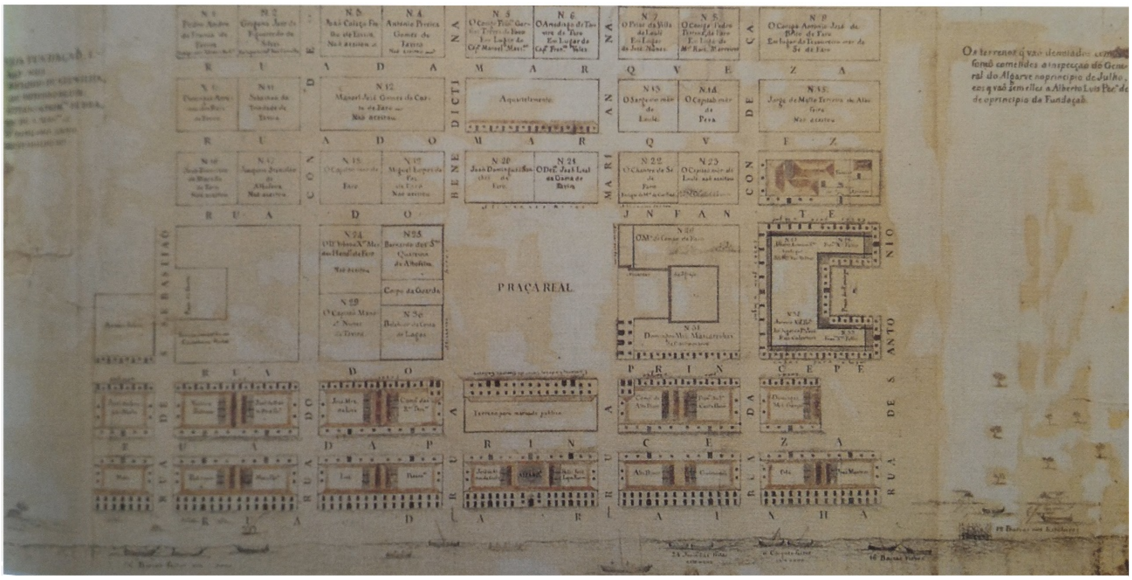


Figure 2-22 (b) –Vila Real de Santo Antònio plant, still incomplete - Some quarters are missing, namely the two turrets of Rua da Rainha from R. Figueiras [38]

The main problem for reconstruction was to level the ground as the area near the river had a different level of the upstream area. It started building near the river, and then inward, levelling the level of sand, and creating deeper foundations for buildings far from the river (about 2.5 mt), and less high for those near the river.

The new city plan provided a grid of seven by six urban blocks organized around a big central square (Figure 2-23 (b)), that marked a rectangular area with one of the long sides placed along the Guadiana River. The dimensions of all the blocks were about 53 m long and 22 m wide, except for the central one, which is slightly longer (55 m). Streets were 9 m wide.

Figure 2-23 - Original plan of Vila Real de Santo António and main building types (Picture from Ortega et al. [25])

The plan is characterised by a great regularity and organization at an urban level with four different construction types. As can be seen in the Figure 2-23, are present:

- ❖ in the riverfront buildings which have two main storeys and a third attic floor (in magenta colour);
- ❖ in the main square there were buildings, with two main storeys (in yellow colour);
- ❖ in green colour are represented single storey dwellings, characterized by their small scale and simplicity;
- ❖ Single storey factories (in orange colour) and warehouses (in pink colour), organized around a patio, are characterized by a system of masonry arcades perpendicular to the façade walls.

Finally, there were also the Customs House, the church, and the 'towers'.

The exterior walls are in disordered stone masonry with schists and sometimes-solid bricks, usually 70 cm thick, while the interior partition walls, usually 45 cm thick, are in solid bricks masonry. The exterior walls of the upper floor had an inner 1/2 palm thickness less than the ground floor, so that they could support the floor beam. The binder used was aerial lime. The stones came from Castro Marim while the wood used was Pine Nordic or Manso. Timber frame walls were used in some buildings only in upper floors, especially in the attic walls that are side by the window. In timber were also the roof rafters, trusses, and floor beams. As in Lisbon's buildings, some ground floor rooms had vaulted ceilings supporting the first floor, as a measure to prevent fires. The buildings outside then were plastered with lime and sand and whitewashed.

Figure 2-24 – Timber frame partition walls Alfândega building with different geometries (Picture from H.Ea.R.T 2015 [25])

Similar to those used in the others Pombalino quarters, frontal partition walls are present only in the South Tower, (showed in red in Figure 2-25) in front of the Guadiana River to connect the timber roof structure with the timber floor structure, as in Lisbon's Baixa. As can be seen in the Alfândega or Customs House (Figure 2-24), one of the most representative buildings of the city that still present an example of these timber frames, with vertical, horizontal and diagonal timber members filled with rubble masonry. Probably in VRSA you can not find so many frontal walls because compared to Lisbon case the buildings were lower, usually of one or two floors, and the builders came from the Algarve area, so they had different techniques, despite they dating back to the Illuminist period. Unfortunately, in the Pombalino core, only very few buildings still possess these original characteristics (Figure 2-25), because a transformation process, with large alterations, demolitions and an extreme densification of the urban fabric, characterized all buildings.

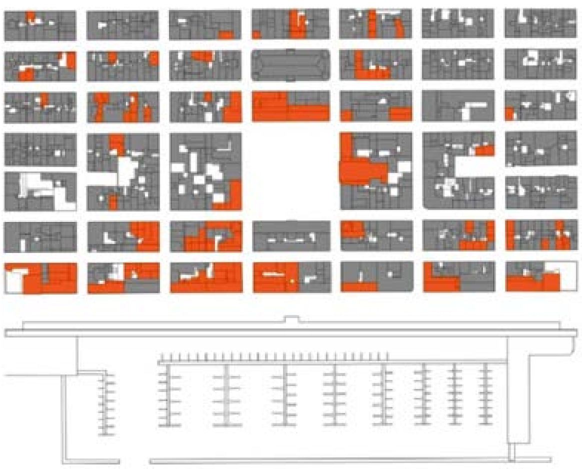


Figure 2-25 - Current plan of Vila Real de Santo António (highlighted in orange those buildings that still preserve some original characteristics) (Picture from Ortega et al. [25])

The original patios, in fact, were continuously occupied by additional constructions. In addition, the original frontal walls, important to improve the buildings' behaviour under the earthquake were eliminated, and in some case, as in the rehabilitation of the South riverfront 'tower', were reconstructed but only as an aesthetic element[11].

This, not only makes buildings more vulnerable to seismic actions, but also makes it lose an important architectural and cultural heritage.

Therefore, as Pombalino buildings' case study is selected the city of Vila Real de Santo António (VRSA), because it followed a development contemporary to the reconstruction of Lisbon. It is important to see, as the Pombalino core in VRSA has most similarity with the Bourbon system, because have usually two storeys, like casa baraccata, a certain regularity in plan and elevation and the foundation system. Strip foundations are directly built below the lowest part of the building. The thickness of the foundation depends on the strength of the foundation material, following a general rule, where the projection of the masonry strip each side of the wall should be no greater than the thickness of the wall in elevation. In this case, the strip is one palm greater than the thickness of the ground floor walls, like the Calabrian technique.

2.3. COMPARISON BETWEEN POMBALINO BUILDINGS AND BARACCATA SYSTEM

The contamination of the experiences coming from the other cultures of the Mediterranean basin, has allowed learning from the destructive earthquake events, improving the local construction systems. For this reason, the Bourbon Code (*Istruzioni Reali*, 1784) could be derived by the study of the solutions and rules, applied by the Portuguese government aftermath the 1755's earthquake, even though this traditional construction technique was used, in Calabria, already before the 1783's earthquake.

Both systems were built to improve the construction under the seismic action.

It can be noted the disposition of wooden elements, that in Gaiola system is only in the inner walls, unlike the casa baraccata that presents also in the exterior walls, like a cage for the masonry. For this reason, the wooden elements for the casa baraccata were embedded in the masonry or were covered, to preserve them by the atmospheric agents and insects.

The main differences, already partly mentioned, concern the external walls, the inner walls and the ground floor. The casa baraccata system were characterised by external walls, constituted by timber framings that enclosing the masonry, and the inner, thinner walls (about 25 cm) were made of wood and uncoated masonry, or sometimes with simple frames filled with brick, without connections to the floors. The Gaiola system, instead, showed external walls were only in masonry and the inner walls, with timber frames, strictly connected to the floor. Therefore, in case of earthquake, should have supported the floors and the same roof, even though the exterior masonry walls had fallen, given its independence from the latter.

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[11] Images of this are present in J. Ortega, G.Vasconcelos, H. Rodrigues and M Correia, "*Local Seismic Cultures: The Use of Timber Frame Structures in the South of Portugal*" including in HEaRT 2015

About the ground floor, the Pombalino buildings do not present wooden frames and were used arches in masonry to carry the loads to the piers/walls and the piers to the foundations, while for the Casa baraccata, arches and stone vaults were prohibited, and wooden frames were used in the walls, on all floors (two), to a maximum height of 28 palms.
The behaviour of the structure of the Bourbon system, under the seismic action is no longer entrusted to masonry alone, but to the mixed structure where the masonry cooperates with the wooden elements. However, the most frequently local wood used, for the casa baraccata, is the Calabrian Chestnut. In the Pombalino buildings, instead, typically were used softwood species, such as Pines, because the hardwood species, such as Oaks and Chestnuts, were harder to find in abundance, so their use was limited.
Highlighting the differences between the Pombalino buildings, used in VRSA and the "baraccata" system, present in Calabria, certainly, both buildings typologies have more similarities than the original Pombalino system, used in Lisbon. The ‘casa baraccata’ was a modest type of townhouse, consisting usually of an isolated block with biaxial symmetry and with one or two floors. The Pombalino buildings in VRSA also are characterized by one or two storeys and a great regularity, deriving from the accurate planning of the quarter.
An important aspect is their deployment, in fact, the VRSA's buildings, constituted by rectangular blocks that formed a quarter, were included into a reconstruction plan, decided by Marques de Pombal and every building have the same geometrical and structural features, while the Calabrian "case baraccate" are scattered throughout the territory. In Calabria, especially for buildings built after the 1908's earthquake, is the owner of the building, a normal citizen, without technical knowledge, that chose to use wooden frames with masonry infill, rather than traditional masonry or concrete, to make his home safer in case of earthquake. In fact, the "Istruzioni Reali" promulgated by the Bourbon Government were guidelines, and did not necessarily have to be applied.
Therefore, in Calabria, there is no core of buildings, like the 'Pombalino core' in Lisbon or in VRSA, and it is difficult to identify, what and how many, "case baraccate", are scattered around the territory, because there is no cataloguing of these.
Thanks to the damages' descriptions, that the buildings had suffered because of the earthquakes, we have a possible cataloguing and localization of these, but few building with timber frames, are still intact and functional, after the 1908’s earthquake, and even less after the 1783's event. Many buildings have been demolished, others have suffered deep changes and other was abandoned.
In conclusion, the use of these two materials, masonry and wood, and their collaboration influence and determine the overall behaviour of the building under cyclic loading. Therefore, to understand all its aspects and in order to evaluate a correct conservation and rehabilitation approach is essential, study their connections, the wooden elements' maintenance and the quality of mortar and masonry.

2.4. STATE OF THE ART

In scientific literature, there are few studies about the use of wooden frames in historic buildings. About Bourbon constructive system, Tobriner[12] reported about several examples with frames braced with Saint Andrew's crosses, e.g. at Seminara (RC), Palmi (RC) and Reggio Calabria, and a private house in Filadelfia (VV), nowadays unfortunately demolished. In addition, Barucci, already mentioned previously, conducted an historical analysis, focused on constructive typologies developed after the 1908 earthquake, including the description of the Vivenzio's seismic prototype. It is also important to mention Ruggieri's studies, which addressed exhaustively the application of the 'casa baraccata' system, under Bourbon Government, in eighteenth century Calabrian buildings.
Focusing to Vila Real de Santo António, the *Plano de Pormenor de Salvaguarda do Núcleo Pombalino* (SGU 2008) is an important recent work, conducted to aim to analyse the transformation of the 1773 Pombaline core. The originally designed plan suffered important alterations and show big degradation in the historical built-up fabric, in fact it was highlighted that only 155-164 buildings still possess Pombalino characteristics.
Subsequently, Ortega et al. (2015) conducted a deep analysis the Alfândega or Customs House. The selected structure occupies a central position in a block in the riverfront of Vila Real de Santo António. About its constructive characterization, this building results substantially less altered than others are, and shows most of its original features in elevation and only some alterations in the volume of the building. Experimental and numerical methods were applied in the analysis to study seismic behaviour of the Alfândega, taking into account the influence of structural alterations done in the building. These structural alterations have jeopardized the seismic safety of the building; in fact the numerical analysis shows the failures only in unoriginal structural elements.

2.4.1 EXPERIMENTAL PROOFS

In Portugal, the first cyclical test on the Gaiola system was carried out by Santos in 1997 on a sample taken from an existing building that was later demilished.
Experiments performed by Ceccotti et al., 2006, on real scale models of masonry with wooden frames, typical of the rural architecture of the Dolomites, were made with two pairs of hydraulic pistons, fixed with metal chains at the four corners of the wooden frame.
Some experiments were conducted in Pakistan, in 2011, to understand the behaviour of the buildings, called *Dhajji Dewari*, under seismic actions.
More recently, in 2012, in the IST Laboratories of Lisbon, experimental campaigns (Figure 2-26) were launched with models reproducing the frontal walls modules (Meireles et al., 2012 [16]).

[12] S. Tobriner (1983) La Casa Baraccata: Earthquake-Resistant construction in 18th-century Calabria. J Soc Arch Historians 42(2):131–138

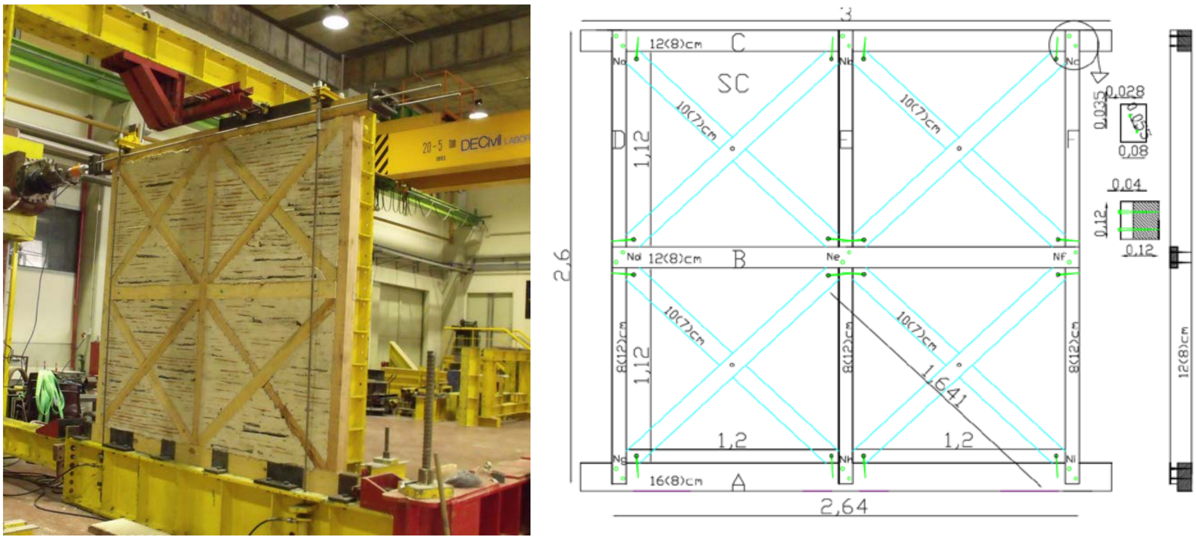

Figure 2-26 – a) Final layout of the experiment with load cell, LVDT and jacks; **b)** Schematic of the “frontal” wall specimen (units in meters otherwise specified) (Pictures extracted from H. A. Meireles PhD Thesis in Civil Engineering, 2012, [16])

In 2012, also Vasconcelos et al. have conducted tests on samples of the Pombalino system, comparing them to other types, reinforced by means of plates or metal connectors in the nodes. Still, in 2012, Quinn et al. tested some models of *Quincha*, a constructive masonry system with wooden frames, used in higher levels of buildings in Latin America and Peru in particular.

Figure 2-27 – Real scale model of the wall of Mileto Bishop’s Palace, Laboratories of the Ivalsa CNR (Trento, Italy), 2013

In Turkey, Aktas et al. investigated the cyclic behaviour of the wooden frame system, known as *Himis*. The tests were carried out on frames of different geometries, with adobe filling and others without masonry, and with the presence of openings. Finally, in 2013, in the laboratories of the Ivalsa CNR (Trento, Italy), a real scale model of a wall of the Mileto Bishop's Palace (Figure 2-27) was reproduced, to test and understand the behaviour of the structure, under seismic actions. Petrographic analyses and chemical analysis on mortar were carried out by the departments of University of Calabria (UNICAL) and then, the wooden frame, with and without masonry, have been tested[13] under cyclic actions. The frame, made of chestnut wood, presents only horizontal and vertical wooden elements, but is devoid of Saint Andrew’s crosses.

[13] N. Ruggieri conducted an experimental campaign on the masonry wall of the Palazzo Vescovile in Mileto and addressed extensive studies on the seismic knowledge in the Naples’ Kingdom and the Bourbon system application in Eighteenth Century Calabrian buildings

This experimental campaign[14] has recorded an extremely ductile panel behaviour [the value about of 7 (Vu/Vy)], so to classifiable among the structures that present a high ductility, according to the Eurocode 8. The wooden frame, showing limited damages, in general, remained in elastic domain for the entire duration of the test.

While the fragile mortar beds have facilitated the slip and in some areas the expulsion of stones. Therefore, the composite system, as a model of the Mileto's Bishop Palace wall, thanks to its masonry features, has dissipated a discrete energy quantity.

2.4.2. NUMERICAL ANALYSES

In the scientific literature are not present studies about the seismic vulnerability analysis by numerical approaches, of the buildings characterized by the Bourbon System.

A simple wall module, deriving from a building, built according the Bourbon rules, was studied by Galassi et al.[15] using the software BrikWORK for a preliminary mechanical interpretation.

The study, conducted by Galassi et al., established the timber frame, modelled by using finite elements with elastic and bilateral behaviour and, the masonry infill, modelled by rigid blocks connected with unilateral elastic contact constraints.

With unilateral contact constraints, taking into account friction, were represented the contact interfaces between wooden elements and masonry blocks.

Kouris and Kappos[16] used ANSYS software, for a model of a masonry walls reinforced with timber elements, in a Greek traditional building. Their approach, to carried out the numerical analysis, consisted in a model where were used linear-elastic beam elements to reproduce the horizontal and vertical elements, while for the diagonal elements were used a link element, characterised by the presence of a plastic axial spring, pinned at its ends.

The results of the numerical analyses were confronted with cyclic loading tests on timber-framed masonry, and has showed a good match.

Ramos and Lourenço[17] created a model in DIANA for traditional Pombalino buildings, and in particular, to evaluate the contribution of their internal walls, the "Gaiola", to the behaviour of the whole structure, under seismic actions. Also, for this model, quasi-static cyclic tests were conducted on three specimens removed from existing Pombalino buildings, to confirm the numerical analyses.

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[14] Ruggieri N, Tampone G, Zinno R (2015) In-plane vs out-of-plane "Behaviour" of an Italian Timber Framed System: the Borbone constructive system. Historical analysis and experimental evaluation. Int J Cult Arch Heritage 9:1–16

[15] S. Galassi, N. Ruggieri, G. Tempesta (2015) Seismic performance evaluation of timber-framed masonry walls. Experimental tests and numerical modelling. In: Ruggieri N, Tampone G, Zinno R (eds) Historic earthquake resistant timber frames in the Mediterranean area. Springer International Publishing, Switzerland, pp 95–103

[16] Kouris SLA, Kappos AJ (2012) Detailed and simplified non-linear models for timber framed masonry structures. J Cult Heritage 13:47–58

[17] Ramos LF, Lourenço PB (2005) Seismic analysis of a heritage-building compound in the old town of Lisbon. In: Proceedings of the international conference on the 250th anniversary of the 1755 Lisbon Earthquake. Lisbon, Portugal, pp 362–368

The results of the seismic analysis showed a safety factor, different by the value required by the Portuguese code and a collapse induced by the out-of-plane failure of the external masonry walls.
A numerical modelling in DRAIN2DX, based on the results provided by an experimental campaign performed in the CNR-IVALSA laboratories in 2013, has been developed to assess the seismic behaviour of a two-storeys building, under the prescriptions including in the Bourbon code.

2.5. SIMILAR CONSTRUCTIONS

The timber framed building has been an historic construction solution adopted in different locations at different times, namely in seismic regions.

Figure 2-28 - Timber-frame houses: in the left typical house of Lefkada Island (Greece), on the centre the houses in the medieval town of bad wimpfen in baden wuerttemberg, on the right St Bartholomew Gatehouse, built in 1595 in London

These constructive systems were spread in all Europe, usually after the earthquakes for the reconstruction. There are examples of these in Portugal (*edifícios Pombalinos)*, in Italy *(casa baraccata)*, as can be seen in Figure 28 and 29, in Germany *(fachwerk)*, in Greece, in France *(colombages* or pan de bois), in Scandinavia, in the United Kingdom (half-timber), in Spain (entramados).
Timber frame buildings, as already mentioned, can be found in areas where there is no earthquake risk, like in Scandinavia. In these places, it is the most common way of constructing buildings, both private and public, especially in areas where bricks could be easily supplied.
The technique is not so different from the others country in the rest of Europe, then can be recognized two or three floors, a timber cage based on the repetition of a little module in plan, lied over a masonry foundation.
The main difference between the constructions in seismic and non-seismic areas is that seems in the Scandinavian wooden frame lacking the presence of crossed braces, which on the contrary is typical in seismic countries' configuration.

Figure 2-29 – Pictures of timber frame buildings in Europe: on the left Anne Hvide's House built in 1560, (Denmark), on the centre a XV century house in Paris, and on the right a traditional house in Lund (Sweden) [Copani, 2007]

Regarding the timber frame, used in Lefkada Greek island, it is completely independent from the masonry structure and is supported both by the ground floor masonry walls and a secondary timber frame that offset at 5 – 10 cm from the inner face of masonry wall.

The Ottoman houses have influenced the conception of the timber frame system. These constructions present load – bearing masonry walls on the ground level, and the infilled timber frame, with horizontal, vertical and diagonal members, for the upper storeys.

Bricks, adobe, or stone with earthen mortar filled the space between the wooden elements, depending on the local available materials.

Figure 2-30 - Hatil at ground floor and himis in upper storeys (Turkey)

Several variations are derived from the Ottoman house, like *himis*, *bagdadi*, *muskali dolma*, and *goz dolmasi*. They have timber frame usually left in sight and a rectangular plan with two or three storeys. The main difference between them are the materials for the infill.

Outside Europe, the construction technique, named as himis, in Turkey, consist in a masonry (used masonry types are brick, adobe or stone) reinforced by timber frame (Figure 2-30). The construction techniques used to build until 1950, included himis, bagdadi and masonry structural elements. After the 1950, the spread of reinforced concrete as an innovative construction techniques structures has been widespread all over the world, also in Turkey, even though these traditional systems are still very much used today. Main posts, posts, tie beams, and the braces characterize its structure. It is important the quality of the masonry wall, that infilling the spaces between the timber elements, has the task of transfer the loads from roof to the in elevation structure, and finally to the foundation. The composition of the masonry structure is different and depending on the floors, usually on the ground floor the lining consists of natural stone and adobe, to isolate building from the humidity of the soil, instead the upper floors are characterized by *bagdadi* (Figure 2-31), since it is lighter compared to masonry. Two main traditional construction systems were carried out after the beginning of the 19th century in Latin America: *Taq* and *dhajji dewari*. The *Taq* construction consists of load-bearing masonry walls with horizontal timber embedment. An important detail, to hold the timber elements in place, is that each timber element is placed into the masonry walls and secured by the ends of floor joists that penetrate through the exterior walls. Without any vertical reinforcements, the *Taq* system is able to resist to the earthquake forces, but the lack of vertical reinforcements enhances the excessive weight of masonry. The other construction system, *dhajji dewari*, use a similar technique to the himis or other construction types that derived from the Ottoman houses. It consists of a timber frame that is infilled with masonry, frequency used for the upper storeys of buildings with *Taq* or unreinforced masonry at the lower levels. In this way, the *dhajji dewari* system, which is lighter than the systems underneath, provides an excessive weight while lowering the centre of gravity of the structure.

Figure 2-31 – *Bagdadi structures* in Turkey

In Kashmir in India, as can be seen in Figure 2-32, the infill is usually brick that made of fired or unfired clay or rubble stones, depending on the local available materials.

A probable evolution of *Taq* construction technique might be *cator and cribbage* system, even though it is heavier and with more timber elements. It can be found in the Himalayan Mountains of northern India, northern Pakistan near the Chinese border, and parts of Afghanistan, only where wood supplies are abundant. A cribbage of timber that filled with masonry constitutes the corner of the walls. The timber belts (cators) extend across the walls, like in *Taq* system. There are many variations in the Middle East, North Africa and Central Asia, Nepal, Bhutan, Tibet and other parts of China.

Figure 2-32 - Dhajji-dewari building in Kashmir (India)

In Haiti, can be seen the famous Ginger-bread house (Figure 2-33) that present many similarities with the Pombalino's frontal walls, and above all with the casa Baraccata's structure, because the external walls are not only in masonry, but entirely in-filled timber frame.

Figure 2-33 - Ginger-bread house in Haiti

Figure 2-34 - The so-called Opus Craticium, an example of timber frame with masonry infill, in the ancient Roman town of Herculaneum, found after the archaeological excavations; Photograph by Randolph Langenbach

Even though, the origin of timber frame structures probably goes back to the Roman Empire, as can be seen in Figure 2-34, where, in archaeological sites, buried in the eruption of Vesuvius, half-timbered houses were found. Vitruvius referred to these as Opus Craticium. Timber members were used also in the Minoan palaces in Knossos and Crete.

Each country has used different typologies of timber frames technique (braced and non-braced, that present vertical, horizontal or diagonal elements, with different connections between each other), different materials for the infill. However, the common idea have been to use the timber elements to make the masonry more ductile. It is well known that, timber frame can resist to tension, contrary to masonry, which resists to compression, so, the ancients had already understood these mixed structures is perfect especially to improve the mechanical properties to shear loads and provide a better resistance to horizontal loads, using the timber elements as a sort of confinement to the masonry structure.

2. PORTUGAL CASE STUDY

3.1. POMBALINO CASE STUDY

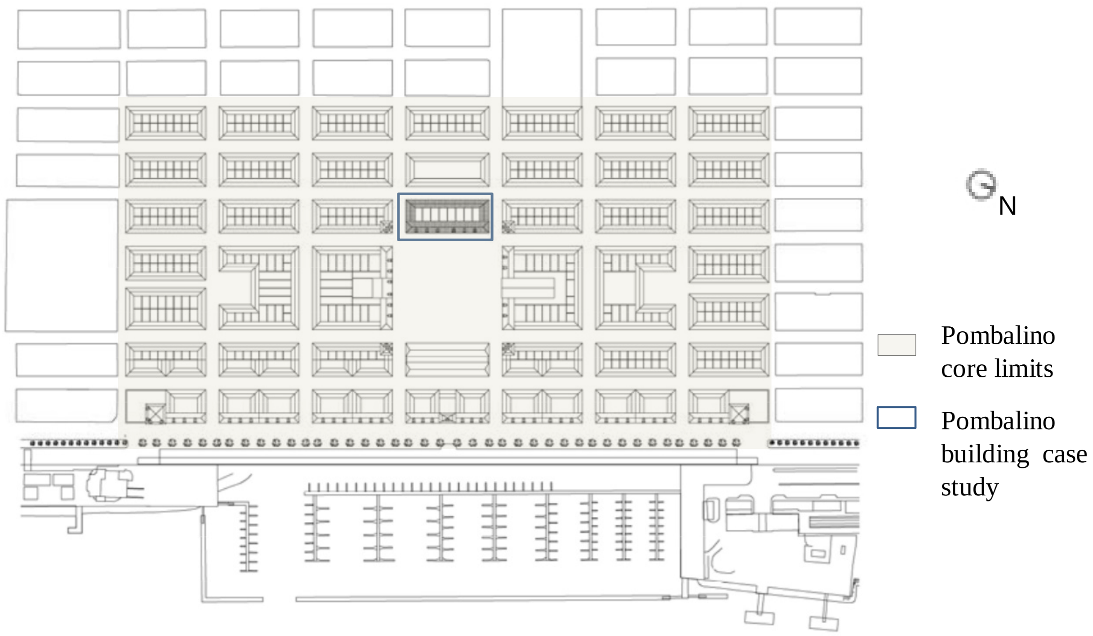


Figure 3-1 – VRSA plan view Scale 1:5000

The building that was chosen is part of the Pombalino Core in Vila Real de Santo António. It presents different features from the Pombalino type that can be found in downtown Lisbon, but is characterized more or less by the same period of construction. This existing building is delimited to the east by the main square of the city, Praça Marques de Pombal, to the west by Rua Dr. Sousa Martins, to the north by Rua Dr. Teofil Braga and to the South from Rua 5 de Outubro.

Figure 3-2 – Picture of the building during the on-site survey in VRSA

It presents two floors plus attic in the block in front of a square, including the ground floor, that has a commercial use (restaurant, bar and shops), and the first floor, mainly for residential use, as only one local is occupied by a sport club. While in the west side, the original block presents only one commercial floor, as can be seen in Figure 3-2.

As is usual for these traditional buildings, the original layout of the building has been subject by many modifications, as it was detected for the building during the on-site survey in VRSA. However, in the present work, one refers to the original layout selected for analysis. Certainly, the existing changed building will have a behaviour that is worse than the original layout analysed in this chapter.

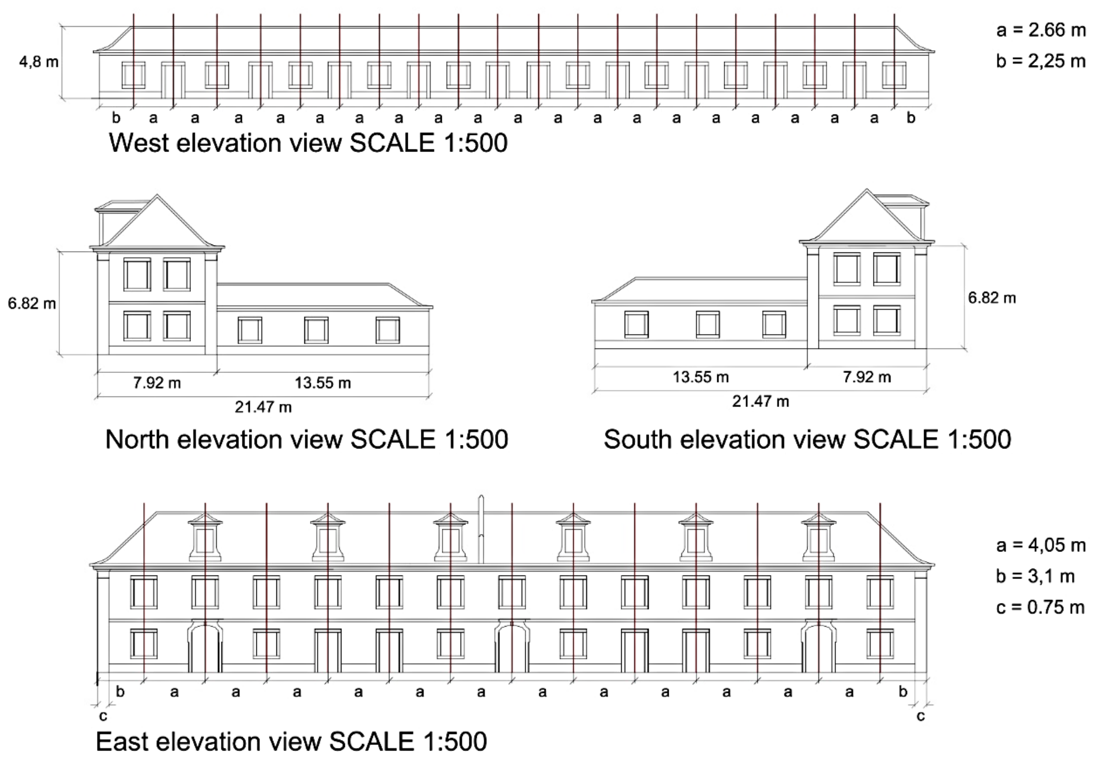


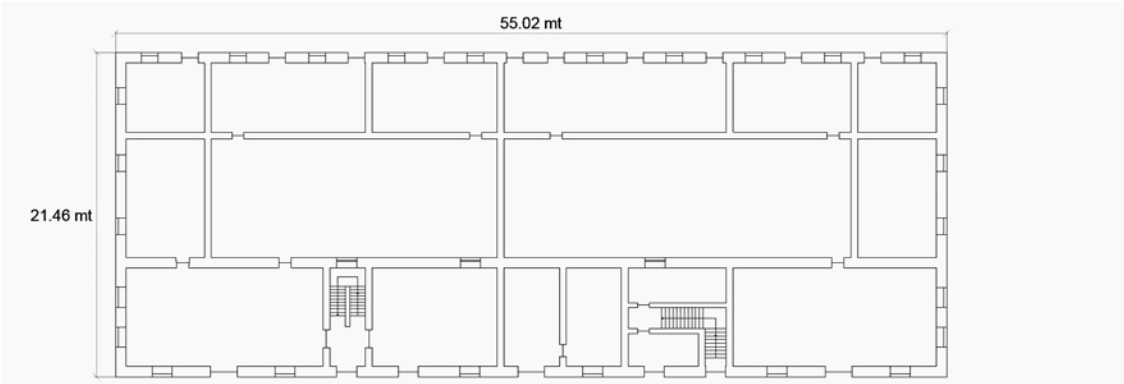
Figure 3-3 – Elevation views of the building's block in front of a Praça Marques de Pombal Scale 1:500

The east façade, that is, in front of the square, is as the original layout but in plan is characterised by many alterations. The other sides of the building, instead, present many alterations in plan and in elevation, because one floor has been added to the original layout of one floor. The drawings that are subsequently showed are based on the drawings present in Plano de Pormenor de Salvaguarda do Núcleo Pombalino de VRSA (SGU), and a personal on-site survey.

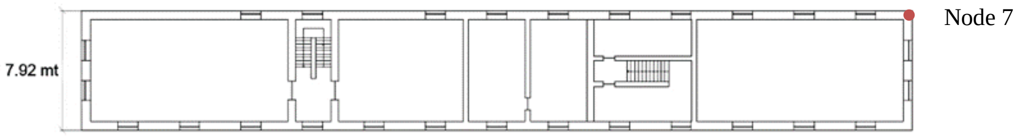
As can be seen in Figure 3-3, the building has not regularity in elevation, because it consists of two blocks of different height.

Table 3-1 – Parameters to classify the building for the seismic demand

Site	Vila Real de Santo António
Importance class	II
Importance factor C_u	1
Reference return period T_{NCR}	475 years
Ground type	B



Plan view of the ground floor of the Pombalino case study



Plan View of the first floor of the Pombalino case study
the red dot highlights the selected node 7, as control node of the structure,
in the 3Muri three-dimensional model to derive the capacity curves

Figure 3-4 – Plan drawings of the ground and the first floor of the building, Scale 1:500

The building has a height of approximately 6.8 m until the last floor (without the height of the roof). The dimension of the ground floor in plan are about 55 x 21 m^2, while the first floor, of the block in front of a square, are in its original layout about 55 x 8 m^2.

Table 3-2 - Type of material and thickness of the structural elements that characterised the building

Element	Material	Thickness [m]
External walls (ground floor)	rubble masonry with schists	0.70
Internal walls (ground floor)	solid brick masonry	0.45
External walls (first floor)	rubble masonry with schists	0.59
Internal walls (first floor)	solid brick masonry	0.45

The mechanical characteristic of the materials to adopt in the three-dimensional model of the building developed in 3MURI software[18] have been identified according to the on-site building relief and an interview with a technical employee of the VRSA municipality.

On the first floor, the thickness of the external walls are about 11 cm less than the walls on the ground floor, so that they could support the floor beam (as can be seen in Table 3-2).

Regarding the mechanical characteristics of the masonry types, shown in Table 3-3, it was decided to use the mean value of the two, a maximum and minimum, present in the Italian Normative [NTC2008 and Circolare 2 Febbraio, 2009, n. 617, table C8A.2.1] for each description (class) of a type of masonry.

[18] The software was distributed by S.T.A. DATA s.r.l. is based on the structural calculation code, TreMuri

Furthermore, it has been assumed that the masonry is already cracked and so a 50% reduction factor was used for the Young's Modulus E and the Shear modulus G.

According to *C8A.1.A.4 Costruzioni in muratura: livelli di conoscenza, included in Circolare 2 febbraio, 2009, n. 617*, it was chosen the LC2 knowledge level, given that have been carried out the geometric relief, on-the-spot investigations, extensive and exhaustive, on constructional details and on the properties of materials; the corresponding confidence factor is FC = 1,2. The binder used in the walls was aerial lime.

Table 3-3 - Mechanical characteristics of masonry types

Materials type	**Average Young's Modulus E** $[\frac{N}{mm^2}]$	**Average Share Modulus G** $[\frac{N}{mm^2}]$	**Weight w**	**Average Compressive Strength** f_m $[\frac{N}{cm^2}]$	**Average Shear Strength** τ_0 $[\frac{N}{cm^2}]$
Rubble masonry	870	290	19	140	2,6
Brick masonry	1500	500	18	320	7,6
Nordic Pine	12000	750	5	-	-

The mechanical characteristics of the wood considered in this work are 12 GPa for the Young's Modulus and 0.2 for the Poisson ratio. The joists of the floors have a section of 11 x 10 cm^2 and the wood pavement a thickness of 4 cm (Figure 3-5). The joists run every 50 cm.

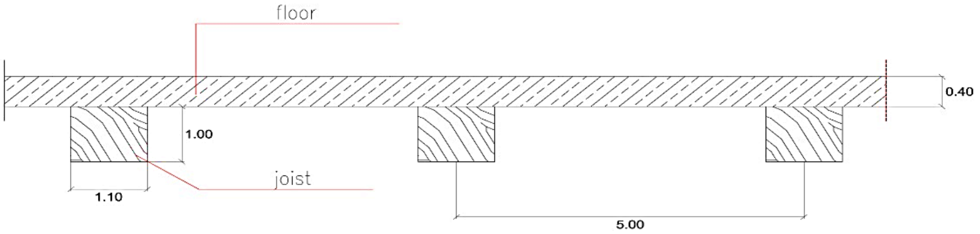


Figure 3-5 –Section view of the floor Scale 1:10

Phases of model development:

Starting from the plan view of each floor, imported in the program in .dxf format, where the middle axis

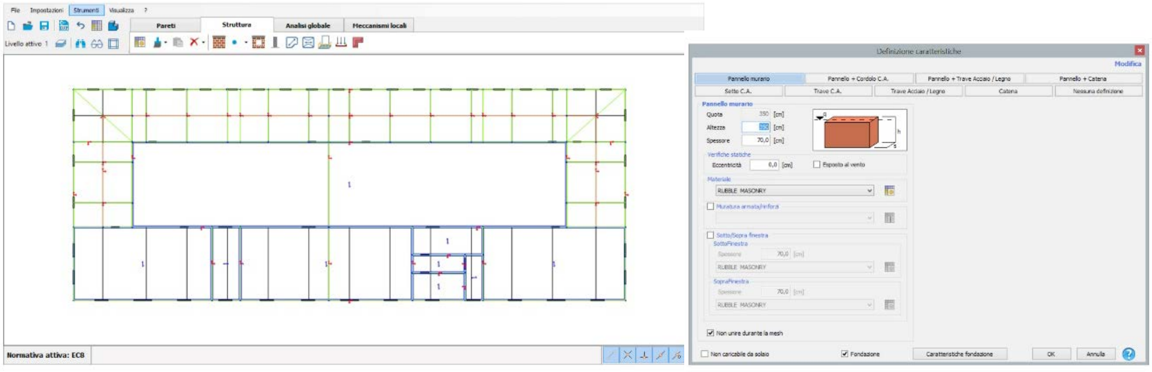


Figure 3-6 – View from 3Muri Program, development of the model

of the wall and openings were highlighted (Figure 3-6), the software allows to generate a model, where the geometry of the structural elements and the openings are introduced.
Then it is automatically discretized and the equivalent frame for each wall is defined.
The figures 3-7 and 3-8 show the threedimensional model of the building, it can be recognize in light blue colours the external walls in rubble masonry and in red colour the walls in brick masonry.

The walls are made into discrete macro-elements, represented by deformable masonry piers and spandrel beams on the level.
Rigid nodes are indicated in the areas of the masonry that are typically less subject to earthquake damage, and connects the piers and spandrel beams (light blue colour).

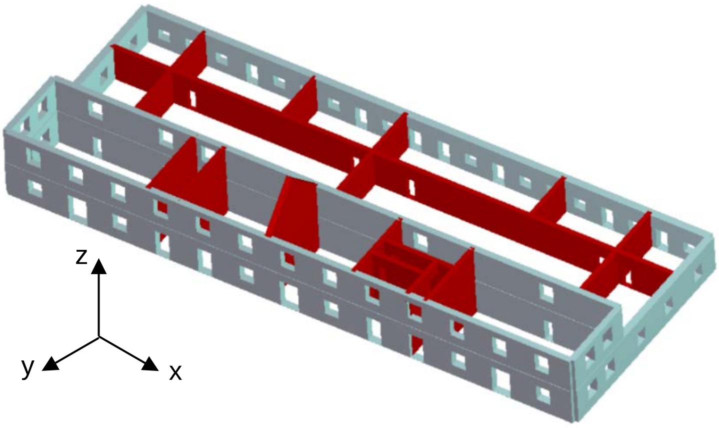


Figure 3-7 - Modelling elements of the building: the walls in 3MURI

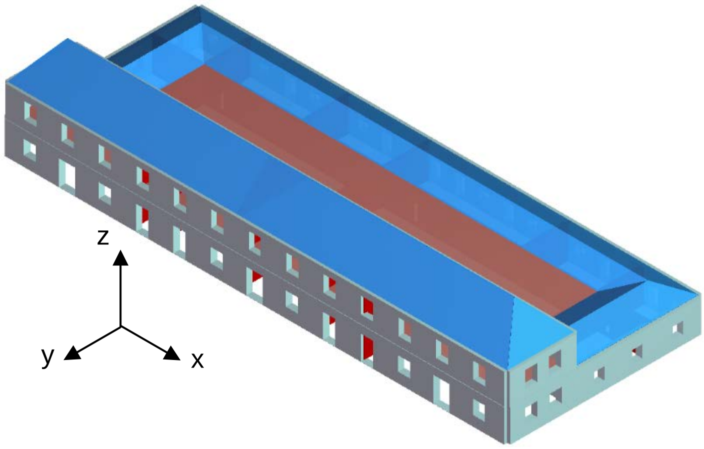


Figure 3-8 – 3D view of the building modelled in 3Muri

The piers and the spandrel beams are contiguous at the openings.
As can be seen in the Figures 3-9, 3-10 and 3-11, in orange are the macro-element piers; in green are the macro-element spandrels and in light blue are the parts of the façade where no damage is foreseen (rigid nodes).

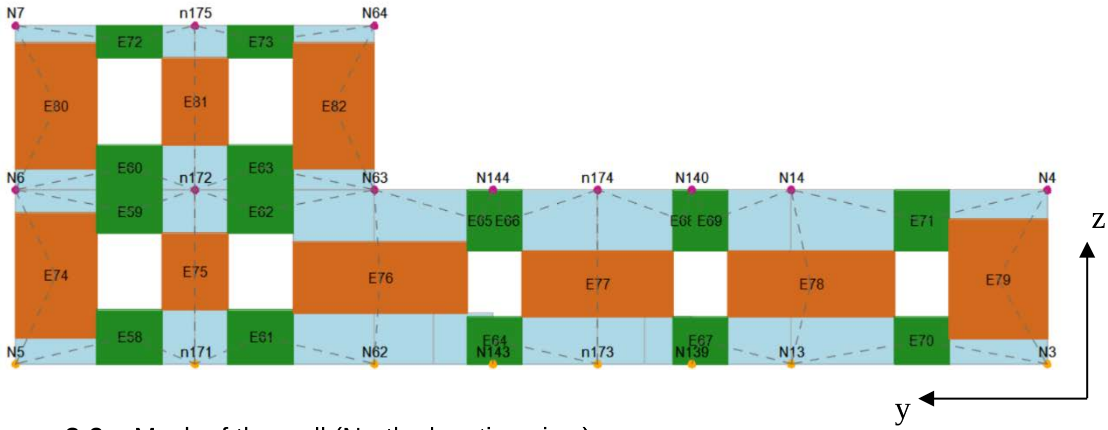


Figure 3-9 – Mesh of the wall (North elevation view)

As can be seen in the Figures 3-9, 3-10 and 3-11, in orange are the macro-element piers; in green are the macro-element spandrels and in light blue are the parts of the façade where no damage is foreseen (rigid nodes).

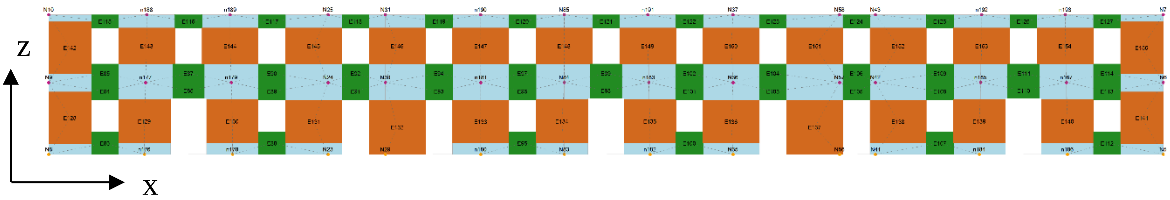


Figure 3-10 – Mesh of the wall (West elevation view)

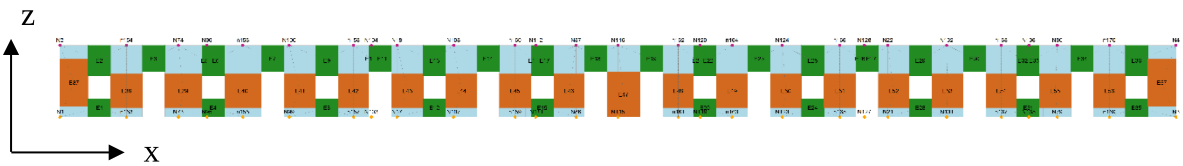


Figure 3-11 - Mesh of the wall (East elevation view)

The nodes of the model are three-dimensional, with five degrees of liberty (three displacement components in the overall reference system and the rotation around the X and Y-axes).

Alternatively, they are two-dimensional nodes with three degrees of liberty (two transfers and the rotation of the level of the wall).

The three-dimensional nodes are used to allow transfer of the actions from one wall to a second wall, which is located transversally to the first. While the two-dimensional nodes only have degrees of liberty on the level where the wall is found, allowing transfer of the force states between the various points of the wall.

The horizontal structures are modelled with the three-node floor elements connected to three-dimensional nodes. They can be loaded perpendicularly to their level using accidental or permanent loads. Seismic actions load the floor along the direction of the level. For this reason, the floor finite element is defined with axial rigidity, but without bending rigidity.

3.2. PUSH-OVER ANALYSIS WITH THE N2-METHOD

This seismic analysis technique, developed at the University of Ljubljana[19] and implemented in the European standard Eurocode 8[20], is based on the pushover analysis (non-linear static analysis) of a multi-degree-of-freedom model (MDOF) and the response spectrum analysis of an equivalent single-degree-of-freedom system (SDOF). The determination of the curve relative to the equivalent system allows determination of the period in which the maximum displacement requested by the earthquake to be found. The pushover analysis is carried out under conditions of constant gravity loads and monotonically increasing horizontal loads [Eurocode 8 - p. 4.3.3.4.2.].
It was performed in 3Muri software to assess the structural performance of this existing building and to estimate the expected plastic mechanisms and the distribution of damage.

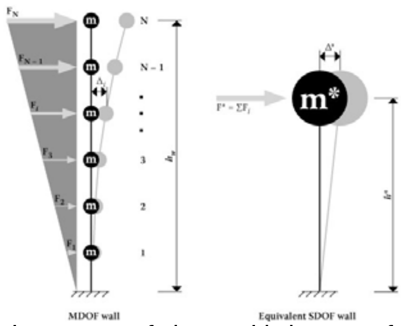
As can be seen in the Eurocode 8, there are two types of load conditions that must be examined:

- Distribution of forces proportional to the masses:

$$F_i = \frac{m_i}{\sum_i m_i}$$

- Distribution of forces proportional to the product of the masses for the deformation corresponding to the first vibration mode.

The phases of the N2-Method are the following:



1. Definition of the capacity curves of the multi-degree-of-freedom model (MDOF);
2. Definition of the equivalent single-degree-of-freedom system (SDOF);
3. Calculation of the capacity displacement, or the supposed ultimate displacement d_u, derived from the capacity curves;
4. Calculation of the displacement demand, or the target displacement of the multi-degree-of-freedom model (MDOF) (d_t)
5. Final step is to compare the presumed ultimate displacement, d_u and the target displacement d_t; It is required that $d_u \geq d_t$

[19] P. Fajfar, Seismic assessment of structures by a practise-oriented method, University of Ljubljana, Faculty of Civil and Geodetic Engineering, Ljubljana, SLOVENIA

[20] Eurocode 8 standard (Annex B of Part 1) (CEN, 2004a)

3.2.1. THE CAPACITY CURVES

The capacity curves or the base shear – displacement relations, can be drawn monitoring the displacement of the control node or a node chosen in the upper level of the structure considered according to the Eurocode 8

A comparison between the capacity curves, identified through a diagram showing maximum displacement base-shear, found for the various prescribed conditions with the displacement request required by the code.

The value for the maximum base displacement of the building generated by the distribution of forces is calculated and this constitutes the ultimate value for the building.

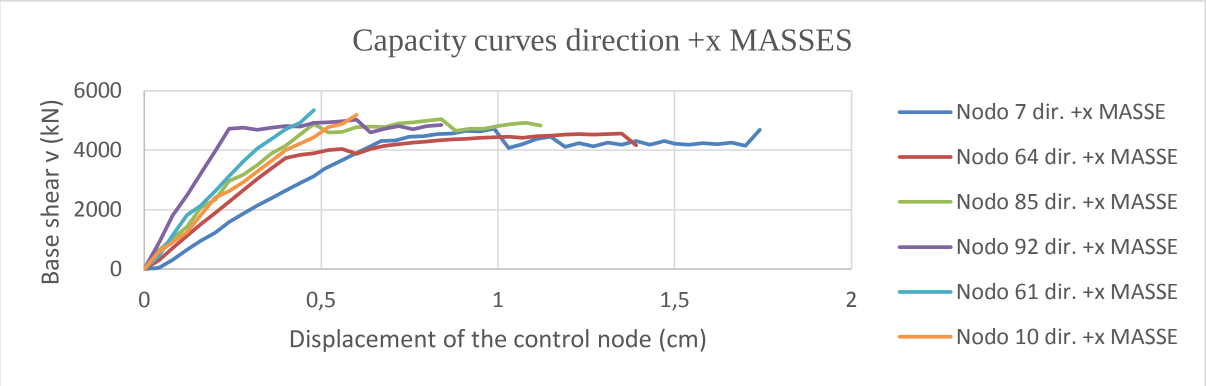


Figure 3 - 12 – The choice of the control node of the building

The displacement examined to trace the capacity curve is the point of the building called control node, in this case, the **node 7** on the level 2, which corresponds to the node that has the most relevant capacity curve, compared to the nodes present at level 2.

In order to identify the gravest seismic load condition, individual analyses were performed for load typology, seismic direction:

- The seismic load identifies which of the two distribution typologies will be examined. (proportional to the mass or first mode);
- The seismic direction identifies the direction in which the structure is loaded (+x or -y of the overall system) by the seismic load;

The selection of an appropriate lateral load distribution is an important step within the pushover analysis. In the proposed method, lateral loading, is applied independently in two horizontal directions, in each direction with + and – sign, related to the control node 7.

The presumed ultimate displacements (d_u) of the curves, related to the load distributions considered (+x and –y, Masses and 1° Mode) were redefined, according to the damage state regarding the last step of the curve, that it is shown in the 3Muri software, corresponding to the distorted view in elevation or in plan of the building's walls.

The related graphs are drawn below:

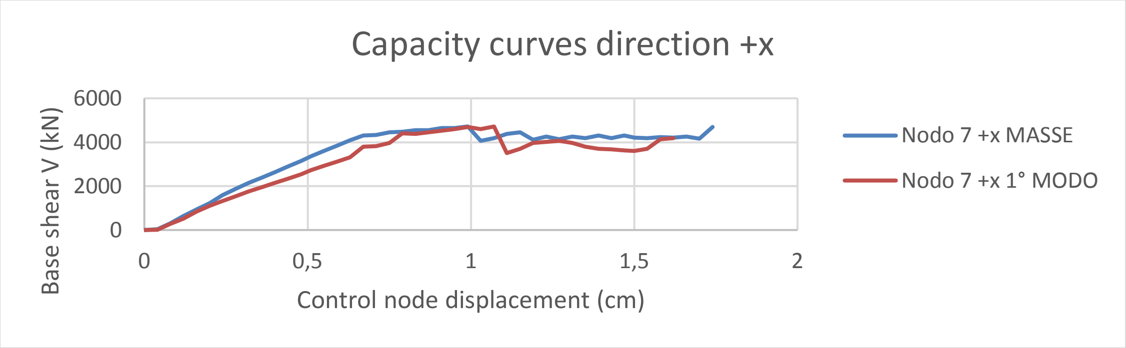


Figure 3-13 - Capacity curves Direction +x FIRST MODE and MASSES

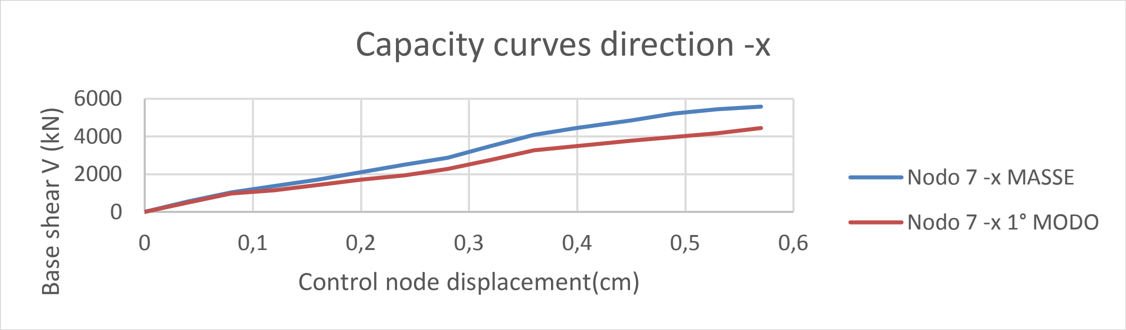


Figure 3-14 – Capacity curves Direction -x FIRST MODE and MASSES

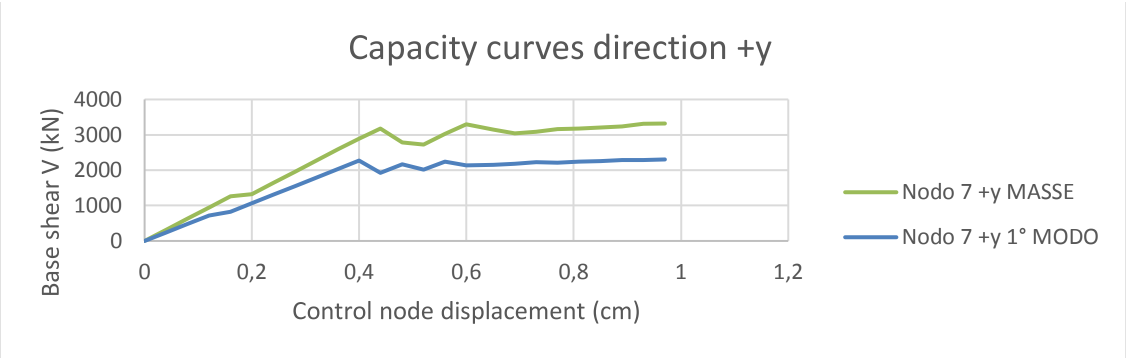


Figure 3-15 – Capacity curves Direction +y FIRST MODE and MASSES

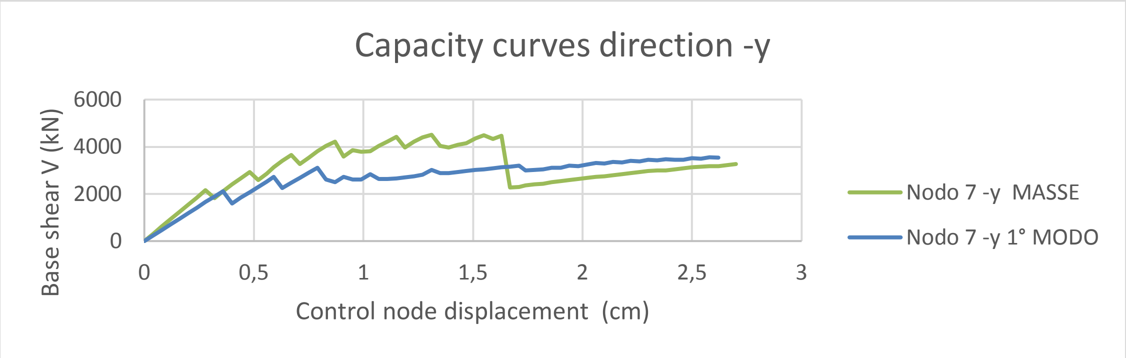


Figure 3-16 – Capacity curves Direction -y FIRST MODE and MASSES

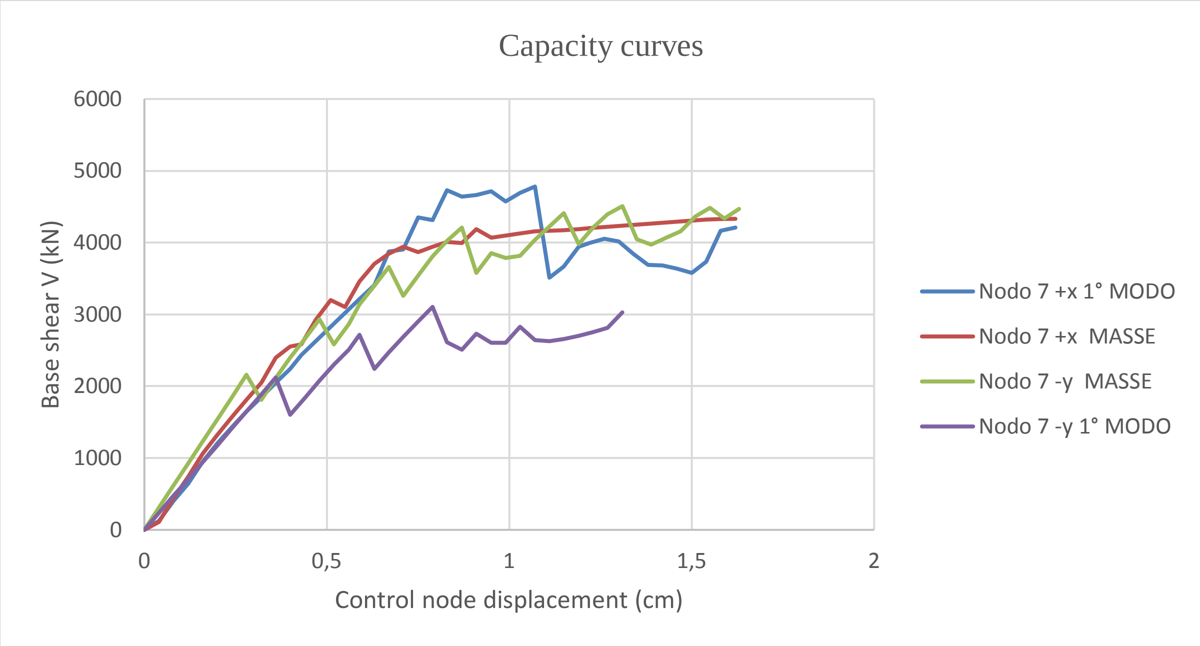


Figure 3 – 17 – Capacity curves progress, Directions +x and –y, distributions FIRST MODE and MASSES

The all capacity curves progress is reported in the Figure 3-17, to show the differences between the directions of the seismic action (x or y) and the distributions load (First mode and Masses).

3.2.2. DEFINITION OF THE EQUIVALENT SINGLE-DEGREE-OF-FREEDOM SYSTEM

It is necessary to transform the Multi Degree of Freedom (MDOF) system to an equivalent Single Degree of Freedom (SDOF) system, or bilinear idealization as are reported in the ANNEX B[21]. The parameters that have characterized the SDOF system are the equivalent mass m^*, the equivalent stiffness k^* and the equivalent period T^*.

The mass m^* is determined as:

$$m^* = \sum m_i * \Phi_i = \sum \overline{F_i} \tag{3.1}$$

where F_i is the normalized, lateral forces and Φ_i the normalized displacements, while where m_i is the mass in the i-th storey.

From the base shear force and the control node displacement of the MDOF system can be calculated the force F^* and displacement d^* of the equivalent SDOF system with the following expressions:

$$F^* = \frac{F_b}{\Gamma} \tag{3.2}$$

$$d^* = \frac{d_n}{\Gamma} \tag{3.3}$$

Where the transformation factor Γ is given by:

[21] ANNEX B Eurocode 8, B.2 Transformation to an equivalent Single Degree of Freedom (SDOF) system

$$\Gamma = \frac{m^*}{\sum m_i * \Phi_i^2} = \frac{\sum F_i}{\sum \frac{F_i^2}{m_i}} \tag{3.4}$$

The period *T** instead is determined by:

$$T^* = 2 * \Pi * \sqrt{\frac{m^* * d_y^*}{F_y^*}} \tag{3.5}$$

where F_y* is the yield force and d_y^* the yield displacement, related to the idealized elasto-perfectly plastic force – displacement relationship. F_y* is equal to the base shear force at the formation of the plastic mechanism.

The yield displacement d_y^* is given by:

$$d_y^* = 2 * \left(d_m^* - \frac{E_m^*}{F_y^*}\right) \tag{3.6}$$

where E_m^* is the actual deformation energy up to the formation of the plastic mechanism and d_m^* correspond with the displacement of the plastic mechanism.

These values are obtained from the spatial model developed in 3Muri software and subsequently verified. In the tables below are shown the values and a graphical representation of the bilinear curves for the two directions of the seismic action and for the two load distributions.

Table 3-4 - Direction +x First Mode

T* [s]	0,275	d^*_m [cm]	0,96
m* [kg]	1033193	$F_{bu}^*=F_{max}^*$ [kN]	2807,74
w [kg]	2890413,3	d_u^* [cm]	0,96
m*/w [%]	36	$d^*(0,70)$ [cm]	0,36
Γ [m/s²]	1,68	k^*	543043478,3
F^*_y [kN]	2498	$0,7*F_{bu}^*$ [kN]	1965,42
d^*_y [cm]	0,46	μ	2,096

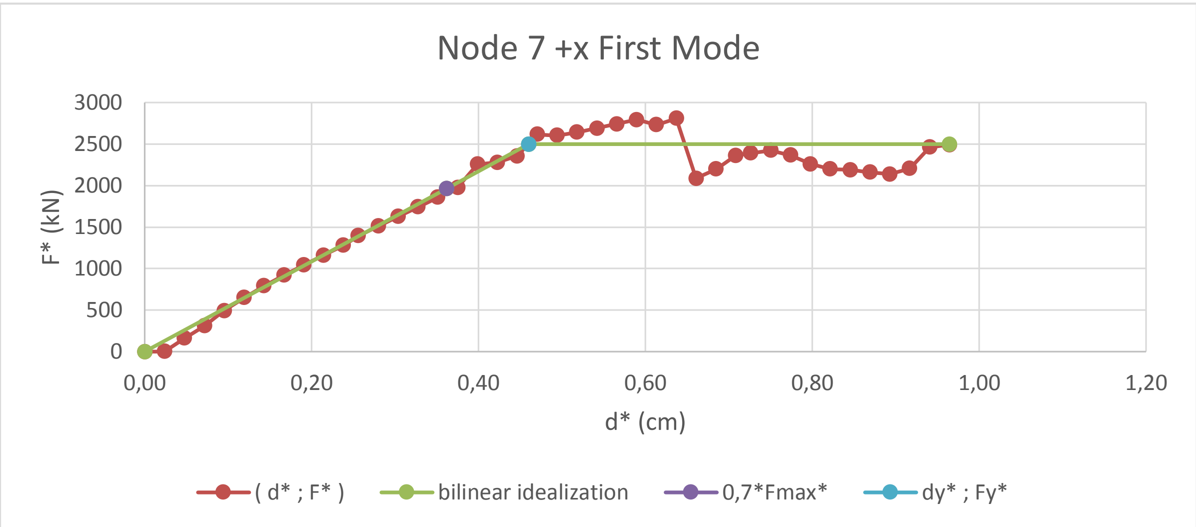


Figure 3-18 – Bilinear idealization of the capacity curve related to the Node 7, +x 1°Mode

Table 3-5 - Direction +x Masses

T* [s]	0,257	d^*_m [cm]	0,96
m* [kg]	1033192,98	$F_{bu}^*=F_{max}^*$ [kN]	2577,98
w [kg]	2890413,26	d_u^* [cm]	0,96
m*/w [%]	36	$d^*(0,70)$ [cm]	0,29
Γ [m/s²]	1,68	k^*	614750000
F^*_y [kN]	2459	$0,7{*}F_{bu}^*$ [kN]	1804,58
d^*_y [cm]	0,4	μ	2,411

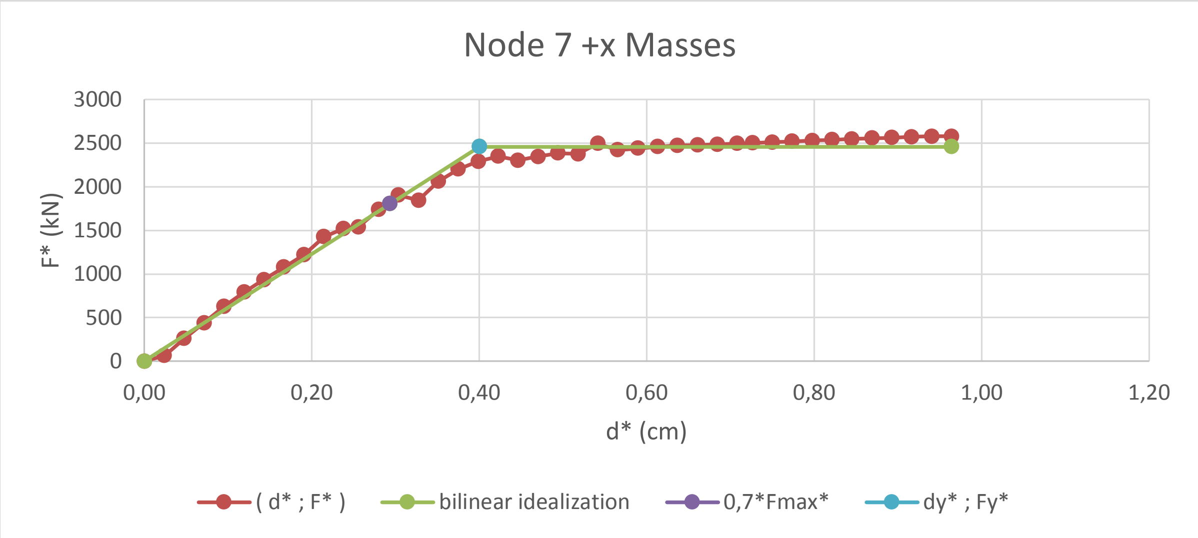


Figure 3-19 - Bilinear idealization of the capacity curve related to the Node 7, +x Masses

Table 3-6 – Direction –y Masses

T* [s]	0,247	d^*_m [cm]	0,83
m* [kg]	814932,028	$F_{bu}^*=F_{max}^*$ [kN]	2300
w [kg]	2890413,26	d_u^* [cm]	0,67
m*/w [%]	28	$d^*(0,70)$ [cm]	0,41
Γ [m/s²]	1,96	k^*	532439024,4
F^*_y [kN]	2183	$0,7{*}F_{bu}^*$ [kN]	1610,00
d^*_y [cm]	0,41	μ	1,630

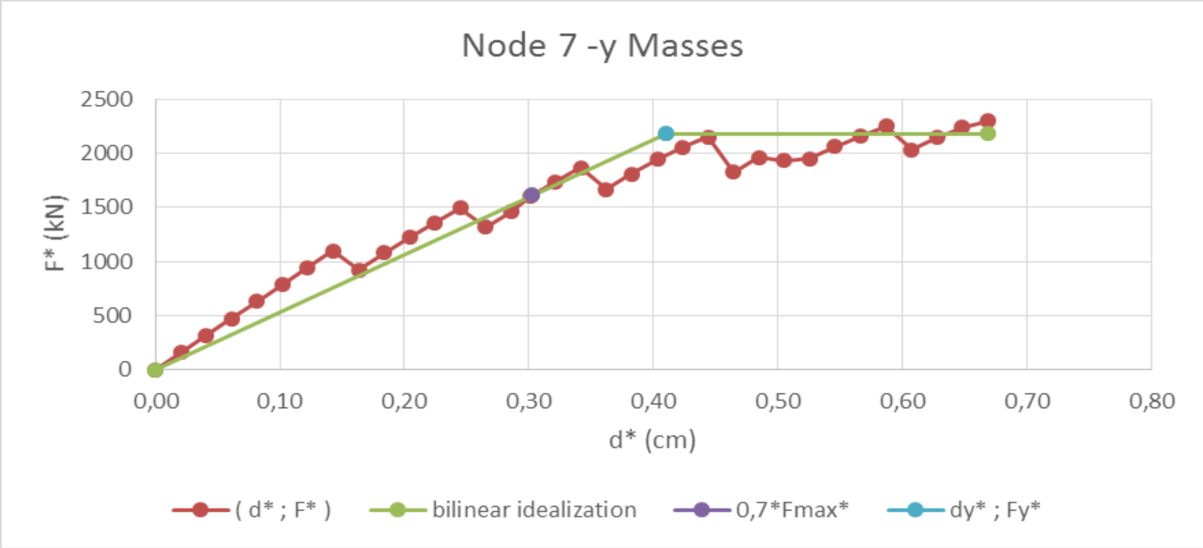


Figure 3-20 - Bilinear idealization of the capacity curve related to the Node 7, -y Masses

Table 3-7 – Direction –y First Mode

T* [s]	0,270	d^*_m [cm]	0,67
m* [kg]	814932,028	$F_{bu}^*=F_{max}^*$ [kN]	1812,76
w [kg]	2890413,26	d_u^* [cm]	0,83
m*/w [%]	28	$d^*(0,70)$ [cm]	0,29
Γ [m/s^2]	1,96	k^*	440303030,3
F^*_y [kN]	1453	$0,7 \cdot F_{bu}^*$ [kN]	1268,93
d^*_y [cm]	0,33	μ	2,520

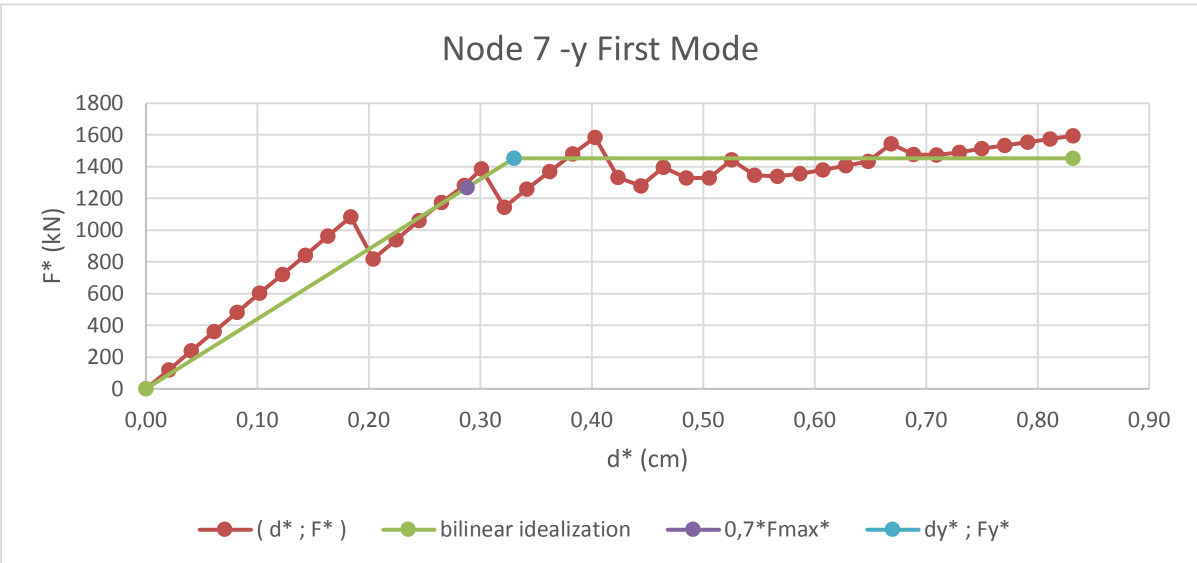


Figure 3-21 - Bilinear idealization of the capacity curve related to the Node 7, -y, 1°Mode

3.3. DETERMINATION OF THE TARGET DISPLACEMENT

According to the guidelines to determinate the target displacement for the pushover analysis, included in the **ANNEX B**[22]**,** the fundamental points are reported below:

1. determination of the target displacement for the equivalent SDOF system;
2. determination of the target displacement for the MDOF system;

To evaluate the target displacement for the equivalent SDOF system it is necessary to define the seismic action by the elastic horizontal elastic spectrum S_e(T) and elastic displacement response spectrum S_{De}(T), presented in follow paragraph (3.3.1).

3.3.1. DEFINITION OF THE SEISMIC ACTION

The seismic demand is usually defined by the elastic horizontal elastic spectrum S_e (T) and elastic displacement response spectrum S_{De}(T).

According to the Eurocode 8 p. 3.2.2.2, the expressions to define the horizontal elastic response spectrum S_e (T) are shown below:

$$0 \leq T \leq T_B : \qquad S_e(T) = a_g * S * \left[1 + \frac{T}{T_B} * (\eta * 2{,}5 - 1)\right] \tag{3.7}$$

$$T_B \leq T \leq T_C : \qquad S_e(T) = a_g * S * \eta * 2{,}5 \tag{3.8}$$

$$T_C \leq T \leq T_D : \qquad S_e(T) = a_g * S * \eta * 2{,}5 * \left[\frac{T_C}{T}\right] \tag{3.9}$$

$$T_D \leq T \leq 4s : \qquad S_e(T) = a_g * S * \eta * 2{,}5 * \left[\frac{T_C * T_D}{T^2}\right] \tag{3.10}$$

Where:

- **S_e(T)** is the elastic response spectrum;
- **T** is the vibration period of a linear single-degree-of-freedom system;
- **a_g** is the design ground acceleration on type A ground ($a_g = \gamma_I * a_{gR}$);
- **T_B** is the lower limit of the period of the constant spectral acceleration branch;
- **T_C** is the upper limit of the period of the constant spectral acceleration branch;
- **T_D** 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

[22] ANNEX B - DETERMINATION OF THE TARGET DISPLACEMENT FOR NONLINEAR STATIC (PUSHOVER) ANALYSIS from EUROCODE 8

The values of the periods $\mathbf{T_B}$, $\mathbf{T_C}$ and $\mathbf{T_D}$ and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type and the seismic zone that depend on where our structure is located.

A ground type B was chosen since it is the most similar situation corresponding to the downtown VRSA soil type.

If deep geology is not accounted for, the recommended choice is the use of two types of spectra: Type 1 (corresponding to a scenario of faraway earthquake) and Type 2 (corresponding to a scenario of nearby earthquake).

The parameters of the reference peak ground acceleration on type A ground, $\mathbf{a_{gR}}$**,** is included in ANEXO NA.I (NP EN 1998-1 2009 p. 228):

Table 3-8 - Parameters of the reference peak ground acceleration on type A ground, a_{gR}

Site	**Seismic action**		
	Seismic zone		$\mathbf{a_{gR}}\left[\frac{m}{s^2}\right]$
Vila Real de Santo António	TYPE 1	1.3	1,5
	TYPE 2	2.3	1,7

An importance factor γ_I equal to 1,0 is assigned to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}). The design ground acceleration on type A ground a_g is equal to a_{gR} times the importance factor γ_I ($a_g = \gamma_I * a_{gR}$).

The configurations of the response spectra for the two types of seismic action are defined from the parameters in the Anexo Nacional NA NP EN 1998-1 2009 (Quadro NA−3.2 for the Type 1, and Quadro NA−3.3 for the Type 2 and is shown in the following table:

Table 3-9 - Parameters of the elastic response spectrum for seismic action Type 1 and 2

	Ground type	$\mathbf{S_{max}}$	**S**	$\mathbf{T_B}$ **(s)**	$\mathbf{T_C}$ **(s)**	$\mathbf{T_D}$ **(s)**
TYPE 1	B	1,35	1,29	0,1	0,6	2,0
TYPE 2	B	1,35	1,27	0,1	0,25	2,0

The value of S is determined according to f) NA–3.2.2.2(2)P where in this case, the value of a_g relatively for the Type 1 is 1,5 while for the Type 2 is 1,7

$$1 < a_g < 4\frac{m}{s^2} \qquad\qquad S = S_{max} - \frac{S_{max} - 1}{3} \qquad (3.11)$$

In the following Figure 3-22 can be seen the corresponding horizontal elastic response spectrum $S_e(T)$ for Vila Real de Santo António for the two types of seismic action.

Table 3-10 - Respective values of the $S_e(T)$

Type 1			Type 2		
Period T (s)		Se (T)	Period T (s)		Se (T)
T_B (s)	0,1	4,844	T_B (s)	0,1	5,390
T_C (s)	0,6	4,844	T_C (s)	0,25	5,390
T_D (s)	2,0	1,453	T_D (s)	2,0	0,674
T (4s)	4,0	0,363	T (4s)	4,0	0,168

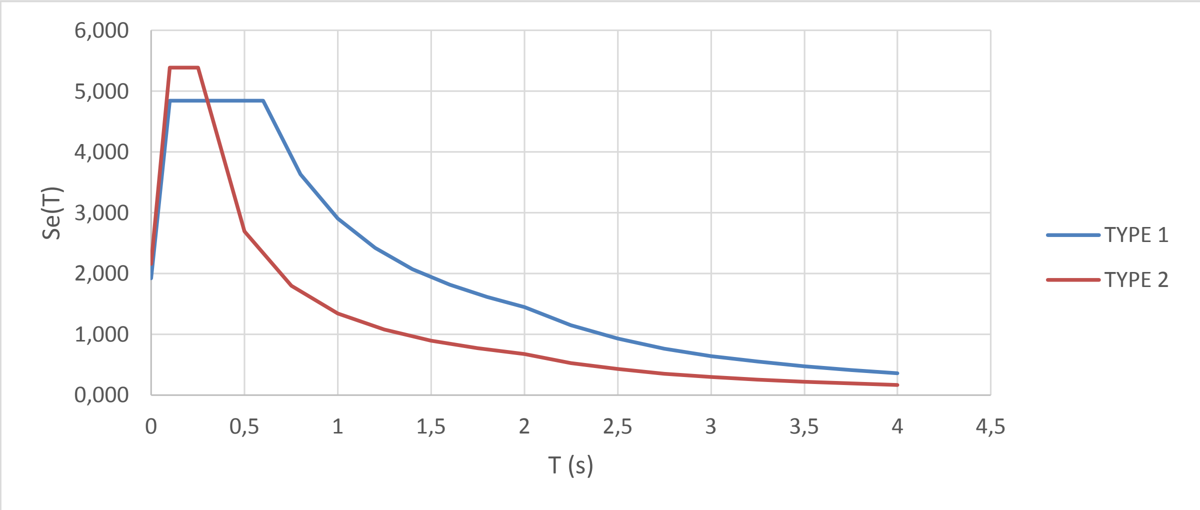


Figure 3-22 – Horizontal Elastic Response Spectrum $S_e(T)$ for VRSA

The elastic displacement response spectrum, ***$S_{De}(T)$***, shall be obtained by direct transformation of the elastic acceleration response spectrum, ***$S_e(T)$***, using the following expression:

$$S_{De}(T) = S_e(T) * \left[\frac{T}{2 * \Pi}\right]^2 \qquad (3.12)$$

Expression (3.7) should normally be applied for vibration periods not exceeding 4,0 s.

The respective value of $S_{De}(T)$ is shown in the following table:

Table 3-11 – Values needed to draw the elastic displacement response spectrum $S_{De}(T)$

Type 1			Type 2		
Period T (s)		S_{De} (T)	Period T (s)		S_{De} (T)
T_B (s)	0,1	0,001227	T_B (s)	0,1	0,001365
T_C (s)	0,6	0,044172	T_C (s)	0,25	0,049151
T_D (s)	2,0	0,14722	T_D (s)	2,0	0,06829
T (4s)	4,0	0,147118	T (4s)	4,0	0,068088

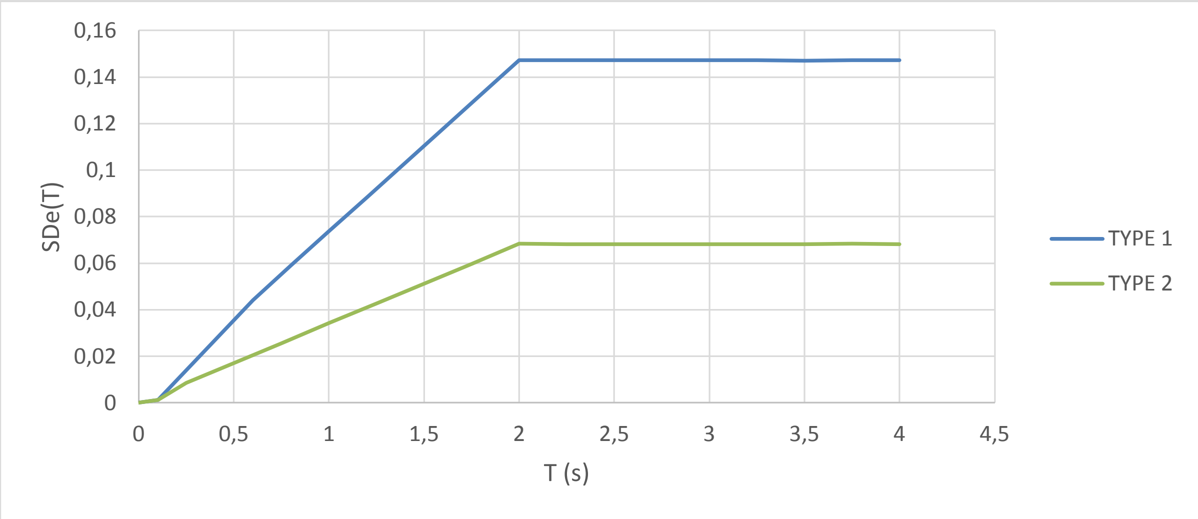


Figure 3-23 - Elastic displacement response spectrum S_{De} (T) for VRSA

3.3.2. DEFINITION OF THE SEISMIC DEMAND IN AD FORMAT

This step is only to have a direct visual interpretation of the procedure. Starting from the usual elastic acceleration spectrum S_e(T), the inelastic spectra in acceleration – displacement (AD) format, for the constant ductility factors μ, can be determined. The following expressions can be used:
For an elastic SDOF system:

$$S_{De}(T) = S_e(T) * \left[\frac{T}{2 * \Pi}\right]^2 \tag{3.13}$$

For an inelastic SDOF system with a bilinear force – deformation relationship, the acceleration spectrum (S_a) and the displacement spectrum (S_d) can be determined as:

$$S_a = \left[\frac{S_{ae}}{R_\mu}\right] \tag{3.14}$$

$$S_d = \mu * \left[\frac{S_{de}}{R_\mu}\right] \tag{3.15}$$

where μ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement, and R_μ is the reduction factor due to ductility, i.e., due to the hysteretic energy dissipation of ductile structures. In the simple version of the N2-Method, we will make use of a bilinear spectrum for the reduction factor R_μ:

$$R_\mu = (\mu - 1) * \frac{T}{T_C} + 1 \qquad T < T_C \tag{3.16}$$

$$R_\mu = \mu \qquad T \geq T_C \tag{3.17}$$

where T_C is the characteristic period of the ground motion.

Using the values of the ultimate displacement d_u and the yield displacement d_y, derived from the pushover analyses, performed in 3Muri, can be calculated the values of μ and R_μ for Type 1 and Type 2 seismic actions considered.

The values of the d_u and d_y to calculate μ and R_μ are presented in the Table 3-12:

Table 3-12 – Values of μ and Rμ for the Type 1 and Type 2 of earthquake

Distribution	T*	μ	T_C Type 1	R_μ Type 1	T_C Type 2	R_μ Type 2
+x 1°Mode	0,275	2,096	0,6	1,502	0,25	2,096
+x Masses	0,257	2,411	0,6	1,604	0,25	2,411
-y 1°Mode	0,270	2,520	0,6	1,684	0,25	2,520
-y Masses	0,247	1,630	0,6	1,259	0,25	1,623

One graph concerning the analysis conducted considering the distribution of forces propotional to the First vibration Mode in x-Direction is drawn below:

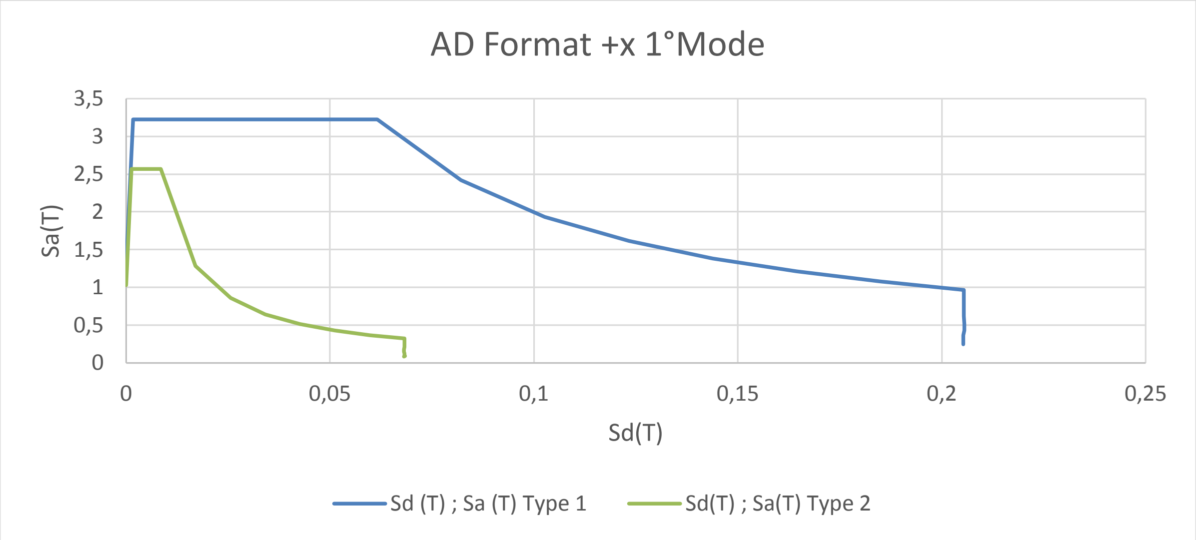


Figure 3-24 – Inelastic spectrum corresponding to T*=0,275; μ=2.096 (+x FIRST MODE)

3.3.3. DETERMINATION OF THE TARGET DISPLACEMENT FOR THE EQUIVALENT SDOF SYSTEM

Once the period of the idealized equivalent SDOF system was determined, i.e. T* for each analyses, the target displacement of the structure is given by:

$$d_{et}^* = S_e(T^*) * \left[\frac{T^*}{2 * \Pi}\right]^2 \tag{3.18}$$

where $S_e(T^*)$ is the elastic acceleration response spectrum at the period T^*.

According to the steps described in the Annex B, it is necessary to use different expressions to determinate the target displacement d_t^*, depending on if it deals with structures in the short-period range and structures in the medium and long-period ranges. The corner period between the short and medium period range is T_C and the procedure used is indicated below:

- Short period range with $T^* < T_C$

If $F_y^* / m^* \geq S_e(T^*)$, the response is elastic and thus

$$d_t^* = d_{et}^* \tag{3.19}$$

If $F_y^* / m^* < S_e(T^*)$, the response is nonlinear and

$$d_t^* = \frac{d_{et}^*}{q_u} * \left(1 + (q_u - 1) * \frac{T_C}{T^*}\right) \geq d_{et}^* \tag{3.20}$$

where q_u is the ratio between the acceleration in the structure with unlimited elastic behaviour $S_e(T^*)$ and in the structure with limited strength F_y^* / m^*.

$$q_u = \frac{S_e(T^*) * m^*}{F_y^*} \tag{3.21}$$

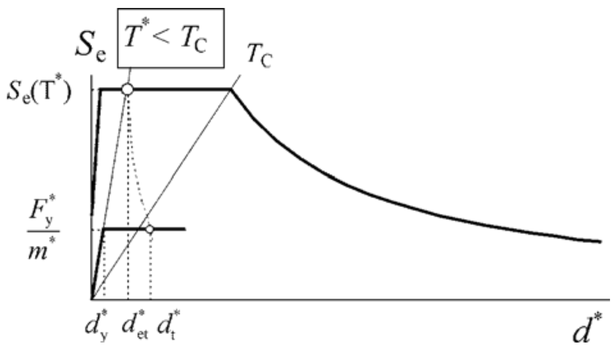


Figure 3-25 – Determination of the target displacement for the SDOF system in a short period range from the Annex B

- Medium and long period range with $T^* \geq T_C$

$$d_t^* = d_{et}^* \tag{3.22}$$

d_t^* need not exceed 3 d_{et}^*.

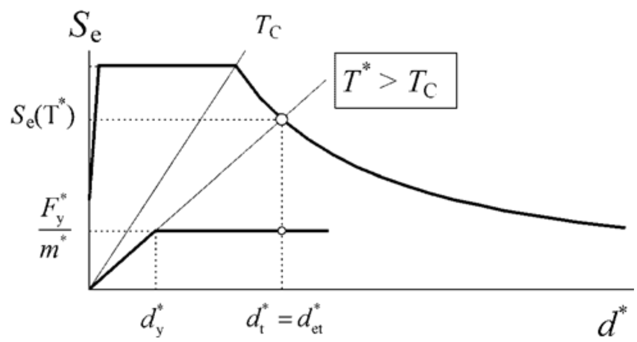


Figure 3-26 – Determination of the target displacement for the SDOF system in a medium and long period range from the Annex B

The figures 3-25 and 3-26 are plotted in acceleration - displacement format. Period T^* is represented by the radial line from the origin of the coordinate system to the point at the elastic response spectrum defined by coordinates $d^* = S_e(T^*)*(T^*/2\pi)^2$ and $S_e(T^*)$.

The values of d_{et}^* and $S_e(T^*)$ are written in the following table:

Table 3-13 – Values of d_{et}^* and $S_e(T^*)$ related to the Type 1 and Type 2 of the earthquake

Distribution	T^*(s)	T_C (s) Type 1	$S_e(T^*)$ Type 1	d_{et}^*(cm) Type 1	T_C (s) Type 2	$S_e(T^*)$ Type 2	d_{et}^* (cm) Type 2
+x 1°Mode	0,275	0,6	4,84	0,93	0,25	4,90	0,94
+x Masses	0,257	0,6	4,84	0,81	0,25	5,24	0,88
-y 1°Mode	0,270	0,6	4,84	0,89	0,25	5,39	0,92
-y Masses	0,247	0,6	4,84	0,75	0,25	4,99	0,83

- Regarding the analysis +x First Mode, with T*=0,275 s < T_c (0,6 s) relatively to Type 1 seismic action, that characterised the short period range, and $\frac{F_y^*}{m^*} = 2{,}41 < S_e(T^*)$, therefore the response is nonlinear and d_t^* can be calculated with the following expression:

$$d_t^* = \frac{d_{et}^*}{q_u} * \left(1 + (q_u - 1) * \frac{T_C}{T^*}\right) \geq d_{et}^* \tag{3.23}$$

where

$$q_u = \frac{S_e(T^*) * m^*}{F_y^*} = 2{,}003 \tag{3.24}$$

The found value of d_t^* is 1,48 cm.

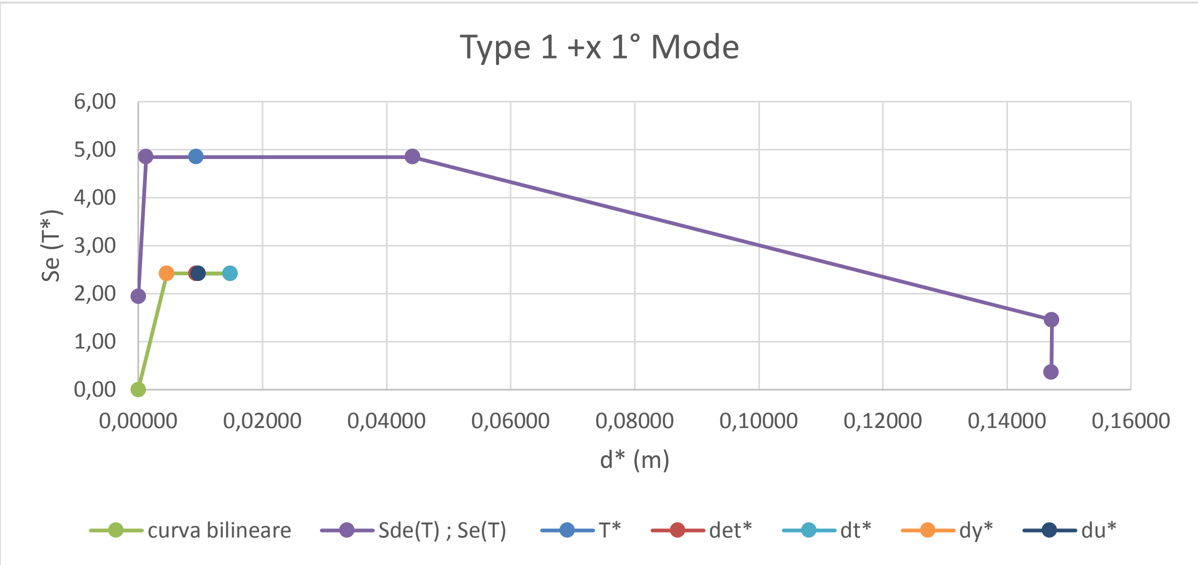


Figure 3-27 (a) – Graphic determination of d_{et}* and d_t* for the Type 1, direction +x, 1°Mode

As can be seen in the Figure 3-27 (b), about the Type 2 seismic action, given that T*=0,275 s > T_c (0,25 s), it is necessary to consider the procedure related the medium and long period range (Figure 3-29) and the target displacement can be calculated according to the expression B.12,

$d_t^* = d_{et}^*$ = 0,94 cm and the graph is the following:

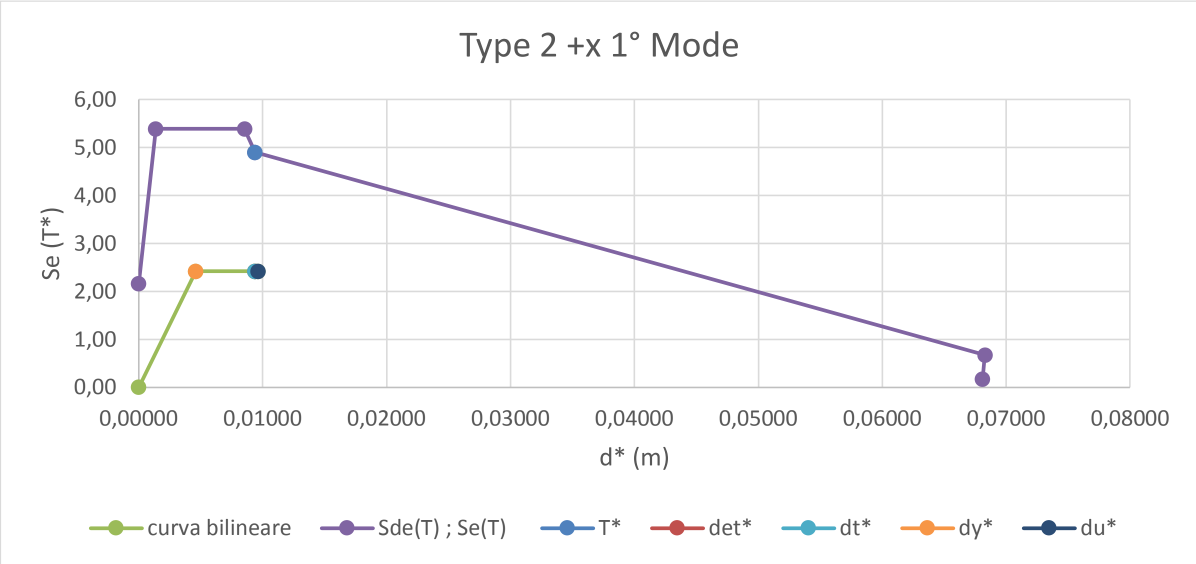


Figure 3-27 (b) – Graphic determination of d_{et}* and d_t* for the Type 2, direction +x, 1°Mode

- For the analysis +x Masses, with T*=0,257 s < T_c (0,6 s) relatively to Type 1 seismic action (Figure 3-28 (a)), that characterised the short period range, and $\frac{F_y^*}{m^*} = 2{,}38 < S_e(T^*)$, therefore the response is nonlinear and d_t^* is equal to 1,36 cm, with the factor q_u = 2,035.

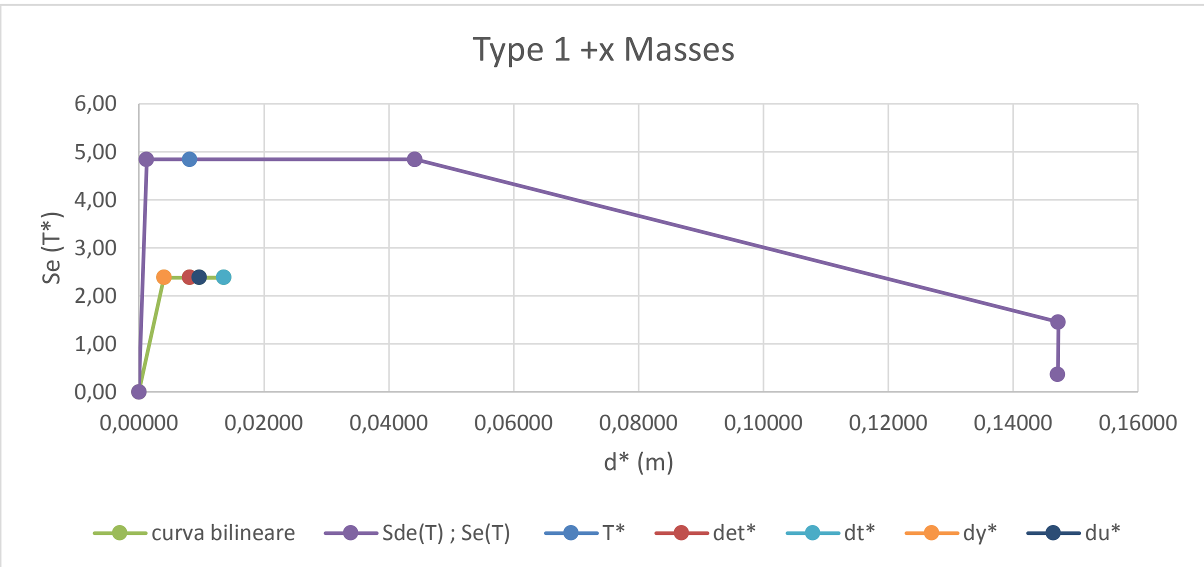


Figure 3-28 (a) - Graphic determination of d_{et}* and d_t* for the Type 1, direction +x, Masses

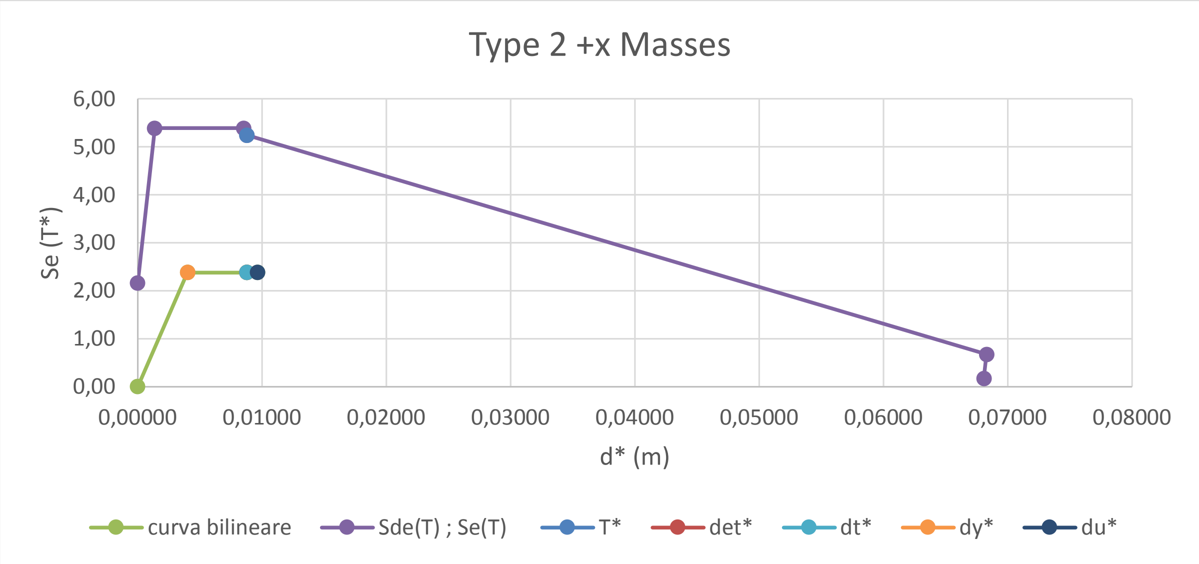


Figure 3-28 (b) - Graphic determination of d_{et}* and d_t* for the Type 2, direction +x, Masses

About the Type 2 analysis (Figure 3-28 (b)), there is T*=0,257 s > T_c (0,25 s), that characterised the medium and long period range, therefore according to the expression B.12, the correspondent $d_t^* = d_{et}^*$ = 0,88 cm.

- Regarding the analysis -y First Mode, with T*=0,270 s < T_c (0,6 s) relatively to Type 1 (Figure 3-29 (a)), that characterised the short period range, and $\frac{F_y^*}{m^*} = 1{,}78 < S_e(T^*)$, therefore the response is nonlinear and d_t^* can be calculated with the expression B.10: considering that the q_u factor is equal to 2,717, the target displacement d_t^* = 1,58cm

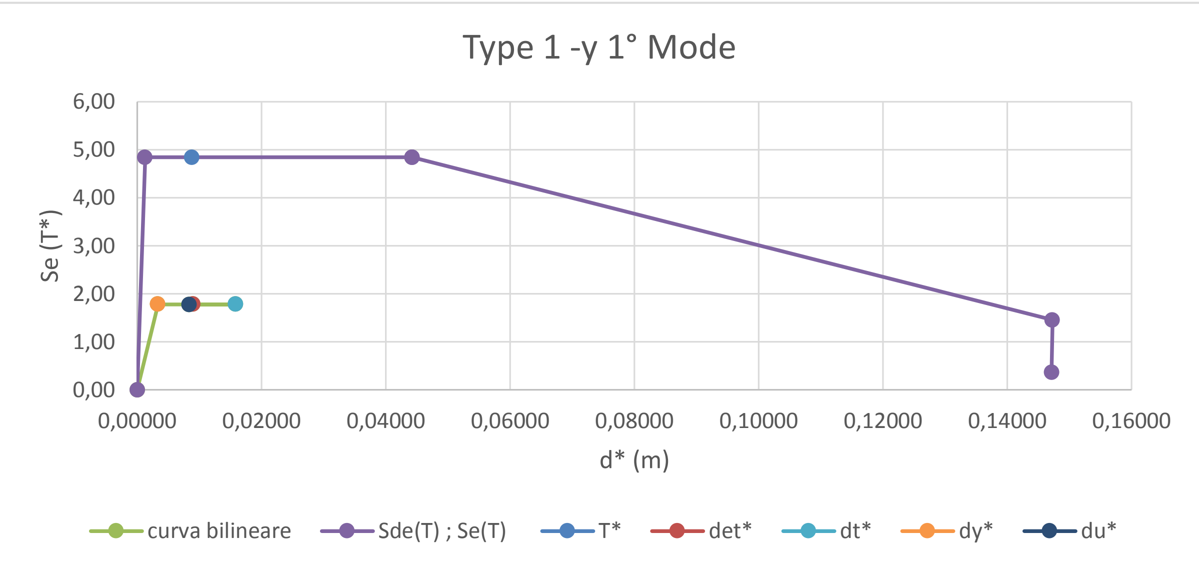


Figure 3-29 (a) - Graphic determination of d_{et}* and d_t* for the Type 1, direction -y, 1° Mode

Considering instead the Type 2 of earthquake (Figure 3-29 (b)), T*=0,270 s > T_c (0,25 s), the found target displacement $d_t^* = d_{et}^*$ is equal to 0,94 cm according to the expression B.12, that characterised the medium and long period range.

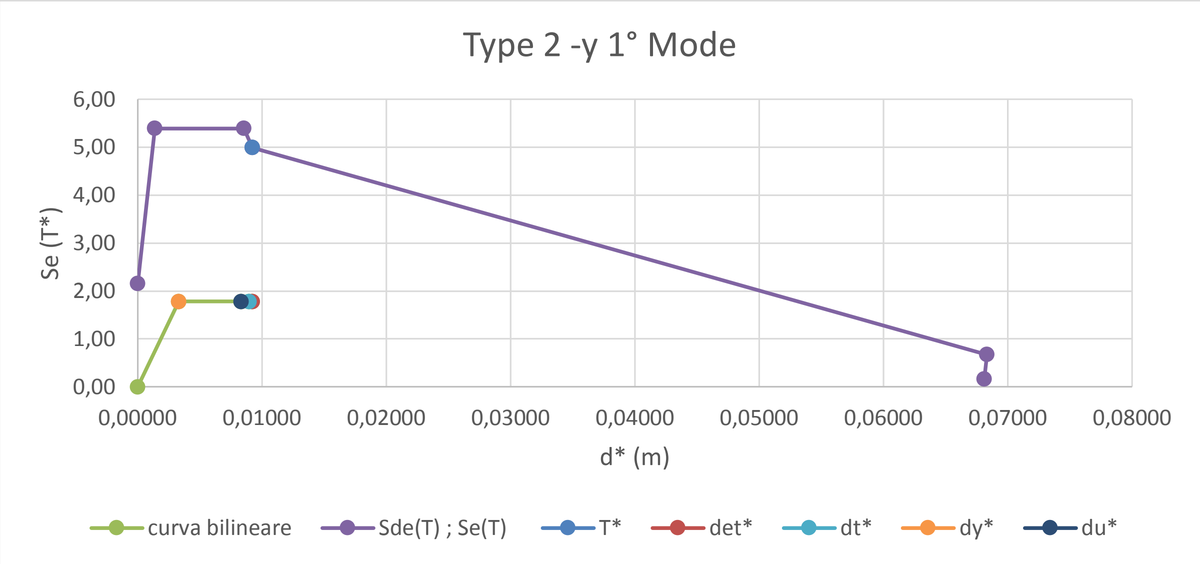


Figure 3-29 (b) - Graphic determination of d_{et}* and d_t* for the Type 2, direction -y, 1° Mode

- The last analysis carried out is the –y Masses, where it can be seen that for the Type 1 seismic action (Figure 3-30 (a), given that T*=0,247 s < T_c (0,6 s) the short period range procedure can be used. The response is nonlinear since $\frac{F_y^*}{m^*} = 2{,}68 < S_e(T^*)$, so the target displacement dt* can be determined, according to the equation 3.20. The values found are: q_u factor = 1,81 and d_t^* is equal to 1,23 cm.

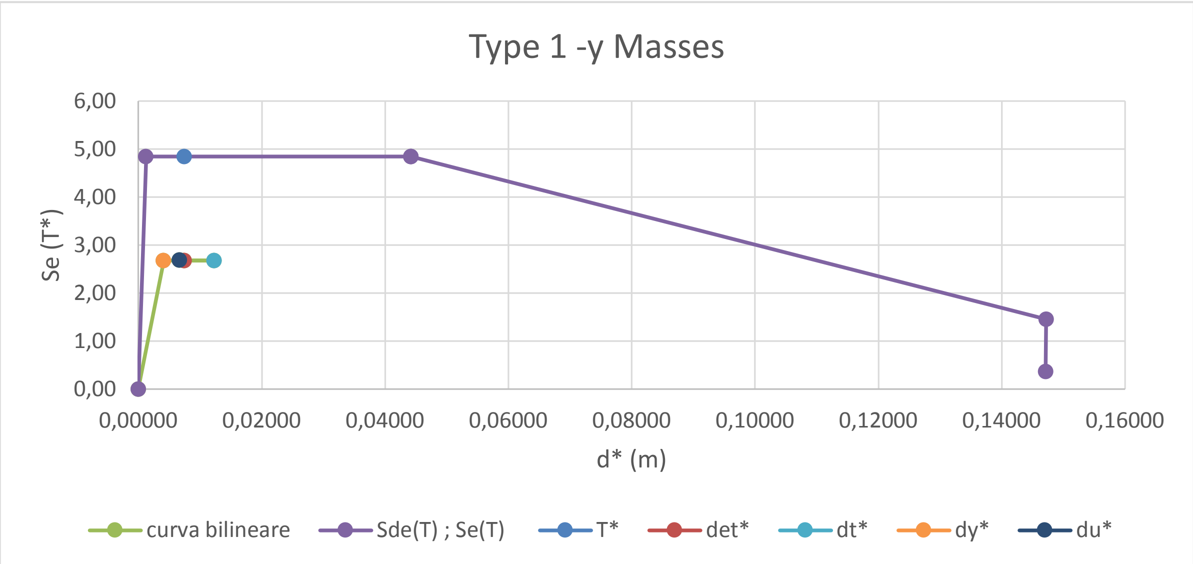


Figure 3-30 (a) - Graphic determination of d_{et}* and d_t* for the Type 1, direction -y, Masses

About the analysis, considering the Type 2 seismic action (Figure 3-30 (b)), also the procedure of the short period range was used, given that the period of the equivalent SDOF system T*= 0,247 s $<$ T_c (0,25 s). The value of $\frac{F_y^*}{m^*} = 2{,}68 < S_e(T^*)$, that characterize a nonlinear response. Therefore, the value of the target displacement d_t^* = 0,84 cm, corresponding to the q_u factor equal to 2,01 was determined, according to the equation 3.20.

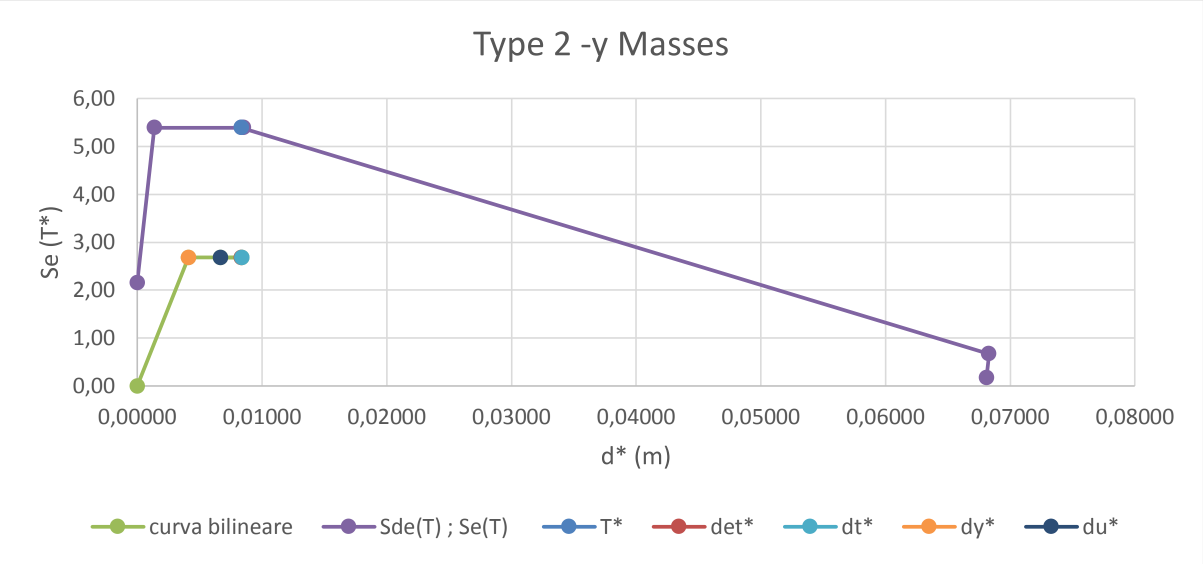


Figure 3-30 (b) - Graphic determination of d_{et}* and d_t* for the Type 2, direction +x, Masses

These graphs have been drawn for all the situations (earthquake Type 1 for +x and -y directions and earthquake Type 2 for +x and -y directions).

3.3.4. DETERMINATION OF THE TARGET DISPLACEMENT FOR THE MDOF SYSTEM

The expression in 3.25 in used to calculate the related target displacement of the MDOF system, which corresponds to the control node, derived from the target displacement for the equivalent SDOF system, d_t^*, is given by:

$$d_t = \Gamma * d_t^* \tag{3.25}$$

In the table 3-14 have been reported the values of the determinate target displacement about each case of direction of the earthquake (+x and –y) related to the Type 1.

Table 3-14 – Values of d_t for the Type 1 of earthquake

Distribution	T^*(s)	T_C (s) Type 1	$S_e(T^*)$ Type 1	d_{et}^*(cm) Type 1	d_t^* (cm) Type 1	Γ	d_t (cm) Type 1
+x 1°Mode	0,275	0,6	4,84	0,93	1,48	1,68	2,48
+x Masses	0,257	0,6	4,84	0,81	1,36	1,68	2,28
-y 1°Mode	0,270	0,6	4,84	0,89	1,58	1,96	3,1
-y Masses	0,247	0,6	4,84	0,75	1,23	1,96	2,4

Instead, the Table 3-15 shows the values related for the Type 2 of earthquake:

Table 3-15 – Values of d_t related to the Type 2 of the earthquake

Distribution	T^*(s)	T_C (s) Type 2	$S_e(T^*)$ Type 2	d_{et}^*(cm) Type 2	d_t^*(cm) Type 2	Γ	d_t (cm) Type 2
+x 1°Mode	0,275	0,25	4,90	0,94	0,94	1,68	1,58
+x Masses	0,257	0,25	5,24	0,88	0,88	1,68	1,47
-y 1°Mode	0,270	0,25	5,39	0,92	0,94	1,96	1,75
-y Masses	0,247	0,25	4,99	0,83	0,84	1,96	1,64

4. CONCLUSIONS AND FUTURE DEVELOPMENTS

The values of the target displacement d_t and the ultimate displacement d_u are compared to each other, and if $d_u/d_t > 1$ the N2-Method is verified according to Eurocode 8

Table 4-1 – Verifications' Results of the Pombalino case study

Distribution	d_t (cm) Type 1	d_u (cm) Type 1	Verification Type 1 $d_u/d_t > 1$	d_t (cm) Type 2	d_u (cm) Type 2	Verification Type 2 $d_u/d_t > 1$
+x 1°Mode	2,48	1,62	No	1,58	1,62	Yes
+x Masses	2,28	1,62	No	1,47	1,62	Yes
-y 1°Mode	3,1	1,63	No	1,75	1,63	No
-y Masses	2,4	1,31	No	1,64	1,31	No

Lisbon is situated in the Portugal Continental zonation; therefore, it is necessary, according to the Anexo Nacional NP EN1998-1 2009, to consider the two Types of seismic action. The Type 1 spectral shape refers to high and moderate seismicity regions, characterized by larger magnitudes (M≥5.5), while the Type 2 is suitable for earthquakes up size M5.5, defined as smaller magnitudes.

How can be seen in the Table 4-1, the analyses in +x-direction is not verified under the Type 1 seismic action considered, or the most dangerous situation.

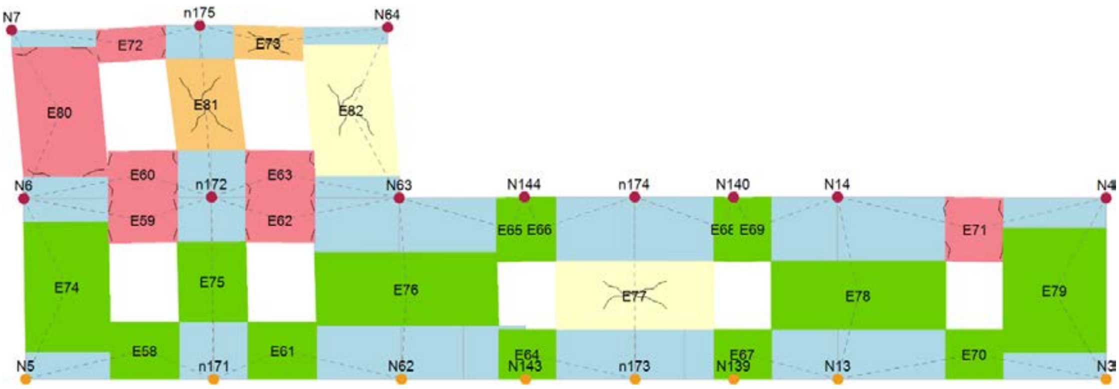


Figure 4-1 - Damage state in the lateral elevation view of the Pombalino case study, seismic action direction –y First Mode

The analysis of the overall behaviour of the structure showed how the building is vulnerable to the seismic actions, especially in the y direction, where both the analyses (-y 1° Mode and –y Masses, related to Type 1 and Type 2 of earthquake) are not verified. This is probably due the fact of the irregularity in elevation (Figures 4-1 and 4-2).

The damage state of the elevation views of the building was drawn by the 3Muri software about the presumed ultimate displacement of the structure in the capacity curves. It can be seen in light yellow colour parts of the masonry characterized by shear damage, and in strong yellow the shear failure. Pink colour represents the bending damage, while the light blue colour the tension failure and the green colour undamaged parts of masonry.

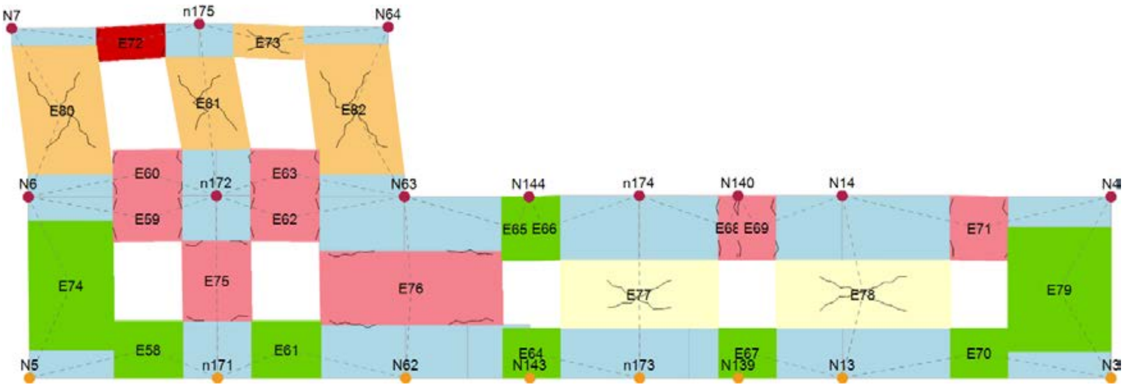


Figure 4-2 - Damage state in the lateral elevation view of the Pombalino case study, seismic action direction –y Masses

The lack of wooden elements is certainly a penalizing factor regard the Pombalino building behaviour under the seismic action. Thus, if all the anti-seismic measures provided by the reconstruction plan, adopted in the XVIII century, including the presence of timber elements well connected to the structure and the respect of the regularity in plan and elevation, probably its performance would be better.

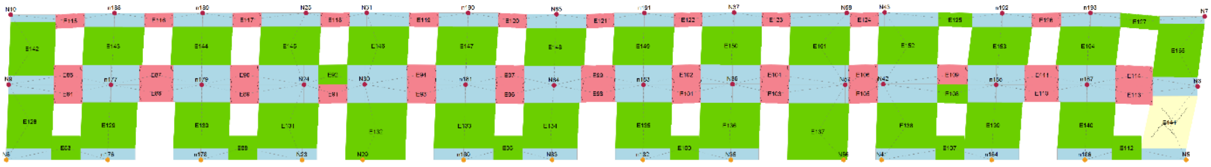


Figure 4-3 – Damage state in the frontal façade of the Pombalino case study, seismic action direction +x Masses

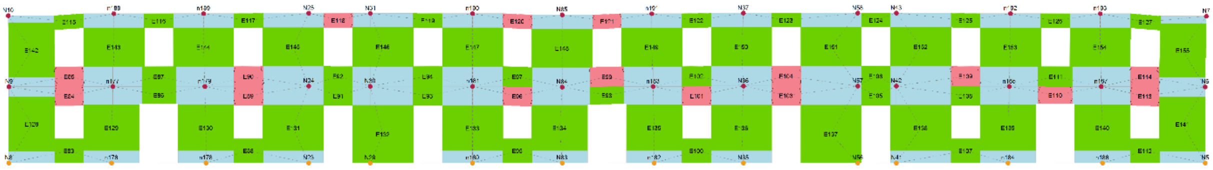


Figure 4-4 - Damage state in the frontal façade of the Pombalino case study, seismic action direction +x First Mode

This work starts up with the intent to clarify if these traditional systems can be used still today and therefore to analyse the retrofitting techniques to improve the existing buildings seismic performance. Therefore, the weaknesses and strengths regard on their seismic behaviour were highlighted and compared each other in the following steps of the thesis work developed at Università della Calabria[23], regard the Baraccata case study.

[23] R. Parrotta Master Thesis work at Unical: Vulnerabilità sismica degli edifici con intelaiature in legno: confronto tra gli edifici "Pombalino" e la casa "Baraccata"

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