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Assessment of Seismic Vulnerability of Vernacular Buildings in Urban Centers: The Case Study of Vila Real de Santo Antonio

U Cheng Vong

Portugal | 2016

ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

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University of Minho

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DECLARATION

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Year: 2016

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I hereby declare that the MSc Consortium responsible for the Advanced Masters in Structural Analysis of Monuments and Historical Constructions is allowed to store and make available electronically the present MSc Dissertation.

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ABSTRACT

Vernacular buildings are vulnerable to seismic action ascribed to inadequate past interventions, as seismic–resistant measures were often incorporated within the original structural system and constructive techniques. The term, Local Seismic Culture, is associated with specific seismic–resistant measures that a local community have been adopted and evolved throughout its history. In the growth of modern construction techniques and materials, however, it is a challenge to conserve its practice. Besides, the negligence of the effectiveness of the specific resistant measures is fostered by the long recurrence interval of great intensity seismic events. An historical center in Portugal, Vila Real de Santo Antonio, illustrates these issues from an urban scale to individual building blocks. As an new city built after the 1755 devastating earthquake, its urban plan and building construction followed the seismic mitigation approaches named after the Marquis of Pombal. Nevertheless, in the last two centuries, the seismic–resistant measures in this region have been altered or erased from the intended functionality. It is thus important to raise awareness of the seismic resistance and effectiveness that the original constructive methodology provided.

In this study, a masonry building aggregate, the Alfândega block in Vila Real de Santo Antonio, located in the south region of Portugal, was chosen as a case study for the assessment of the seismic vulnerability based on a nonlinear static (pushover) analysis. Four numerical models were used to assess the seismic vulnerability of the structure as isolated buildings and as an aggregate block. The analyses demonstrated that the global behavior of this type of construction are interdependent of the constituted buildings in a complex manner. Generalization of the seismic behavior of such construction type should be averted. A parametric study of floor beam stiffness was also included in order to provide insight for future interventions. It was found that higher stiffness of floor system resulted a higher capacity and lower level of damages present in structure.

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RESUMO

Os edifícios tradicionais de alvenaria (arquitetura vernácula) são vulneráveis à ação dos sismos, o que está muitas vezes associado a intervenções inadequadas no passado, como medidas sismo-resistentes incorporadas nas técnicas construtivas e sistemas estruturais. O termo Cultura Sísmica Local, está associado com o conceito de medida sismo-resistente que uma comunidade local adota e que evolui ao longo do tempo. Com o aparecimento de técnicas construtivas modernas e materiais, a sua conservação permanece um desafio.

O abandono da cultura sísmica local ou mesmo a remoção de elementos sismo-resistentes existentes no passado na arquitetura vernácula é uma consequência do período de recorrência dos eventos sísmicos. O centro urbano de Vila Real de Santo António, localizado na região do Algarve no sul de Portugal, ilustra esta questão desde os edifícios agregados ou isolados existentes na cidade. De facto, a seguir ao sismo devastador de 1755, o plano urbanístico e as técnicas construtivas foram baseadas nas medidas de mitigação do risco sísmico propostas pelo Marquês de Pombal. Todavia, nos dois últimos séculos, as medidas sismo-resistentes foram abandonadas e os edifícios alterados relacionadas com sucessivas mudanças de uso. Assim, considera-se importante ter conhecimento do comportamento sísmico na estrutura original e após as alterações introduzidas ao longo do tempo para avaliar a vulnerabilidade inicial a que resulta das alterações.

Neste trabalho, foi selecionado como caso de estudo o edifício da Alfandega localizado no centro urbano de Vila Real de Santo António para a avaliação da vulnerabilidade sísmica de edifícios pertencentes à arquitetura vernácula em contexto urbano através da análise não linear estática (análise pushover). Foram definidos quatro modelos para avaliar numa primeira fase a vulnerabilidade sísmica dos edifícios isolados e que se consideram constituírem o agregado e numa segunda fase a vulnerabilidade sísmica do edifício completo.

As análises efetuadas permitiram concluir que o comportamento sísmico global depende da interação entre os diferentes corpos que constituem o edifício, ainda que devido à geometria particular do edifício fosse possível verificar que o comportamento global do agregado está associado à vulnerabilidade de um dos corpos que compõem o edifício. O estudo paramétrico em relação à rigidez dos pavimentos de madeira revelou que a presença de pavimentos mais rígidos altera o comportamento dos edifícios, tendo influência da capacidade, nos modos de colapso e no nível de dano.

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1.0 Introduction

Architecture is a tangible way for expressing the development of civilization and identifying a cultural heritage, it embraces the values that are significant for the past, present and future generations in terms of aesthetic, historic, scientific, environmental, social and spiritual aspects. For this reason, architectural conservation is of paramount importance to the concept of sustainable development. As defined by the United Nations General Assembly in 1987:

> *“Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”*

Sustainable development or simply termed as sustainability, comprises of three pillars for: people, planet and profit. The practice of architectural conservation satisfies all of the pillars by demonstrating the human progression and limiting the lifecycle energy required for new materials (energy consumed in raw material extraction, manufacture, transportation and disposal); enhances the past understandings while offering a visionary perspective. The difficulty in architectural conservation involves various issues that are particularly distinctive and dependent on the local traditional practice and the environment. For example, the issue of authenticity amplifies the discrepancy between the conservation approaches adopted by eastern and western culture. Japanese has a tradition of replacing timber elements of a heritage structure, contrast to the emphasis of preserving the maximum amount of the original materials as possible in the western standpoint.

Contrary to monumental architecture, vernacular architecture is more suitable for portraying the development and characteristic of a place. Nevertheless, the conservation of non – monumental structures that built up the fabric of a historical city is usually being neglected. Vernacular buildings undergo continuous and inevitable changes as a result of different requirements proposed by the inhabitants. Such as architectural appearance, interior comfort, sustainable use of energy, and structural safety, etc. These aspects seem to hold against its conservation. Asides, the fast - paced evolution of innovative technology and materials challenges the preservation and rehabilitation of original construction systems and used materials; dramatically transforms the local architectural features that associated with the tangible and intangible values of that place into some indistinguishable built forms that are considered as more comprehensive and economical. In terms of structural safety, implement any interventions without considering and understanding the intended functions of the original constructive system is often detrimental for both the old and the new parts of a structure, which is usually the case in seismic – prone areas. Since historical constructions were built without the modern technology, seismic – resistant measures were taken into account along the constructive system and materials. Moreover, these measures were evolved over time, developing a distinctive building practice of the region related to its seismicity; flourishing its own heritage and cultural identify that differentiate itself from other places. Therefore, regarding the conservation of a historical city, it is essential to recognize the traditional building techniques and methodologies employed in vernacular architecture.

Portugal had been suffered from numerous catastrophic earthquakes between 1531 and 1998, which caused considerable damages and losses in various regions. Afterwards, the most affected regions tended to generate and adopt its own seismic – resistant measures to mitigate the consequences of earthquakes, which is known as Local Seismic Culture (LSC). Local Seismic Culture hence refers to a distinguishable and regional constructive methodology developed by empirical knowledge of the local community. However, long recurrence interval of seismic events in some of the regions has foster oblivion or abandonment of the practice of Local Seismic Culture. One of the examples can be seen in Vila Real de Santo Antonio, which is a city built after the 1755 Earthquake. Although the city was taken all seismic – resistant measures into account at the time of erection, from an urban scale to each individual building, the frequency of seismic events in this region does not lead to a realization on the effectiveness of LSC. Furthermore, since the seismic – resistant measures in historical buildings were built without scientific verifications, new materials and constructive systems replaced by the inhabitants cause unfavorable actions toward horizontal movements. Therefore, it is indispensable to raise awareness on both the practice of Local Seismic Culture and its potential interventions.

1.1 Objectives

The scope of this project aims at reducing the seismic vulnerability of vernacular buildings located in the zones of moderate and high seismic hazard. The idea is to assess the seismic behavior and analyze the associated seismic vulnerability of typical vernacular buildings found in an urban center. A case study in the city of Vila Real de Santo António (Algarve region, south of Portugal) was proposed to be studied.

The numerical simulation intends to: (1) calibrate the numerical models based on known mechanical properties of materials and dynamic properties of the selected buildings; (2) understand in a more detailed way the resisting mechanisms of the different structural elements in masonry buildings under seismic loading; (3) assess the influence of distinct factors on the global seismic behavior, namely the influence of the assemblage of distinct parts of the structure, the presence of timber floors and the associated stiffness; and (4) use different timber floor stiffness as a measure for strengthening, with the effect on the global behavior.

1.2 Organization of the Thesis

Besides the introductory chapter, this thesis is composed of other five chapters.

In Chapter 2, an overview of the seismic vulnerability assessment of historical masonry structures is carried out, with the general concepts, the main factors affecting the seismic vulnerability, possible methodologies for seismic assessment and some remedial measures to mitigate seismic risk.

In Chapter 3, the concept of the Local Seismic Culture is outlined and some examples around the world are given. Additionally, a description of the urban organization of the city of Vila Real de Santo António located in the south of Portugal and selected structure as case study are presented.

Chapter 4 deals with the numerical simulation of an aggregate block situated in the old town of Vila Real de Santo António as described in Chapter 3. The first analysis is carried out on the identification of the natural frequencies of mode shapes of the constituted buildings that are considered to compose the complete block. Afterwards, a comparison is made with the frequencies and mode shapes obtained in the complete structure. Secondly, a nonlinear static (pushover) analysis is performed with the aim of understanding the global seismic behavior and the most vulnerable parts within the structure. The analyses of the component buildings and the complete block with the results of capacity curves and failure pattern that governs the seismic vulnerability of the complete structure are further explained.

In Chapter 5, the effect of flexible diaphragm in the global seismic behavior of the complete buildings is considered. Three different stiffness of timber beams are compared, including the case of zero stiffness which corresponds to the absence or very poor quality of timber floors.

Finally, in Chapter 6, conclusions of the work are presented and some future works are indicated.

2.0 Seismic Vulnerability of Vernacular Buildings in Urban Centres

2.1 General Concepts

Vernacular buildings are non-engineering designed structures that resulted from the use of traditional construction techniques and materials. They adapt to the local climate, functional requirements, loading conditions and other physical and environmental factors to which buildings are exposed. According to Rapoport, this type of architecture is more viewed as a progressive consolidation of the previously adapted models, than as a primitive building process. It originates specific typological models for each region and restrains the use of unconventional elements. It is often associated to the concept of Regional or Local architecture (Carlos, Correia, Rocha, et al. 2015).

Vernacular architecture reflects the building knowledge at certain period of time, as well as the available local materials, social, technological, and historical context of a specific society. It stands as the testimony of the evolution of empirical building knowledge and can be recognized as a regional icon (Varum et al. 2015; Carlos, Correia, Rocha, et al. 2015). About 80 to 90 percent of the total building stock in the world is considered as being part of the vernacular architecture, and indisputable that more than a half of the actual population reside or dwell in vernacular buildings (Ferrigni 2015b). Therefore, it is important to address the structural safety in vernacular buildings, especially for old structures in the seismic-prone areas.

The term *seismology* was developed by a civil engineer Robert Mallet about 150 years ago, but old buildings were merely constructed based on past experience (empirical approach) and without any scientific basis (Shearer n.d.). Despite the modern seismography offers a considerable reasonable warning interval, earthquakes cannot be predicted or prevented at the current level of scientific development (Sousa 2015; Fonseca & Vilanova 2015). However, the risk mitigation in vernacular buildings can be accomplished through seismic risk assessments. The seismic risk of vernacular buildings can be viewed the combination of three factors: seismic hazard, structural vulnerability, and exposure. These three factors emphasize the interrelation of structures with its environment rather than focus only on individual buildings. Seismic hazard describes the seismicity for a specified geographical location, by estimating the intensity of ground motion with a given probability of being exceeded during a time interval (Mallardo et al. 2008; Fonseca & Vilanova 2015). Thus, evaluation of seismic hazard is the pre-requisite in the risk mitigation procedure. The other two factors, vulnerability and exposure, are the variables. During earthquake events, structural vulnerability is mainly related to potential developed damages in buildings. Accordingly, a lower risk level can be achieved by adequate construction methodologies (low vulnerable buildings) and proper city planning. However, for an existing city, it is impractical to re-plan its layout in a large scale. Hence, controlling the vulnerability of the structures is the most desirable solution. Several authors have been exemplifying considerations in the reduction of seismic vulnerability, and will be explained in details in the following chapters. In the next Chapter, the procedures in seismic vulnerability assessment are examined in order to provide a holistic understanding on the parameters that determine the seismic vulnerability of a structure.

2.2 Seismic Vulnerability Assessment

Seismic vulnerability is usually evaluated at different levels and with distinct methodologies, to which various Italian documents and the European code can be referred to. According to Mallardo et al. (2008), the assessment can be classified into two levels. The primary level is rather coarse and somewhat empirical and its application is appropriate to urban centers but also to single buildings. The secondary level, which is more detailed, can be applied to single or aggregate buildings (Mallardo et al. 2008).

The first level contains "typological" data of the structure. Construction methodologies and materials are often examined and described related to their eventual structural performance and seismic response. Different factors are considered in each of the examined element, such as the overall resistant system, materials, total resistance of horizontal loads, structural organization, structural irregularity in plan and in elevation, etc. The objective is to assign the examined element with vulnerability classes that are associated with each parameter analyzed. Furthermore, seismic vulnerability indexes can be derived following different, continuous vulnerability curves, relating the damage with a seismic intensity, can be built based on mathematical models (Calvi et al. 2006).

The second level is a more detailed investigation of the structure. This level is common for performance assessment and design verification of buildings. In other words, the capacity of the structure is usually compared to the demand corresponding to a certain seismic hazard. For the seismic vulnerability assessment of historical constructions, material nonlinear analysis is often used, and generally from nonlinear static analysis (pushover) capacity curves can be derived for vernacular and historical masonry structures. Nonlinear dynamic analysis (time - history) can be also employed. Although the latter is considered to be more accurate in seismic assessment, the former is often adopted due to the complexity and computational effort required by the latter (Mallardo et al. 2008).

In this scope, different works are available in literature (Lang 1982; Ramos & Lourenço 2004; Cardoso et al. 2005; Mallardo et al. 2008) provides insight on the use of numerical analysis for the seismic evaluation of seismic vulnerability of masonry buildings). Nonetheless, vulnerability assessment for historical structure remains a challenge. The lack of documentations and heterogeneity in construction techniques and materials increase the uncertainties in determinations of governing factors and decisions in analyses.

2.3 Seismic Vulnerability Parameters

As seismic vulnerability is related to the degree of structural damage induced by a given earthquake, the parameters with respect to the building geometry, material properties and construction techniques are the most influential in the seismic response of a structure. Regarding the building geometry, plan and elevation of a structure govern its distribution of mass and stiffness. When the structure is under a combination of vertical and horizontal loads due to earthquakes, torsional effects can be developed depending on the building plan. A compact, uniform and homogeneous plan, which has a better distribution of mass and stiffness, limits the potential of torsional effect and contributes for the structural stability. For a structure comprised of different buildings, symmetry of the plan is required taken into account those requirements for an isolated building above mentioned, since the distribution of forces is determined by the symmetrical continuity of each volume. A symmetrical plan provides uniform distribution of mass and stiffness and thus provide a greater torsional resistance (Lima, Correia, et al. 2015). In terms of seismic loading capacity, an unsymmetrical plan has no remarkable discrepancy compared to a symmetrical one unless the unsymmetrical part has a significant size. However, capacity of the buildings under seismic action are possible to be dominated by the eccentricity of the stiffness and plan configuration (Javier Ortega et al. 2015). Plan configuration such as length to width ratio is associated with a better seismic resistance to the horizontal directions. A low length to width ratio provides also similar trend and thus reduce the seismic vulnerability in both directions. Nevertheless, long walls are vulnerable and tend to exhibit large out–of–plane deformation when no proper connections to the transversal walls or floors of structures are provided (Ortega & Vasconcelos 2015; Correia et al. 2015).

Elevation determines the force concentration and the deformation demand at certain levels by the distribution of mass and stiffness. The concentrations in elevation are the floor height, opening layout, and different exterior structural or non–structural elements. As the height of load bearing walls are associated with the capacity of the seismic resistance in both horizontal directions, the height not only governs the stiffness distribution at the levels but also the failure mechanism (Javier Ortega et al. 2015). Besides, the in–plane behavior is highly dependent on the opening layout since openings introduce discontinuity and weakness on the walls. Distribution, quantity, dimension and framing elements are the critical factors. The distribution and numbers of openings induce different locations of stress concentration. Openings near the wall corners are extremely hazardous due to the presence of discontinuities between the perpendicular walls. The dimension of openings is associated with bending stress and deformation on the wall system, which can lead to irregular behavior of the building (Correia et al. 2015). Hence, the framing elements that are used to minimize and distribute the stress concentration are essential to reduce the seismic vulnerability regarding the failure mechanism of the wall systems. The exterior structural or non–structural elements, such as balcony or decorative elements in the seismic–prone area are often eliminated, because these elements increase the level of vulnerability by interfering the stiffness distribution in elevation.

Another factor that can affect the structural vulnerability with respect to building geometry is the type of roofing system. The weight of the roofing system can lead to mass concentration on top of the building, and induce high concentrated loads and damage when heavy roofs are selected. The roofing system can also lead to the formation of new vulnerable elements in the structure. For example, an unbraced gable wall resulted at the end of a truss roof is susceptible to collapse towards out–of–plane direction and consequently decrease the seismic loading capacity (Javier Ortega et al. 2015).

In additions, the location of center of gravity modifies the seismic capacity of a structure, as it can lead to torsional effects. When the mass of the structure is concentrated towards the ground level, the base has a higher seismic demand and tend to offer a better response to horizontal components of the seismic shock (Ferrigni 2015a). The concern for gravitational center is common in seismic -prone regions. Heavy materials such as stone are used at the base, and lighter materials such as timber are superimposed on top of the heavy materials. An alternative method observed in order to lower the center of gravity is to reduce the wall thickness at the upper stories.

Along with the building geometry, the materials used for structural elements are also critical in the contribution of seismic vulnerability, especially for masonry structures. The quality of the construction materials determines the strength of structural systems and, therefore, governs the structural behavior under seismic loadings. Since masonry has low tensile strength, the application of other tensile reinforcements is required to improve the integration between the structural elements or minimize the consequences of collapse mechanisms. Compatibility of reinforcements with the original structural elements is an additional aspect that modifies the structural performance. Besides, durability of the used materials plays an important role on the maintaining the intended capacity. Material aging and deterioration contribute to a great extent to the vulnerability and decrease the seismic resistant of buildings. Some timber structural elements are prone to biological attacks hence decreasing or limit the intended functionality for seismic resistant measure.

Furthermore, the connections between structural elements such as wall-to-wall, wall-to-floor, and wall-to-roof, are the key issues contributing for the seismic vulnerability. The lack or improper interlocking systems facilitate out–of–plane displacement and decrease the global capacity of the building. Since box behavior of the structure relies on these connections for translating the horizontal forces exerted by earthquake to be absorbed in the same plane of the wall. Structures with box behavior have better seismic performance than those without adequate connections (Ortega & Vasconcelos 2015). For a historical building, quality of construction and environmental effects are the prime concerns that define the conditions of the connection details, as well as the past interventions of the structure. The increase of vulnerability in a masonry building is usually ascribed to the significant modification of the initial structural system, and/ or with the insertion of new materials that are not compatible with the original ones. Poor maintenance and lack of continuous monitoring also contribute a great extent on the vulnerable measure (Paula & Coias 2015).

The parameters of seismic vulnerability explained above are the most influential to the structural response of isolated buildings. The investigation and strengthening strategies of these parameters sometimes are relatively complex and difficult due to the physical constraints and considerations regarding the compatibility to existing materials.

2.4 Typical behavior of historical masonry buildings in seismic events

The accurate definition of masonry as a single structural material has been examined over time due to the variety of materials used for the units and joints, as well as the corresponding arrangement and geometry (Lourenço et al. 2011). The materials commonly used in historical masonry buildings are stones, bricks, adobe, ashlars and others, depending on the available material sources near the construction site. The joint between each unit, known as mortar, can be clay, cement, hydraulic or hydrated lime based mortars. Thus, the strength of masonry buildings depends partially on the properties of the units and joints, along with the construction methodology and techniques. The compressive strength of the stone varies according with the type and quality. The compressive strength of limestone can be ranged from 5 MPa (low quality limestone) to 130 MPa (good quality limestone). For the mortars that mixed with cement, lime, and, soil and water, its compressive strength can be varied from 1.5 - 3.5 MPa. The specific mass of stone masonry usually ranges between 1700 kg/m^3 to 2200 kg/m^3. Most of the masonry structures exhibit good compressive capacity, but the tensile strength, shear capacity and ductility are relatively low. The mechanical behavior of masonry also shows that nonlinear behavior is predominant. For a slender and low-quality existing masonry structures, the parameters that most influence the seismic performance are the quality of masonry walls, associated to adequate or inadequate mechanical properties like compression and tensile strength, and existence of timber floors, assuring in certain extend some confinement against out-of-plane loading (Mendes & Lourenço 2013; Mallardo et al. 2008).

In general, masonry buildings are composed of load-bearing walls (Figure 1), and sometimes include partition walls in the interior. As the in-plane wall dimensions (length and height) are usually greater than the thickness, the in-plane stiffness is substantially higher than the out-of-plane stiffness (Lourenço et al. 2011).

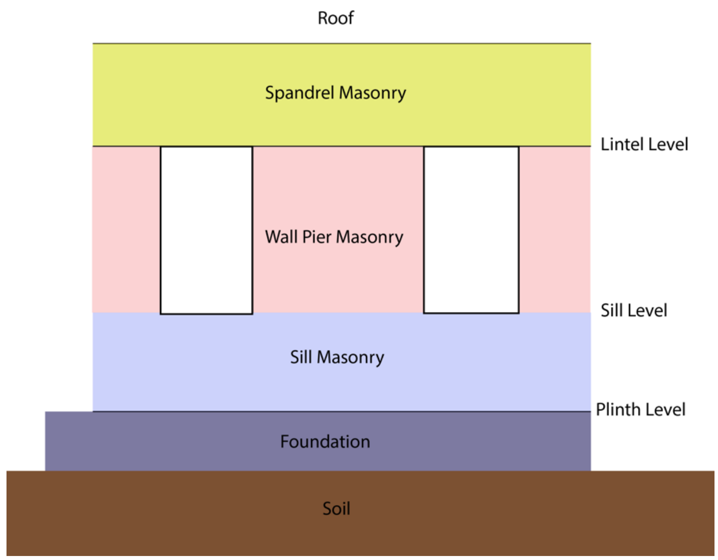


Figure 1: Example of shear walls in masonry structure (credits: U. Vong).

This indicates that the structural response of a masonry building is determined from loading combination and by the relative resistance of the structural walls to in-plane and out-of-plane loading. In connections among the structural elements re ensured, preventing premature out-of-plane collapse of the walls, in-plane resisting mechanism develops and the global stability of the buildings is ensured by the shear walls. Depending on different factors, such as the materials, vertical load pre-compression but particularly the geometry of the walls (height to length ratio), the typical in-plane failure mechanisms for shear walls are rocking, sliding, diagonal tension and toe crushing, which are associated with the wall piers next to the openings. For a slender masonry wall pier, with a light weight superimposed structure, rocking is possible to occur under seismic loadings. Horizontal cracks are generated on top and bottom of the pier at opposite and diagonal corners, as shown in Figure 2. Rocking can also induce crushing at the corners. Contrarily to rocking, sliding is characterized by horizontal cracks generated in the full width of the pier between openings. It occurs when the horizontal forces at the piers exceed the shear strength of the bed joints. Sliding occur at different levels, such as at the connection of the piers to the spandrels (Figure 3). The predominance of different crack and failure modes depends on several factors: the magnitude of the earthquake - induced inertia force; the building weight (the vertical load); the area of opening; and the framing type, namely at the level of the height to length ratio of the piers. Nonetheless, the most common failure mechanism for masonry buildings is diagonal cracking associated. When the principal tensile stresses resulted by the combination of vertical and horizontal forces exceed the tensile strength of masonry, diagonal cracks are propagated along the bed and head joints, or through the units depending on the strength of the mortar, units and the unit-mortar interface. As the toes of the wall piers are usually subjected to high compressive stresses, compressive failure (which is known as crushing) occurs when the principal compressive stress exceeds the compression strength of masonry. Toe crushing is often appeared in combination with flexural rocking or diagonal tension (Mendes & Lourenço 2013).

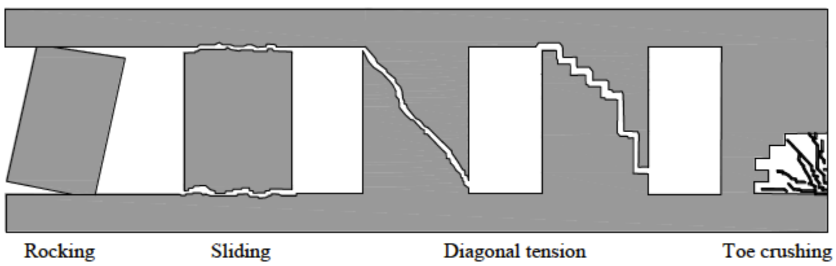


Figure 2: In - plane collapse mechanisms of wall piers beside openings (credits: Mendes and Lourenco, 2013).

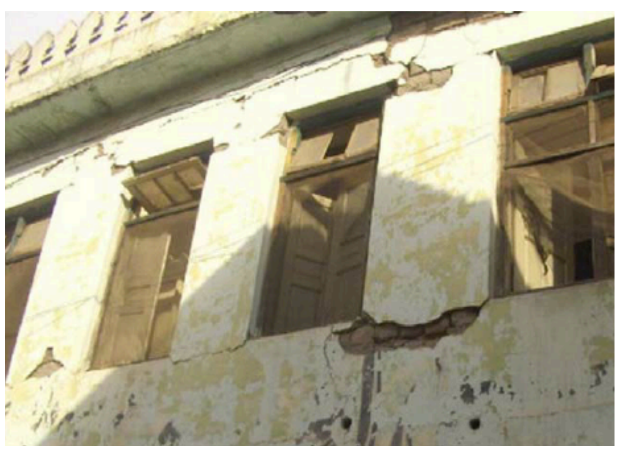


Figure 3: Rocking failure of masonry pier in 2005 Kashmir earthquake (credits: Javed, 2008).

Besides the typical cracks in the masonry shears walls, additional cracks can develop at the connections between structural elements. For unreinforced masonry, cracks can be occurred between walls and floors, at the corners, at the wall intersections, in spandrel beams and/or parapets. Furthermore, additionally to diagonal cracks, partial disintegration of the structural elements, out-o -plane collapse of the perimeter walls, or complete collapse of the building can also be resulted from an earthquake (Mendes & Lourenço 2013; Mallardo et al. 2008). When a historical masonry structure behaves without a “box behavior”, generally presents local out-of-plane collapse of the walls, leading often to the global collapse of the buildings. Masonry structures whose structural elements exhibit good connections (between intersection walls and between walls and floors), global behavior is more effective, being characterized by the “box behavior”, and thus show better performance under earthquakes. Therefore, the capability of redistributing the horizontal loads among structural elements is more effective (Lourenço et al. 2011). In effect, “Box behavior” refers to the diaphragmatic action provided by the floors. When the floors are rigid and adequately connected to the masonry walls, shear resisting mechanisms can effectively develop, since rigid diaphragm distributes the horizontal forces exerted by earthquake into the vertical elements with respect to the location and lateral stiffness of the elements. Nevertheless, in historical masonry structures, timber floors are usually found, behaving as rather flexible diaphragms. These can lead to a a significant bending and shear deformation under horizontal forces, and influence the distribution of the loads among the vertical elements of the structure. There are four main conclusions on flexible diaphragms (Lourenço et al. 2011):

1. The supports at floors behave as a spring supports;
2. As the floors present high strength and large deformation capacity with respect to the mass, the failure mechanisms of the flexible diaphragms are related to the inadequate connection between the diaphragms and the walls;
3. Flexible diaphragms exhibit a highly non-linear hysteretic behavior when peak ground acceleration is high;
4. The increase of in-plane stiffness of diaphragms can be achieved by in-plane strengthening. However, it cannot be sufficient to improve the global response of the building.

When a flexible diaphragm is presented, the predominance of a more local or a global response of a masonry building under seismic loads is associated to the flexibility of the diaphragm and its connection with the walls, which in turn influences the capability of seismic load redistribution. A masonry building with flexible floors with inadequate connections will exhibit a global behavior which is independent on the floors.

The non-structural elements such as partition walls that are connected to the structural elements also contribute to the global seismic performance (Mendes & Lourenço 2013; Lourenço et al. 2011). Examples of this situation are the *Pombalino* buildings in Portugal where timber frame partition walls should behave as additional seismic resisting elements. When connected transversely to the masonry walls at the envelope, can serve also as stiffening elements that prevent their premature out-of-plane collapse.

Another important issue to consider in the seismic behavior of old masonry buildings is that at the urban level, buildings are structurally connected to the neighboring buildings. Isolated buildings are rarely the case in an urban setting. The overall structural behavior of masonry structures is therefore not able to be generalized and considered without the “aggregate effect” (Vicente et al. 2011).

2.5 Seismic Behavior of Building Aggregates in Urban Fabric

At the urban scale, a building aggregate refers to an integration of different buildings with a compound structural arrangement. Rather than as a group of buildings, an aggregate behaves as an interdependent system within the aggregate itself and sometimes with the adjacent aggregates or buildings. Unlike isolated buildings, the seismic response of aggregates is strongly influenced by the interaction between the constituted buildings and the urban environment as part of the urban fabric. The dimension of a building aggregate depends on the size of the located area and its urban planning. Four types of urban plan can be distinguished in seismic prone regions: (1) seismic–oriented arrangement; (2) historical centers with organic matrix; (3) scattered or rural environment; and (4) conglomerated urban context (Viana et al. 2015).

Building aggregates within a seismic–oriented urban exhibit a strong regularity in terms of plan, elevation and other architectural features. Examples of the seismic–oriented urban plan can refer to Lisbon and Vila Real de Santo Antonio in Portugal, as illustrated in Figure 4. This type of urban planning is implemented either after a devastating deconstruction by an earthquake or as a completely new planning that take seismic action into account. A seismic–oriented urban plan is comprised of a strict orthogonal grid, by organizing the building aggregates and the common spaces deliberately. As suggested by Viana et al. (2015), the composition of the urban fabric, such as the aggregates’ volume, height and proportion establish a “closer type–morphological relationship” that forces the adaptation of buildings in a more interdependent way with the surrounding environment. The orientation of the building aggregates is also considered within such urban plan. The aggregates are often oriented to the direction of the main concussion in order to reduce the exposure and impact of seismic waves. In both cases of Lisbon and Vila Real de Santo Antonio, the façade, height of the buildings, width of the streets, building orientation and other aspects were defined in the planning proposal. More details of these examples will be explained in the next chapter of the Local Seismic Culture in Portugal.

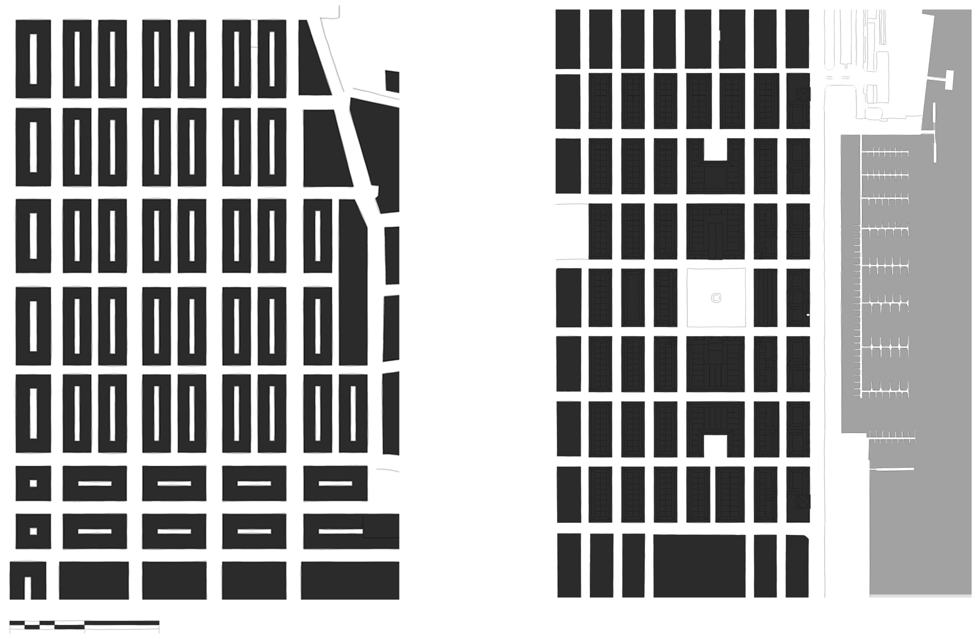


Figure 4: Seismic - oriented urban plan: (left) Lisbon; (right) Vila Real de Santo Antonio (credits: CI-ESG).

Unlike the seismic–oriented urban arrangement, for an urban center that had been suffered from an earthquake without a complete destruction it shall be rebuilt or consolidated on the existing urban fabric. The existing urban fabric for urban centers are usually as an organic matrix, where seismic provision in urban planning had not been realized at the time of erection. Therefore, the building aggregates follow the confinement of the existing plan and forming a more intricate structural arrangement, increasing the impact and vulnerability from seismic activities. Examples of this urban planning can be seen in the Portuguese historical centers of Evora and Lagos, as shown in Figure 5. These historical centers have no specific urban arrangement but present a compact built environment.

The urban plan for scattered or rural environment is characterized mainly by the occupation of isolated buildings, but sometimes with small sized aggregation systems. The open spaces, density and topography of the located territory define the arrangement and dimensions of the building aggregates. The constraints in composition and size of each aggregate are not the major concerns compared to those in a compact urban environment. Some of the rural urban fabrics are illustrated in Figure 6.

A conglomerated urban plan refers to a hybrid or coexistence of seismic–oriented and organic matrix arrangement. This type of urban plan resulted from the contrasts between the initial reconstruction proposal by administrative authority after an earthquake and the actual implementation by the private owners. This is a common post–earthquake phenomenon for infrequent seismic areas. Nevertheless, this urban type can also be regarded as the growth of original urban context with an extension of new part of the city after an earthquake. This can be illustrated by the case of Benavente in Portugal. In Figure 7, the highlighted area represents the new development after the 1909 earthquake. This part as an expansion for the population affected by the earthquake adopted some strategies that can be seen in the seismic–oriented urban plan previously mentioned.

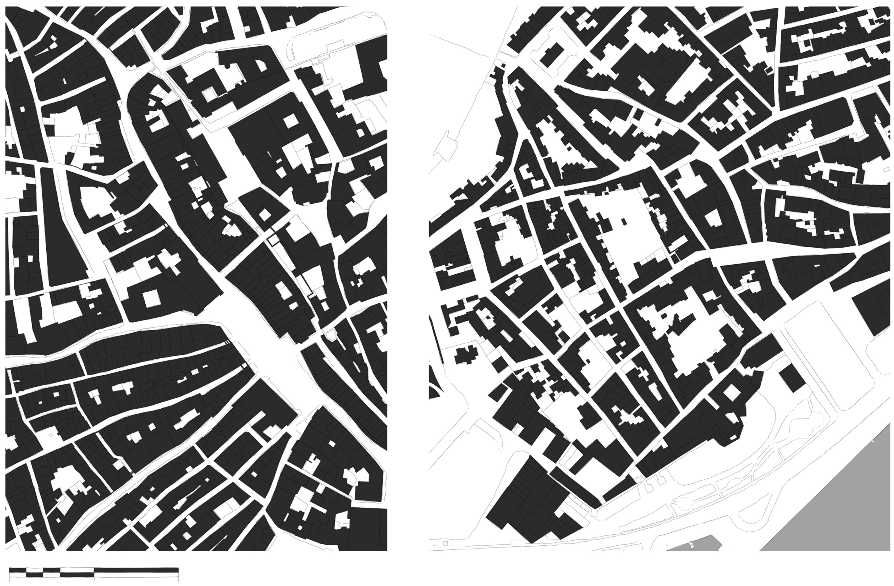


Figure 5: Historical center with organic matrix: (left) Evora; (right) Lagos (credits: CI-ESG).

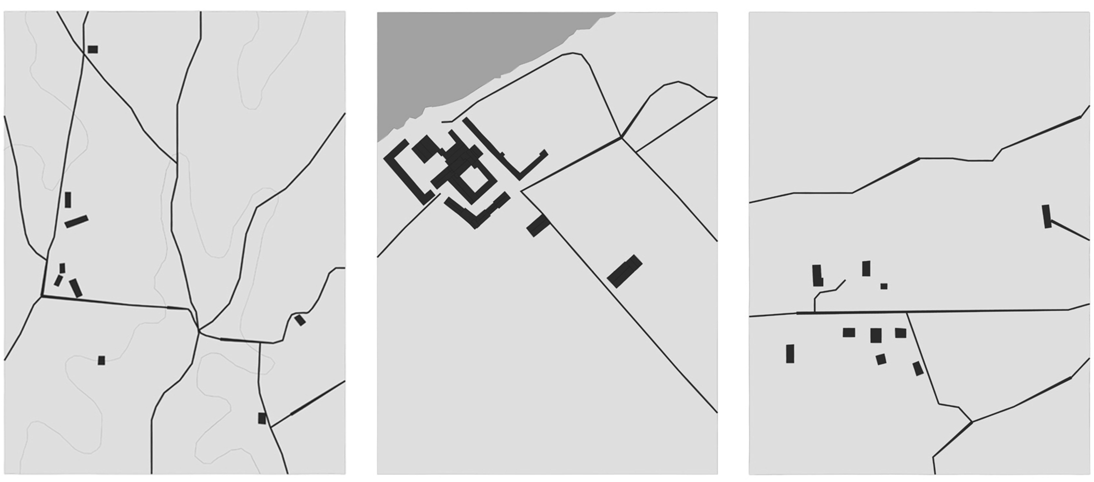


Figure 6: Scattered or rural environment: (left) Trindade; (middle) Coruche; (right) Melides (credits: CI-ESG).

Building aggregates can be divided into two distinctive typologies, continuous (or known as row) buildings and blocks (Figure 8 and Figure 9). Each compound within a continuous or row building aggregates has a similar construction technique and materials to the other compounds. It is evident that the continuous or row buildings in Portugal combined symmetric and regular plan with two orthogonal axes of symmetry. On the other hand, the block–type building aggregates are comprised of a diversity of forms. Nevertheless, in the seismic demand context, the components of an block aggregate are usually geometrically regular and symmetrical with a homogeneous architectural language (Viana et al. 2015).

Figure 7: Benavente Plan after 1909 earthquake; new extension of city highlighted in color (credits: CI-ESG).

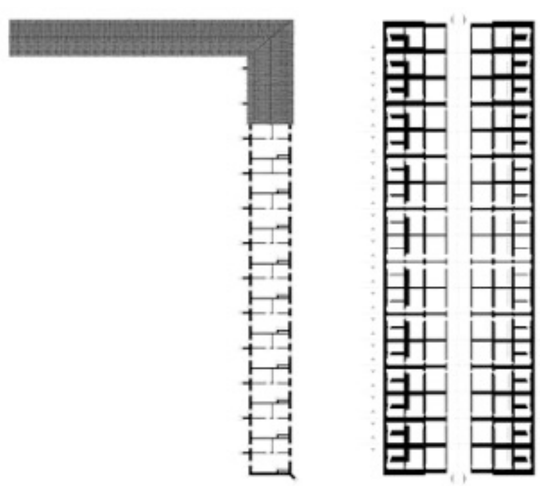


Figure 8: Continuous or row building aggregate. (Left) a row building in Coruche; (right) municipal district plan designed after the 1909 earthquake in Benavente (credits: CI - ESG).

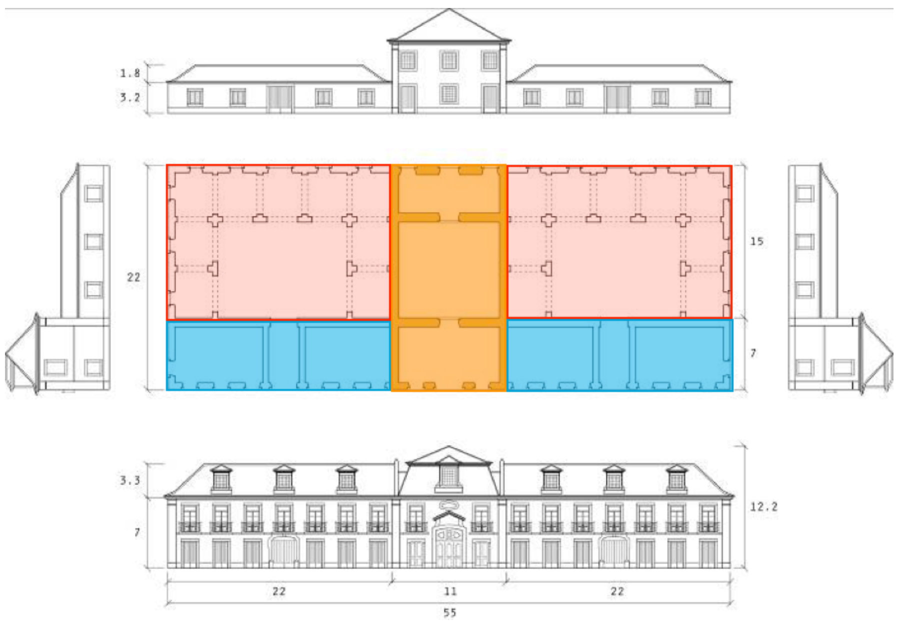


Figure 9: Building aggregate block. The Alfandega block in Vila Real de Santo Antonio comprises of five different buildings (Modified from J. Ortega).

Aside from the located urban environment with the influence of adjacent (which from now on, denoted as two different isolated buildings or aggregates that connected together by some structural elements) structures, the global seismic behavior of an aggregate is related to the interaction between the constituted (adjoining) buildings with respect to its own organization and constructive building features (Vicente et al. 2011). The response of each constituted building can have advantages and disadvantages toward the seismic resistance of the aggregate. The most common aspects regarding the global seismic behavior of aggregates are:

- Material properties of structural elements within each constituted building
- Position or layout of each building within an aggregate
- Typological heterogeneity among adjoining or adjacent buildings, such as the relative height, presence of staggered floor, and area of opening with respect to the other constituted buildings

These aspects are associated with the structural response of each building within an aggregate, in terms of mechanical principles, continuity and lateral displacement control. For example, a reinforced concrete floor slab in one building is possible to introduce hammering action to its adjoining buildings constructed with timber floors. The difference in stiffness and the relative position within an aggregate determine the global structural response under seismic action. Sometimes the stiffness difference and location can lead to premature detachment between two buildings (Ramos & Lourenço 1999). Therefore, each structural element, or any typological heterogeneity has a potential to impose risk to the adjoining buildings. On the other hand, the adjoining or adjacent structures can be used as preventive and reactive measure to limit and minimize damage produced by earthquake. When the adjoining or adjacent structures are in collaboration or behave as interdependent systems, which react to the impact of seismic demand as a whole, each structure is functioned as reinforcement for the other structures in contact and thus enhance a better structural continuity and seismic response. The concerns about such behavior are often correlated to the sharing structural elements and connection details.

Numerous case studies have indicated that the constituted buildings within an aggregate are dependent and interrelated. For a more realistic approach, the structural analysis of this type shall not consider each adjoining building as an isolated structure (Ramos & Lourenço 1999; Vicente et al. 2011). The effect of these adjoining buildings can either be beneficial or detrimental in a complicated manner regarding the three aspects mentioned above. Each aggregate has a unique structural response with respect to its own configuration, geometry and material properties. It is hence inappropriate to generalize the seismic behavior of this type of structure.

2.6 Seismic Mitigation Approaches and Measures

There are two approaches for seismic mitigation: morphological and tectonic. The basis for morphological approach regards to the specification of the constructive typology, in which controls the response of a structure under seismic loading. On the other hand, the tectonic approach is associated with the integration of seismic – resistant elements in the structural system. These two seismic mitigation approaches can be divided as reactive or preventive measure, or a combination of the both (Lima, Correia, et al. 2015).

Reactive measure aims to mitigate the already made damages. It is usually employed in regions with less frequent seismic occurrences. The identification of reactive measure is often associated with the presentation of three or more reinforced techniques and elements in a building (Correia & Carlos 2015). These techniques and elements will be “activated” as a reaction to prevent further damage and intend to avoid total collapse of the structure. On the other hand, preventive measure anticipates future seismic event using seismic resistant features that are incorporated in a building during construction or retrofitting with respect to the past recurrent experiences.

2.6.1 Morphological Approach

The strategies in morphological approach involve the building geometry and stiffness distribution of a structure. This approach focus on the concerns about the stress concentration and torsional effects that interfere the structural stability under any horizontal movement. It suggests that the structural stability can be achieved by the simplicity in plan and elevation. A compact, uniform and homogeneous building plan for an isolated building provide an even distribution of stiffness and mass thus prevent the potential of torsional effects of the volume. This type of building usually consists of a rectangular form, with few internal partition walls and window openings, as well as a regular internal arrangement in order to improve the global structural behavior. In terms of continuous or row aggregates, when the plan adopts a T, L or U shape, the inner walls are usually extended in both directions in order to prevent the volumes perform as isolated units. Sometimes, buttresses are applied at the intersecting corners to avoid separation and restrain torsional effect, since these are the location subjected to high stress concentration (Lima, Correia, et al. 2015). For a building aggregate, symmetrical building plan is required to yield a uniform distribution of mass and stiffness and thus results an even force distribution. Nonetheless, building aggregates are considered as a combination of preventive and reactive measures. Asides, from the building plan, aggregates in seismic – prone area eliminate any composition elements on the façade as a preventive measure. While the rotation of the building under seismic loading can be restrained by the interaction within the constituted buildings and react together against seismic action. At the urban scale, the arrangement and orientation of the building aggregates are considered as the strategies for preventive measures. In a seismic – oriented urban plan that mentioned above, the orthogonal grid is outlined by rigorous street dimensions. These street dimensions are proportional to the height of the façade, and thus provide preventive measure to diminish the risk of collapse over the streets. The orientation of aggregates or building to the direction of the main concussion of seismic waves is also considered as a preventive measure with the anticipation of future earthquake events (Viana et al. 2015). Another strategy such as reducing the weight of the constructive material in the upper stories, thus lowering the center of gravity is considered as reactive action in the morphological approach.

2.6.2 Tectonic Approach

The tectonic mitigation approach can be categorized into four different strategies regarding the structural functionality: (1) use of constructive technique as structural reinforcement; (2) application of perimeter seismic – resistant elements; (3) utilization of arches as reinforcing systems; and (4) implementation of combined reinforcing elements for continuous or contiguous buildings. These four strategies are also seen as either reactive or preventive measure, or as a combination of the both. It depends on the intended action against the occurrence of earthquake (Lima, Correia, et al. 2015).

The use of constructive technique as structural reinforcement is to incorporate reinforcement layers during construction. For instance, course of tiles, ceramic brick, stone or timber elements are inserted among the monolithic block of rammed earth structure in the form of horizontal layers. These horizontal layers transfer the bending action and inertia forces between the transversal walls, as well as prevent vertical crack propagation on the walls. Other built – in structural reinforcing systems such as relieving arch, which integrate into the wall above opening used for diverting the vertical loads, and Pombaline systems, which employ timber elements as a secondary structure, are also evident in historical buildings. These strategies may or may not require sophisticated planning before the construction phase, depending on the building and the designated reinforcing systems. Yet, they can be considered as a combination of preventive and active measure.

Perimeter seismic – resistant elements are elements applied at the perimeter of the exterior wall, and there are four common types in seismic – prone areas: buttress; corner strengthening elements; reinforced plinth; and tie – rod (Lima, Correia, et al. 2015). The perimeter seismic – resistant elements have a greater integration in the rural context. On the other hands, these elements incorporate with the structural system rather than function as isolated systems in the urban context.

Buttress is used as a preventive and active action against earthquake; it provides stiffness and additional support to consolidate the structure. Buttresses are often applied at the center of edge walls and in a continuous manner for longitudinal walls. Sometimes, buttresses are leveled with the interior walls. Due to the limited space in the dense urban centers, buttresses are mostly used in the rural environment. When buttresses are identified only as a reactive measure, it is common to observe that the lack of cohesion with the exterior walls.

Corner strengthening elements are more appropriate for the urban environment. In a trapezoidal form, these elements are applied on the corners of the enclosure walls and usually raised up to the first floor of the building. They are defined as proliferation of the corners, and functioned similarly as buttresses. However, these elements aim to enhance the weakest points on the structure, since the intersection of perpendicular walls is extremely vulnerable to two directional horizontal movements.

Figure 10: Corner reinforcement in Giraldo Square, Evora (credits: CI - ESG).

Plinth level of masonry structures are often consisted of an addition of masonry in order to provide additional strength and stabilize the structure from overturning under seismic loadings. These reinforced plinths are also used to transmit the vertical loadings on the wall to ground. Reinforced plinth has a diversity of form in terms of the applied building geometry. As shown in Figure 11, reinforced plinth in a trapezoidal form is known as base plinth. For reinforced plinth with a rectangular cross section applied between buttresses, it is known as *poial* or stone bench. In some areas, such as Azores in Portugal, the reinforced plinth is applied at the full perimeter of exterior walls in order to resist the forces in both longitudinal and transverse directions as an reinforced ring.

Tie rods are the most common seismic – resistant element applied in masonry structures. It can be seen as a preventive or active measure against horizontal movement. When tie rods are placed within the floor structure, they can be seen as a preventive measure. This is a common practice for structures that had been suffered destruction after an earthquake. As a reactive measure, tie rods are inserted between two opposite (parallel) walls as a confinement to restrain out – of – place displacement of the walls. Sometimes, tie rods are applied in both longitudinal and transversal directions taking into account the different earthquake movements.

Arch elements can be presented in a variety of form, such as arches, vaults, and flat (or jack) arches. Arches and vaults are used to transfer the vertical loadings and distribute them evenly to other supporting structural systems. When an earthquake occurs, these elements confer more resistance to the base of the structure. Flat or jack arches are considered as a replacement of lintels above openings. These elements are typically made of fired brick and each thickness is usually the same as the masonry wall’s.

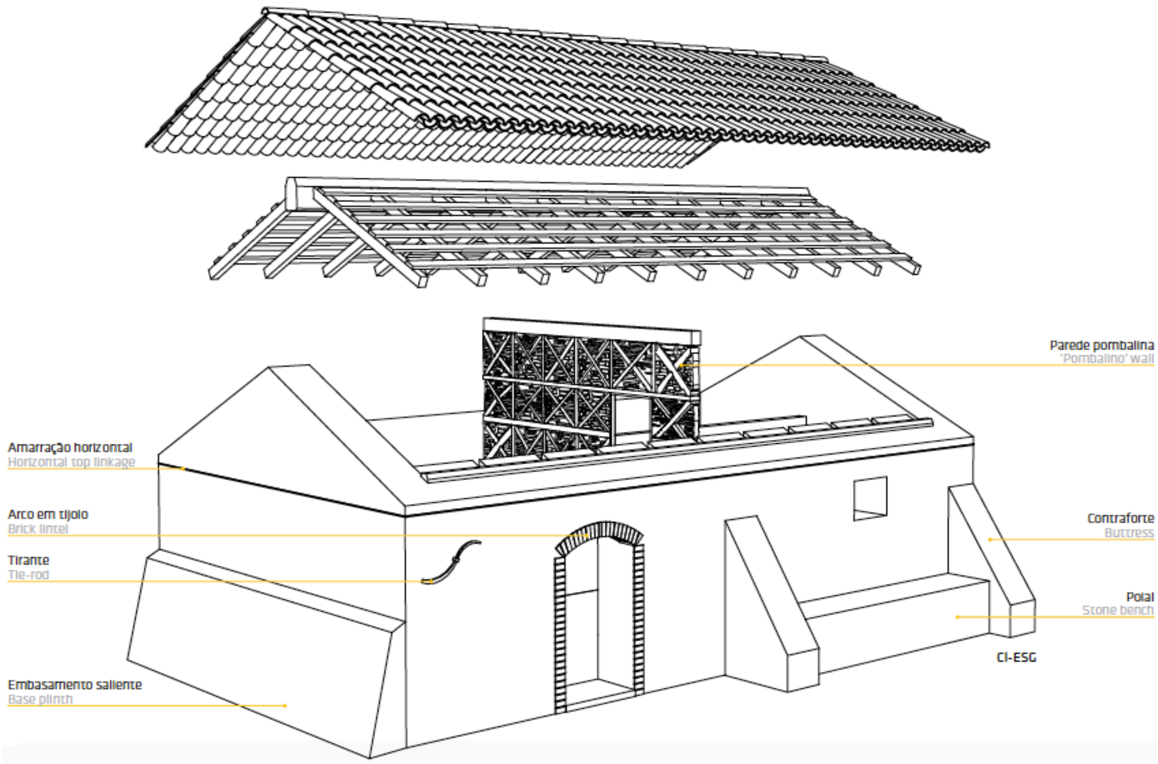


Figure 11: Seismic - resistant elements (credits: CI - ESG).

Counter – arch and continuous cornice are reactive reinforcements used for connecting adjacent and adjoining structures, respectively. Counter – arch is frequently used in historical centers between adjacent structures. This structural element was intended to block the collapse mechanisms between adjacent buildings at different heights. In the past, it was assumed to be used for consolidating already damaged structures. However, the main purpose of this structural element is to impose a collaboration between the adjacent structures, so that they behave uniformly under seismic loadings. By redistributing the horizontal forces from one building through the floor structure to the vertical walls of the adjacent building, consecutive collapse of the built structures is therefore limited. On the other hand, continuous cornice is used to tie the adjoining structures together in order to prevent separation between contiguous walls. Typically, it is made of solid brick or stone masonry, but sometimes metal straps are used to constitute this element. Metal strap cornice is often left at sight in stonework. When the structure is built with brick masonry, the metal strap cornice is usually plastered and painted. Nonetheless, both of these reinforcements can be used for preventing the hammering or pounding action from one structure to the neighbouring one.

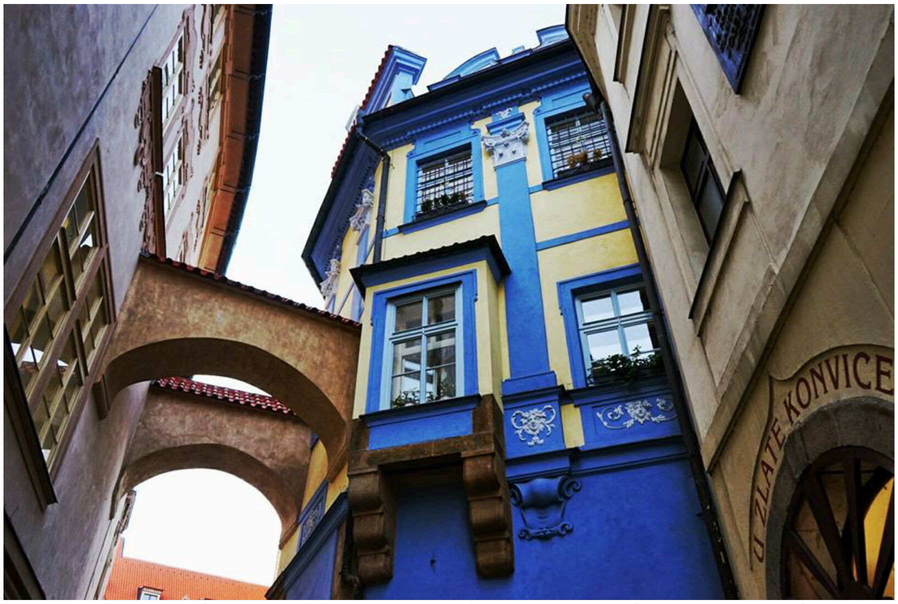


Figure 12: Counter - arch in Prague, Czech Republic (credits: U. Vong).

These seismic mitigation approaches are commonly used in seismic – prone areas. However, each region is more likely to adopt the most suitable seismic – resistant measures associated with its geographical location, and thus resulted in a different and distinctive form. In the following chapter, some of the specific resistant measures implemented by the local community are examined.

3.0 Local Seismic Culture

3.1 Definition and Relevance

Local Seismic Culture (LSC) refers to the seismic resistant approaches and techniques that the local community has adopted over times, regardless of the type of architecture. It can be identified as a common practice in both monumental and vernacular architecture, and its effectiveness against earthquake loading is demonstrated and evaluated after each seismic activity, by the damage pattern or failure mechanisms presented in the structures. Local Seismic Culture was first defined by Ferrigni as “the combination of knowledge on seismic impacts on buildings and behavior in their use, and retrofitting consistent with such knowledge” (p.3). Thus Local Seismic Culture is evolved through trial and error, and closely related to the intensity and recurrence of earthquakes.

As the major building stock in the world, vernacular architecture is always associated with LSC. The present of LSC can be identified when a minimum of three buildings in the region share common seismic resistant features (Correia & Carlos 2015). LSC for masonry buildings consists of several common characteristics in the building typologies and techniques: flexibility, redundancy, and adequate connection details in response to environmental context, climatic condition, available resources and materials (Dipasquale & Mecca 2015). In the following sections, different LSC around the world will be examined, along with the illustrations of structural failure after seismic events.

3.1.1 Local Seismic Culture Mediterranean Region

The region around the Mediterranean, North Africa and Southern Europe is subjected to moderate magnitude earthquakes due to the interaction along the boundary between the African and Eurasian plates. Large magnitude earthquakes are mostly occurred along Hellenides and around Aegean, while the Tunisian part of Northern Africa and the boundary between the Mid Atlantic Ridge and the Strait of Gibraltar are exposed to frequent and medium to high intensity earthquakes. Regarding the vernacular construction, in the Mediterranean region, two seismic resistant techniques can be identified. One technique uses structural elements to counteract the horizontal forces exerted by earthquakes, while the other technique accommodates deformations in order to “metabolize” the forces (Conservation 2014).

For counteracting the horizontal forces in different directions, vernacular buildings in this region often include the application of buttresses and counter – arches. Buttresses, also known as *counterfronts*, are usually made of strong materials, such as brick or stone masonry with rectangular or trapezoidal cross - section. Buttresses were either built at the time of the construction or added afterward; they are visible on the exterior wall of a building to resist the thrust exerted by the load on roof or arches, and the horizontal movements induced by earthquake. Buttresses can also be used to restrain the out-of-plane movement of walls. In Italy, Croatia and southern France, the buttress system has evolved into the form of *loggia* at the base of the buildings. This feature can be seen in Figure 13, where the *loggia* covers the exterior corridor and supported by a series of arches. Hence, the buttress system is used as a structural reinforcement while provides shading area for the entrance as an architectural feature. In Mediterranean historical centers, it is common to detect the presence of stone or brick counter – arches (also known as discharging arches) set between two opposite structures that separated by a distance. Such as in Chios Island and Nisyros Island in Greece. Some cities such as Lunigiana in Italy, instead of using counter – arches as reinforcements, vaults are placed on top of the narrow passageways. By enabling the redistribution of the horizontal movements from earthquake, the connected structures behave as dynamic blocks rather as isolated buildings. To consider this effect, the connection details are important for the local and global behaviour of such systems, as the intended functions rely on the transmission of horizontal forces and movements from the vertical structural elements to these systems, and then further to the ground level. In the Mediterranean region, anchor or *patress* plates are often used. These metal plates are made, typically, of cast iron, sometimes of wrought iron or steel that are connected to a tie rod or bolt. The purpose of the anchor plates is to assemble with the braces for masonry walls against the out - of - plane displacements. These types of connections are also observed at the ends of wooden floor beams in typical historic masonry buildings. The dimension and shape of the end plates is often an element to characterize the local building culture. The dimension of the plate is disproportional to the strength of the masonry while the shape is an architectural feature of the area. There are variety of shapes: square, circle, cross, double C or S, etc. Recommendations of the use of anchor plates were made in some regions. In Provence, it is recommended that using anchor plates at each floor after the 1909 Earthquake (Dipasquale & Mecca 2015; Ortega & Vasconcelos 2015).

The other common technique to increase structural stability that can be seen in the Mediterranean region is the consideration of the center of gravity. In the western Mediterranean, the mass of a structure is concentrated closer to the ground level by using lighter materials in the upper stories, such as timber frame filled with small stones, bricks, or earth blocks. Often the upper stories are built with reducing wall thickness in a combination with the lighter materials, while the ground level walls are made by heavier and compact stone. The ground level is built with vaults as a strategy to lower the center of gravity. This can be seen in south - east Sicily. Wood was used in the upper stories, while masonry vaults were employed at the ground levels of the reconstructed buildings after the 1793 earthquake. Moreover, it is also contributed by the external staircase and buttress at the base of a structure (Dipasquale & Mecca 2015).

Figure 13: Loggias attached to building exterior with arch openings in Ischia, Italy (Alamy, 2013).

Instead of counteracting the horizontal movements, timber elements are used in the structure for "metabolizing" the forces by favouring the deformation demand and thus preventing the propagation of diagonal cracks in walls. Hooping is one of the main systems that use timber as reinforcement. It is inserted at the level above openings and lintels, and usually distributed regularly along the elevation of a multi-storey structure. As illustrated in Figure 14, a wooden hoop is disposed within the load bearing masonry walls horizontally. The arrangement of the wooden hoop is usually composed of two beams placed parallel to the wall, in which one on the inner face and the other one on the outer face. The two parallel beams are connected together by the transversal wooden pieces through nails. The space in between is filled with fragment of brick or stone. However, the dimensions of these elements are different from region to region, as well as the implementation. Places such as in the eastern Anatolia (near Erzurum and Arapgir), the wooden beams are aligned with the edge of the walls, and crossed or overlapped at the corner with scarf joint or half lap joint. They are usually placed at an interval of every 50 – 100 cm along the wall height. Different forms of this timber system are also found in Greece, Turkey, and Algeria. The system in portico is often in the form of chains inserted between capitals and spring lines of arches. For dwellings, wooden ties are located on both sides of the wall, with a diagonal tie at the corner. The other widespread system that employed timber elements is the timber frame system. The timber frame system has been subjected to the influence of the Ottoman Empire – the construction of Ottoman house, can be seen in Macedonia, mountain area of northern Greece, south of Hungary, north – west and central Bulgaria. This seismic – proof structure is based on the use of load – bearing masonry walls on the ground level, and the infilled timber frame for the upper stories. The top of the masonry walls is laced with horizontal timber elements, such as thin boards or squared beams, allowing the timber frame system to be placed easily and secured. The frame is constructed by horizontal, vertical and diagonal members. The space between the members is filled with bricks, adobe, or stone with earthen mortar, depending on the local available materials. There are several variations that derived from the Ottoman house:

himis, *bagdadi*, *muskali dolma*, and *goz dolmasi*. These variations all have a rectangular plan with two or three stories, and timber frame is often left in sight. The substantial differences between these typologies regard to the warping pattern of the frame and the chosen materials for the infill. Scientific researches have validated this construction type is more seismic – resistant than others, such as reinforced concrete and masonry structures, and demonstrated that the failure locations are often coincided with the connections (Dipasquale & Mecca 2015).

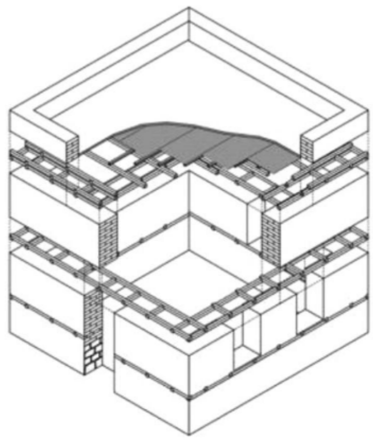


Figure 14: Hooping wooden system (credits: D. Omar Sidik).

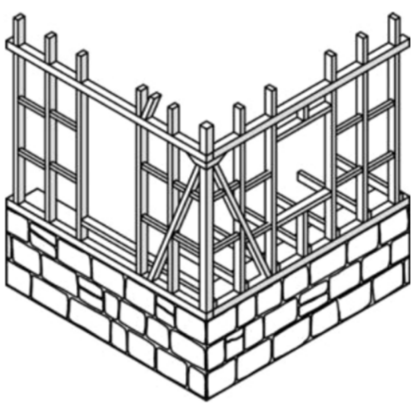


Figure 15: Ottoman house building system (credits: D. Omar Sidik).

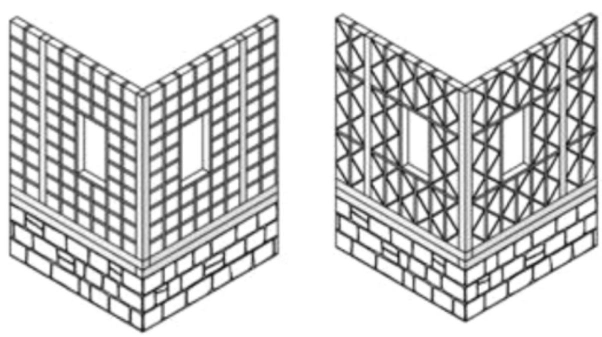


Figure 16: Building system of goz dolmas (left) and muskali dolma (right) (credits: D. Omar Sidik).

In some Greek island, timber frame was firstly introduced as reinforcement, but from 19th century afterward, it is used as part of the new building structure. In Lefjkada, the timber frame system is completely independent from the masonry structure. The frame 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. This secondary timber frame, known as *pontelarisma*, is rather a simple structure that comprised of timber columns and floor beams for the upper story (Dipasquale & Mecca 2015).

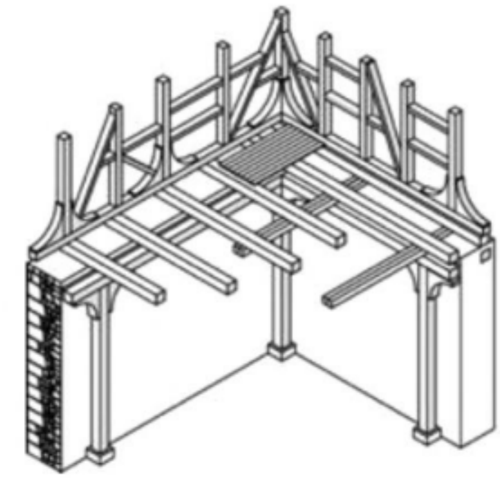


Figure 17: Lefjkada traditional house (credits: D. Omar Sidik).

Before the 18th century, Italian had a similar system called *baracca* in which the timber frame structure was hidden behind the exterior wall and embedded in rubble construction. It was the model for a seismic – resistant system named *la casa baraccata* (Dipasquale & Mecca 2015; Tobriner 1983). The conceptual difference between the two systems is that *la casa baraccata* employs rigorous architectural scheme such as height limitation and “X” bracing elements , in order to create solid connections for box – like behavior using specific elements and features. In the early 20th centrury, the *la casa baraccata* system has proven by the Italian National Research Council (CNR – Ivalsa) and University of Calabria about its seismic – resistance and effectiveness after the 1905 and 1908 Calabria earthquakes. However, in the following decades, this system has not been implemented with the original rigorous scheme, and thus often found failures associated with insecure timber connections (Dipasquale & Mecca 2015).

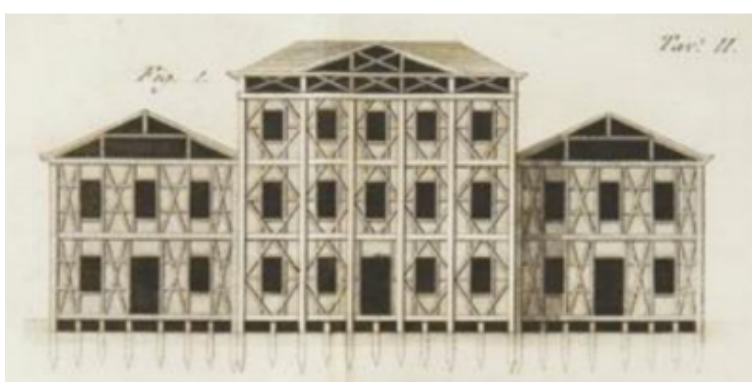


Figure 18: Drawing of casa baraccata (credits: Giovanni Vivenzio).

3.1.2 Local Seismic Culture in Latin America

The cyclical earthquakes in Latin America are responsible to the geographical location of the continent: Latin America lies on the North American, Caribbean, and South American plates, and in contact with Pacific, Antarctic plates, and other minor plates (Cocos and Nazca). Contrary to other countries and cultures, in Latin America, seismic resistant elements and systems employ natural materials, such as plant fiber, earth, wood and canes. These elastic materials are combined together and then inserted in the structural systems. For instance, stick or branches are tied with plant fibers, and combined with earthen mortars for masonry structures to ensure the structural flexibility and improve the deformation tolerance in different directions. The most prominent used seismic – resistant system of this type was wattle and daub building technique. Structures in the seismic – prone areas are designed to be able to move in harmony with the ground, while rigid and heavy structures are absent (Guerrero Beca & Vargas Neumann 2015).

Latin American common seismic – resistant techniques include: foundation platform, meshed system, wooden wall, small window, regular building geometry, low center of gravity and lightweight roofing structure. Some of these techniques are also seen in other Local Seismic Cultures. In the Andean region (such as Peru, Bolivia, Northern Chile and Argentina), some of the houses in the highland were built of stone and earthen materials with circular plan. The circular shape limits stress concentration that presents in the joint of two perpendicular walls; when under seismic loadings, the joint between perpendicular walls tends to turn in different directions and triggers a collapse mechanism. Besides, the foundation platform is similar to the reinforced plinth at the perimeter walls; these platforms are used to prevent the rising damp from groundwater and stabilize the structure in seismic events. Built with compacted superimposed layers, the foundation platform aims to discontinue and divert possible subsoil failure. Also, it is not extraordinary to discover that structures are covered with straw materials, such as grasses, palm leaves that intertwined and tied to the wooden roof structure in order to lower the center of gravity (Guerrero Beca & Vargas Neumann 2015).

One of the most distinctive Latin American seismic techniques in vernacular architecture is the meshed system. This construction technique can be applied to walls, floors and roofing systems. Earth with vegetal materials or wattle and daub with natural fibers are often used. Structures built with this meshed system have an outstanding resistance toward multidirectional stresses as the flexible materials constitute the core of the structural systems and function as a reinforcement mesh. Excessive displacements of these flexible materials are restrained by the confinement of the coating layer. The coating layer, dissipates the energy introduced by the earthquake movement and produce repairable cracks on the walls. The effectiveness of the meshed system can be seen in an example of a house located in Lamas, Peru. The house is built with rustic wattle and daub that made of wood, cane and earth. Even though there are extraordinary large windows, the house was able to survive after the 2005 earthquake. Although this construction technique is typically for a single storey structures, it is still appropriate for structures with two or more stories such as churches, monasteries and public buildings (Guerrero Beca & Vargas Neumann 2015).

3.1.3 Local Seismic Culture in Central and Eastern Asia

The high seismicity of Central and Eastern Asian also flourish different seismic – resistant approach toward the Local Seismic Culture. Similar to the approaches mentioned above in the different regions, the Asian Local Seismic Culture has adopted two approaches that counteracting and absorbing the horizontal movements. On the other hand, there is a third approach that is somehow contradictory to the aspect of Local Seismic Culture in other regions.

The first two approaches characterize the regularity in plan and low center of gravity as equivalent to those seen in other Local Seismic Cultures. The “deformability” approach aims to metabolize the energy produced by the earthquake through displacement, friction and elastic deformation of the structural materials. According to Ferrigni, this approach in the Asian Local Seismic Culture often lighten the structure in order to increase the deformation resisted by friction. When the structure has less weight, it reduces the total amount of “captured” energy by the structure. The “captured” energy, which represents the amount of energy transmitted to the structure by an earthquake, includes (1) metabolized energy – the energy that the structure can accommodate or metabolized through elastic and plastic deformation; and (2) damaging energy – the energy causing structural damages. Reduction of the “captured” energy implies the difference between the amount of these two energies is decreased, thus lowering deformation demand and minimizing the damaging energy. As a consequence, the structure is able to reduce the structural vulnerability and withstand seismic forces.

On the contrary, the “stiffness” approach resists the seismic activity by increasing the amount of the metabolized energy. Structural redundancy or increase the dimensions of structural elements provides a greater metabolized capacity and thus reduce the proportion of damaging energy within the captured energy. The extremity of these two approaches can be seen in the Chinese Hanging Monastery and Bhutanese Taktsang Monastery. As shown in Figure 19 and Figure 20, the Hanging Monastery is made of light wood, while The Taktsang Monastery is built with heavy masonry. The difference in dimensions of opening are also significant: narrow windows in Takstang Monastery but enormous fenestrations in Hanging Monastery (Ferrigni 2015a).

The third approach of Asian Local Seismic Culture is called the “passive” approach. This approach does not contain any seismic resistant measures, but with the acceptance of earthquake damage and ensure safety evacuation for the inhabitants. As some of the regions in the Pacific area are exposed to earthquakes and hurricanes frequently, lightweight materials are used for construction without jeopardizing inhabitants when the structures collapse. Such as the traditional Japanese houses that are built with wood and paper panels (Ferrigni 2015a).

Figure 19: Chinese Hanging Monastery (image source: www.mychinatours.com).

Figure 20: Bhutanese Takstang Monastery (credits: Jay Tindall).

3.1.4 Local Seismic Culture in Himalayas

The Himalayas chain was formed by the collision of continental plates, along the convergent boundary between the Indo – Australian Plate and the Eurasian Plate. This world's highest mountain is one of the most active seismic hazard areas. As western part of the Himalayan Mountain, the areas around the Kashmir Valley have developed their own Local Seismic Culture. The Kashmir Valley is located on the site of a prehistorical lake that created by the uplift of mountains between Indian and Pakistan Administered Kashmir. Over geological time, the lake was silted in and alluvium from the mountains. The soil condition in this area is extremely vulnerable to earthquakes, and thus its Local Seismic Culture takes the soil conditions into account. For example, as the one of the most waterlogged soft soil site, Srinagar, uses clay as the bonding between brick units instead of mortar. Another distinctive strategy for masonry structures that had been used since the 12th century is the timber laced masonry system. As the structures constantly suffer from differential settlement on soft soil and, in a seismic event, the lack of tensile strength trigger either brittle failure from shear forces within the walls or out – of – plane mechanisms. The timber laced system restrains the movement of the walls, creates a uniform and flexible structural system on soft soils. It can also be found in the mountains where soft soils are not an issue. This timber laced system has evolved into two main traditional construction systems after the beginning of the 19th century: *taq* and *dhajji dewari* (Langenbach 2015). *Taq* is a timber laced system that similar to the hoop system. On the other hand, *dhajji* dewari is similar to the timber frame structure seen in the Ottoman houses and Italian *la casa baraccata*.

The term *Taq* or known as *bhatar* in Pakistan, refers to the modular layout of piers and window bays. It was used to define or describe one of the timber laced seismic – resistant systems because t*aq* is the name for the building type in which the system is usually found. The *taq* construction consists of load – bearing masonry walls with horizontal timbers embedment. Similar to the hoop systems, the timber elements in *taq* are tied together like horizontal ladders at each floor and lintel level. Each of them is placed into the masonry walls and secured by the ends of floor joists that penetrate through the exterior walls. This detail confines or prevents splitting within the masonry walls, and at the same time, holding the timber elements in place. Without any vertical reinforcements, the *taq* system is able to resist to the earthquake forces. It is because of, according to Langenbach (2015), the vertical compression resulted from the overburden weight of the masonry. The lack of vertical reinforcements enhances the overburden weight of masonry to act in a vertical manner and without shifting this weight onto the columns buried in the walls (Langenbach 2015).

Figure 21: Taq timber - laced system, Srinagar, Kashmir, India (credits: R. Langenbach).

The other prominent timber lacing seismic – resistant technique, *dhajji dewari*, is similar to the *himis* or other construction types that derived from the Ottoman houses. The terms, *dhajji dewari* comes from the meaning "patchwork quilt wall" in Persian, may also indicates the Persian influence in the origin of this system. This type of constructive technique can be found in both seismic – prone areas, and those that are not affected by earthquakes in which due to economic and efficient use of materials, and reduction of the required amount of masonry work. The *dhajji dewari* system consists of a timber frame that is infilled with masonry, frequency used for the upper stories 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 overburden weight while lowering the center of gravity of the structure. The wall thickness is usually one – half of a brick. The materials used for infill depend on the local available materials and the region; in the Vale of Kashmir, the infill is usually brick that made of fired or unfired clay; in the mountain region of Kashmir extending to Pakistan or regions where soft soils and related settlements are not a concern, rubble stones are often used. Regarding the soil settlement, the *dhajji dewari* is able to hold the walls when the structure has been settled unevenly or out of plumb. However, regarding the pattern or configuration of the *dhajji dewari* system, there is no evidence to demonstrate which pattern performs better in seismic event. Some patterns that are lack of the diagonal bracing members rely on the masonry to provide lateral resistance; there is an increasing research to show the diagonal members are not necessarily, and may even affect the structure in a negative way (Langenbach 2015). Besides, the size of panels does not have significant influence in the structural response toward earthquake, unless the panels are remarkably sizeable and lack of overburden weight. Yet, structures with numerous and small panels has better performance than those with fewer and larger panels, since the seismic behavior rely on the interaction between the timber frame and the masonry (Langenbach 2015). The validity of these issues requires more research in the future.

Figure 22: Building system of dhajji dweari (credits: R. Langenbach).

Another specific timber construction, *cator and cribbage*, can be found in the Himalayan mountains of northern India, northern Pakistan near the Chinese border, and parts of Afghanistan. This robust construction technique is only executed in the regions where wood supplies are abundant. It is a heavier and more timber – intensive version of *taq*, which might be an evolution of the *cator and cribbage* system due to the limited amount of wood used in the construction. Similar to the *taq* system, the timber belts (cators) extend across the walls. However, the corner of the walls consists of a cribbage of timber that filled with masonry. Its variations can also be seen in the Middle East, North Africa and Central Asia, Nepal, Bhutan, Tibet and other parts of China (Langenbach 2015).

Figure 23: Cribbage construction system. The interior is clad with plywood in this photo (credits: R. Langenbach).

3.2 Portuguese Vernacular Construction and Seismic Culture

As Portugal is located in the southwest of the Eurasian tectonic plate, in which near the border of African and North - American plates, Portugal is seismically active and classified as a low to moderate seismic hazard country. As indicated in Figure 24, the SHARE's 2013 European seismic hazard map showing a 10% exceedance probability in 50 years with peak ground acceleration, Portugal is color coded in green to orange. Throughout the history, numerous on and off - shore earthquakes with different magnitudes had impacted the country and its population. The intensity of onshore seismic events is ranged from moderate to large magnitude, while offshore seismic events are ranged from large to very large magnitude. Furthermore, Figure 25 shows the Macroseismic intensity in Portugal. The southern part of Portugal suffers a greater intensity than the northern regions, especially the offshore areas. The intensity scale ranges from destructive to very destructive level, causing most of the buildings damage or collapse when an earthquake occurs. Notice that the highly intense seismic activity areas, the Tagus Leziria, Alentejo, and Algrave regions are within the areas covered by the tectonic faults of Tagus Lower Valley, Messejana, and Loule, respectively.

Since 16th century, the Portuguese continental has suffered severe earthquakes. Table 1 lists some of the most noteworthy earthquakes with MCS intensity that had influenced the Local Seismic Culture in Portugal.

Table 1: Major historical earthquakes in Portugal Continental

Major Earthquake	MCS Intensity
1531 Lisbon	IX
1755 Lisbon	IX
1858 Setubal	IX
1909 Benavente	V
1969 Western Portugal	VIII

The 1755 earthquake was considered as the most significant seismic event in the Portuguese historical seismicity. After the earthquake, new building systems and urban policies were introduced by the awareness of the immense power that a natural phenomenon could result. On the other hand, the seismic resistant reinforcements became a priority in the seismic – prone regions after the 1909 Benavente earthquake. However, these measures were lost until the recurrence of the destructive consequence of the 1969 earthquake happened in Western Portugal (Sousa 2015). Nonetheless, Portugal has developed a distinctive Local Seismic Culture across its continental today.

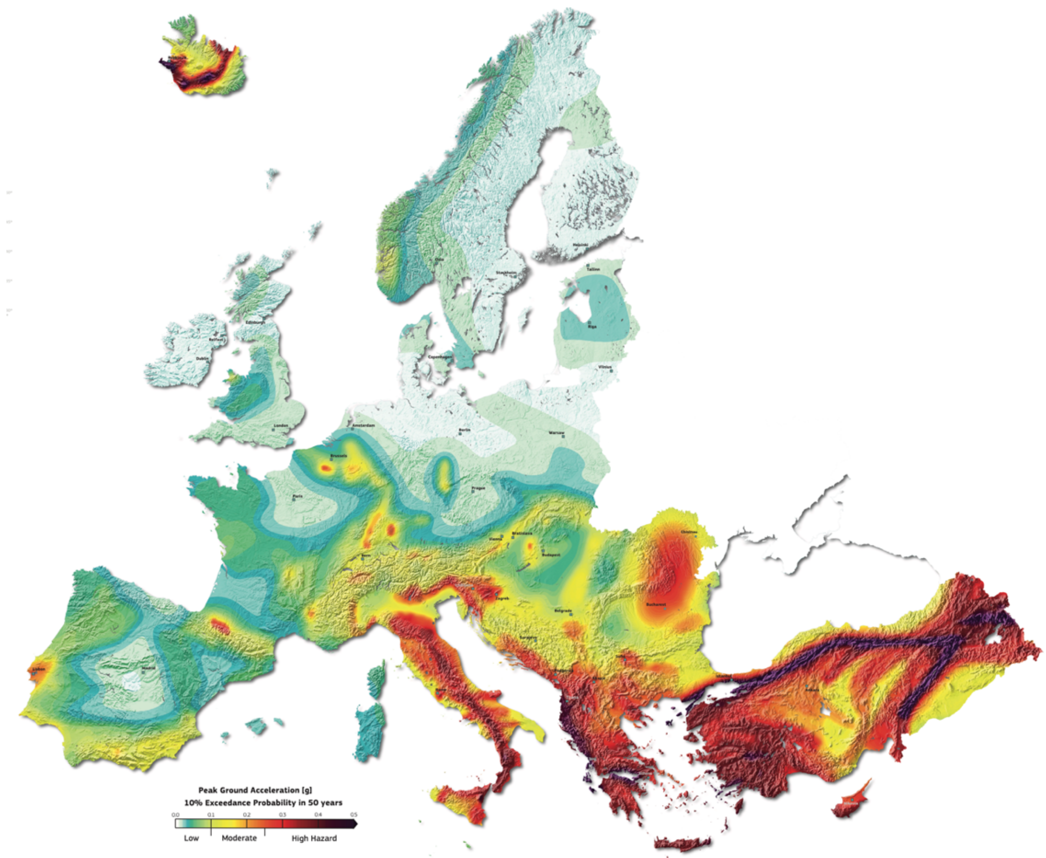


Figure 24: European Seismic Hazard Map. The map displays the peak horizontal ground acceleration to be reached or exceeded with a 10% probability in 50 years, corresponding to the average recurrence of such ground motions every 475 years. Low hazard areas (PGA ≤ 0.1 g) are colored in blue – green; moderate hazard areas in yellow – orange; and high hazard area (PGA>0.25g) in red (credits: SHARE).

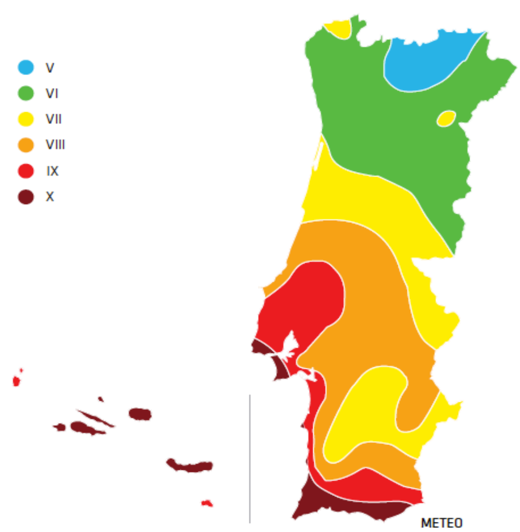


Figure 25: Macroseismic Intensity in Portugal. According to the European Macroseismic scale (EMS) divisions, V represents strong intensity; VI, slightly damaging; VII, damaging; VIII, heavily damaging; IX, destructive; and X, very destructive (Credit: METEO).

3.3 Overview of the Local Seismic Culture in vernacular construction in Portugal

More than half of the Portuguese building stock is comprised of masonry structures. This construction typology is very vulnerable to earthquakes, and is therefore considered as a significant risk to the population even when subjected to moderate events (Vicente et al. 2010). Across the country, local communities had adopted different strategies in order to diminish the losses and costs resulted from earthquakes, especially after the earthquake in 1755. The Local Seismic Culture in Portugal can be divided into six regions as shown in Figure 27, according to the SEISMIC - V project (Lima, Viana, et al. 2015): (1) Tagus Leziria; (2) Coastal Alentejo; (3) Central Alentejo; (4) Lower Alentejo; (5) Algarve; and (6) Azores region. The reconstruction of Downtown Lisbon is also considered in the SEISMIC - V project as an independent case study, since it is considered as the milestone in Portuguese Local Seismic Culture.

Figure 26: Major faults in Portugal Mainland (Credits: IGEO).

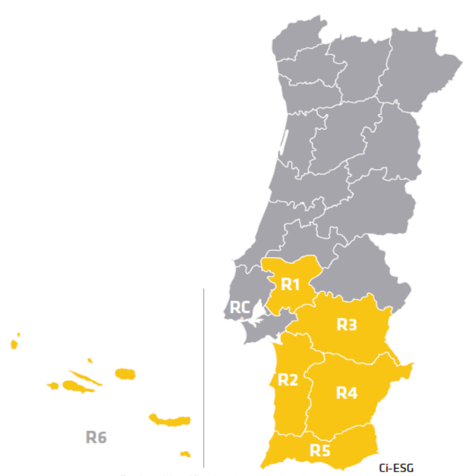


Figure 27: R1 - Tagus Leziria; R2 - Coastal Alentejo; R3 - Central Alentejo; R4 - Lower Alentejo; R5 - Algarve; R6 - Azores; RC - Reference Case of Lisbon Downtown Reconstruction (credits: CI-ESG).

As Lisbon was devastated after the 1755 earthquake, this capital city developed an urban plan proposal that took into account of the improvement of the structural performance and post - disaster recovery. As illustrated in Figure 4, the proposed urban plan is an orthogonal grid of rectangular building blocks instead of consisting cluttered buildings like before the earthquake. Moreover, the facade of each building is restricted to be placed perpendicularly to the usual direction of earthquakes. The building model for Lisbon's reconstruction is known as the Pombaline building. This is the most prominent Portuguese seismic resistant system that named after the Marquis of Pombal. This reactive approach constitutes a construction paradigm to all type of architecture in the Portuguese territory hereafter (Amos & Lourenço 1999).

The ground floor of a Pombaline building was designed to be used for trade or industry, while the three upper floors (later four) was intended to be used for housing. It can be in the form of an isolated or aggregate block. The foundation is constructed by wooden vertical pilings grid and locked by the brick arches at the ground level. The other elements at the ground level were also made of dry stone masonry, such as walls, or columns that support the vaults. Vaults, either barrel or cross, are only used at this level for providing large stiffness at the base and preventing fire propagation to the upper stories. The concern of fire propagation was addressed because the lower part of Lisbon had been reduced to rubble due to the combined action of earthquake and fire in 1755. The infill of the vaults was recycled from the ruins of the earthquake. The exterior walls were made of limestone with lime mortar joints, and the thickness decreases with the elevation of the building. The wall thickness at the ground floor is typically 0.90m. There were multiple reinforcing elements integrated in the walls, such as relieving arches and cornices. For the interior walls perpendicular to the exterior masonry walls, each thickness is about 0.50m and rises continuously from the ground to the roof without any openings. The maximum height of the buildings is limited to the width of the main streets, in order to reduce the effect of out - of - plane collapse mechanism. Elevation of the building has symmetrical and regular composition; the dimensions of opening also decrease with height and no decorative elements are allowed. The cornices of the roof were continuous through different buildings, thus act as a tie strap unified the structures as a whole (Carlos, Correia, Sousa, et al. 2015; Amos & Lourenço 1999).

For considering the seismic action further, the structure above the ground floor was mainly built by wood. There are three wall types existed inside the building: the cage walls (*gaiolas*), the transverse walls (*frontais*, also known as frontal wall), and the non - load bearing walls (*tabiques*). These wood stiffening systems were made of oak or holm - oak with the intention to provide strength and energy dissipation capacity for the structure. The cage walls, *gaiolas*, which was hidden by the wall renderings, were used as a backing structure for the exterior masonry walls. The cage wall system is usually comprised of three main timber elements: vertical (uprights), horizontal (window sills) and diagonal members (cross bracing). *Frechais* is denoted to the timber elements that grouped together to form the *gaiolas* system. The dimension of these rectangular timber elements was usually 0.14 x 0.10 m^2. An example of this system is shown in Figure 28. It was articulated with the timber floor beams forming a three dimensional wooden structure. As the floor beams were recessed in the exterior masonry walls, the cage walls were about 0.05 m away from the internal side of the exterior walls. The other element used for connecting the masonry walls is *mão*. This *mão* element was attached to the cage wall system by metallic devices and embedded in the stone masonry structure. The spaces between the timber elements of the cage walls were filled with rubble masonry with lime mortar; the rubble masonry was made of small stones and ceramic materials recycled from the ruins of the earthquake. The cage walls were often built without any diagonal members because they do not contribute "actively" to the structural support of the masonry walls (Carlos, Correia, Sousa, et al. 2015). The construction of the *frontais* walls was similar to the cage walls, but with different connection details and implementation of diagonal members, as they were used to divide the interior space with the *tabiques* walls. For an isolated building, the frontal walls do not provide significant seismic resistance. The reaction at the base of a frontais wall is about 10% of the shear forces that induced by the seismic action parallel to the wall. When the seismic action is perpendicular to the wall, the resistance of the wall can be neglected. Therefore, the function of frontal walls is highly dependent on the direction of seismic loads and the integration with adjacent buildings (Carlos, Correia, Sousa, et al. 2015; Ramos & Lourenço 1999).

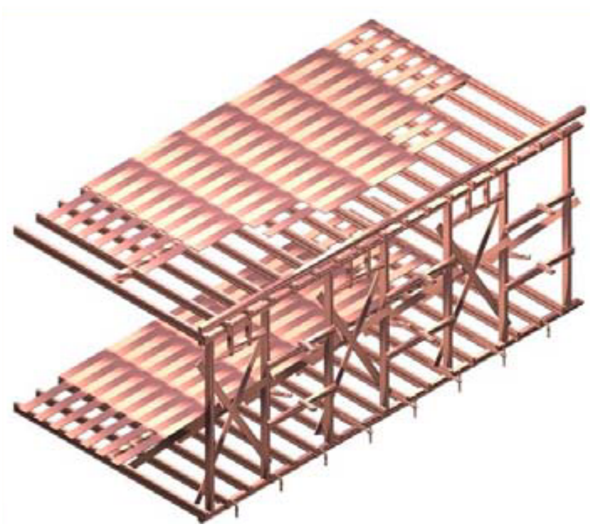


Figure 28: Load Bearing Structure of Pombaline buildings: (left) external cage wall; (right) frontais wall.

Several studies had been conducted regarding to the structural performance of Pombaline buildings, with respect to the properties of the materials, geometry, and connections between various systems present in the structures. It was found that the wooden structure performed well as a locking and energy dissipation system when subjected to an earthquake similar to the one happened in 1755. However, the connection elements *mão* presented a pronounced level of damage in order to ensure the security of the interior, resulting an out - of - plane failure mechanism of the exterior masonry walls. Similar result of the collapse mechanism was also found by Ramos and Lourenco (1999). The wooden system has demonstrated its ability to provide structural stability even when the exterior walls had collapse. However, when a Pombaline block had been modified significantly, the global structural behavior is no longer complied with the current building code, even some of the Pombaline structural systems were still in present (Ramos & Lourenço 1999).

Aside from the Pombaline system, applied in Lisbon downtown, the system was introduced in a small scale in other parts of the country, or including only some particular details of the Pombaline system, the Local Seismic Culture across Portugal has similarities to those seen in other countries and region. Such as the use of buttresses, perimeter reinforcements, light roofing systems, mix techniques and materials in structural systems, simplicity and regularity in plan and elevation. The considerations in the urban scale are also taken into accounts, such as counter arches and seismic – oriented city planning. One of the outstanding examples that illustrated the Portuguese specified seismic culture is Vila Real de Santo Antonio, in the Algarve region. The Algarve region is classified as the zone of high seismic intensity, and in the past, has suffered several devastating earthquakes with induced tsunamis along the coastal area.

Table 2: Major Seismic Events in Algarve (M. Correia & Carlos 2015)

Year	**Mercalli Scale (MCS) Intensity**
1719	X
1722	IX - X
1755	IX - X
1856	VIII
1858	VI - VII
1969	VIII

3.4 The case of Vila Real de Santo Antonio (VRSA)

The followings chapters will focus on the case study of an urban center in Algarve. The selected case was Vila Real de Santo Antonio. It is located presumably at the former border town of Santo Antonio de Arenilha and near the margin of Guadiana River. The current location of Vila Real de Santo Antonio is shown in Figure 29.

The construction of Vila Real de Santo Antonio was begun in 1773. Before that time, this area was already abandoned and devastated by the tsunami after the 1755 earthquake. The current location was chosen because of its strategic position for potential economic revitalization and political display, as located at the extreme south of Algarve facing the Spanish border. This city was intended to boost the Algarve local community through industries, taxes and customs controls. The construction was finished within a few months, using pre - fabricated and standardization techniques and processes that similar to the reconstruction in Lisbon (Ortega et al. 2009a).

Figure 29: Location of Vila Real de Santo Antonio, enclosed in red rectangle (Google Maps, 2016).

This new planned town followed the Pombaline plan: orthogonal plan consists of rectangular blocks that organized around the central square, Praça do Marquês de Pombal; the facade of each block is strictly aligned and four distinctive architectural types can be found, except for the *Alfândega* building, known as the Customs House, the church and the towers at the corner of the central square. The four distinctive architectural types are: (1) waterfront buildings, which are two - stories buildings with attics located beside the river; (2) square buildings that surround the central square; (3) single - storey dwellings that consists of different typologies, but are characterized by the similarity in scale and geometry; (4) one - storey buildings with a patio that used for commercial purpose, such as the salting factories and warehouses that located behind the waterfront buildings.

The buildings in the city were similar to the Pombaline buildings. Stone was transported from the neighbouring quarries and used in foundation and exterior walls. Walls are plastered with lime and sand, and whitewashed. Solid brick masonry was used for the partition walls at the ground level; some of the buildings include vault ceiling as a fire prevention measure. While timber elements that used for roofing and flooring were prefabricated and imported from Lisbon. The single storey dwellings only comprised of masonry walls with the roofing system resting on the top. Having said that, the only seismic resistant measure that had been developed in the reconstruction of Downtown Lisbon, the frontal wall system that filled with light masonry is applied in the buildings at Vila Real de Santo Antonio. Since most of the buildings were low – rise, *gaiola* or cage wall system were not implemented. Besides, most of the buildings in Vila Real de Santo Antonio had been undergone interventions in the past. Only 155 - 164 buildings still possess the characteristics of Pombaline building, such as the structures located at the main square and beside the river. The Alfândega building is one of well-preserved buildings that includes most of the original structural elements (Ortega et al. 2015).

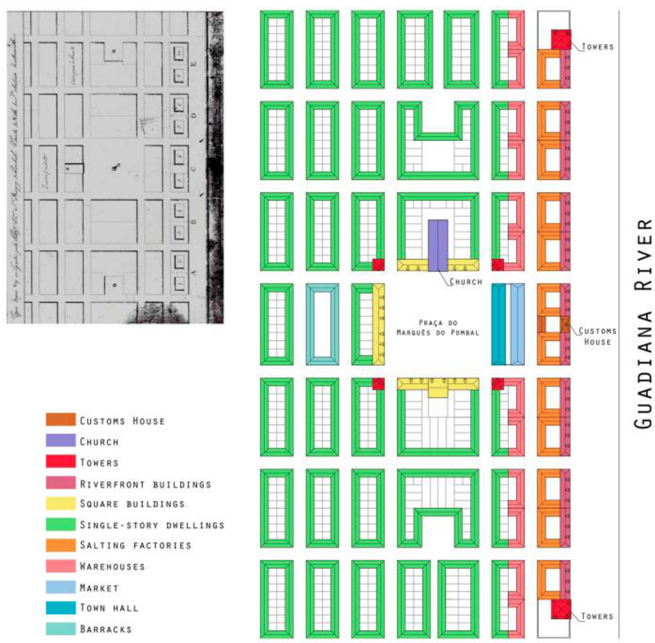


Figure 30: (Left) Initial new city plan in 1774; (right) original plan of Vila Real de Santo Antonio (credits: J. Ortega).

3.4.1 Original Construction of the *Alfândega* block

The *Alfândega* block, or the Custom House is located at the center position that facing the Guadiana River. The block consists of five buildings: two Waterfront Buildings; one Central Building that was used for customs; and two U – shaped Salting Factories with patios at the rear. The Waterfront Building is two stories with rigorous elevation with dormer windows on mansard roof. The ridge of the building was aligned with the façade of the Central and the other Waterfront Building. The Central Building, which was a two independent two stories buildings pronounced its central position by emphasizing the difference in height. The building that aligned with the Waterfront Buildings was slightly higher in elevation in order to maintain the rigorous architectural scheme, while the one at the rear was significantly higher than the one story Salting Factories. A patio was in between these two independent buildings. The Salting Factories, enclosed the Alfândega block as arcades. The roof of the Salting Factory was constructed by rafters resting on the masonry walls and timber joists, and its ceiling was covered by wooden boards. On the other hand, the Central Building employed a more complex timber roof frame structure, along with the simpler timber truss system in the Waterfront Buildings. Regional ceramic tiles were used for roof covering. Timber elements were also used in other structural systems. Floors are composed by timber beams, wooden floor joists and boards. These timber elements were prefabricated and imported from Lisbon. Inside the buildings, partition frontais walls were found in the upper level connecting other structural systems, such as roofs, floors and exterior walls. Having said that, the partition walls at the ground level were constructed by the heavier material, solid brick masonry, while the exterior walls were built of stone masonry.

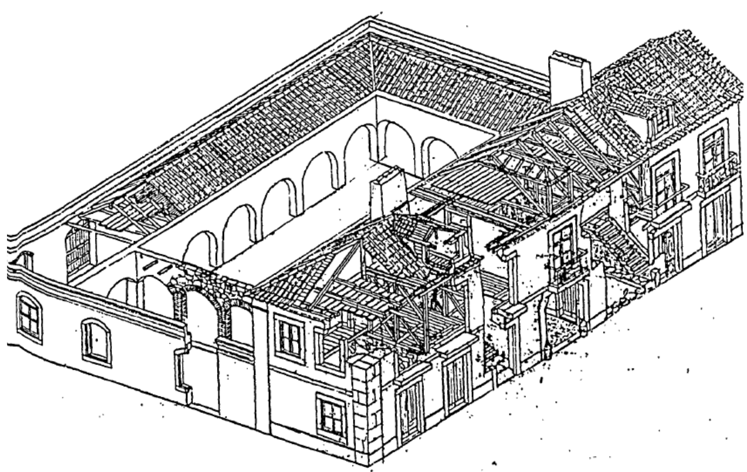


Figure 31: (Left) Construction Details of Waterfront Building and Salting Factory (credits: Mascarenhas, 2004).

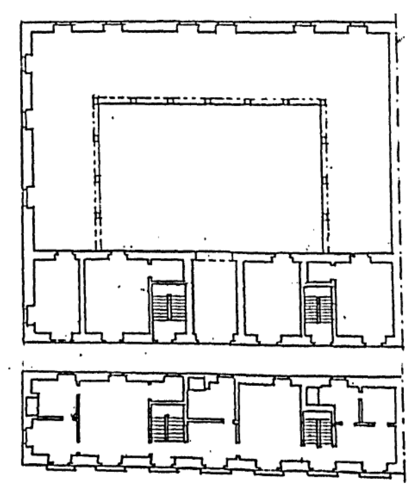


Figure 32: (Top) Original ground plan of Waterfront Building and Salting Factory; (bottom) first floor plan of Waterfront Building (credits: Mascarenhas, 2004).

3.4.2 Current Condition of the *Alfândega* block

In the past, the structure of the Alfândega block had been modified to different configurations. Most of the original characteristics are preserved in elevation of the Central and Waterfront Buildings. However, in terms of volume, the Alfândega block was completely different compared to its original configuration. The alternations were mainly done in the 19th and 20th century. As illustrated in Figure 33 and Figure 34, the alternations include: introduction and enlargement of openings; additional floors, structural elements and constructions, such as extensions and new buildings in the patios; substitution or reconstruction of original roof; extension of parapets; compartmentalization of interior space; and replacement of original buildings. These alternations introduced irregularity and expected new seismic – vulnerable elements into the structure, especially the difference in height among the old and additional buildings and the location and alignment of openings. Moreover, according to a recent architectural survey, the load – bearing masonry walls are composed of units with highly irregular size and shape, bonded by mortar with pieces of bricks and rubble; no horizontal courses were observed in the masonry walls. The morphology of the walls is most likely to be two – leaf with rubble infill, which can be found in other buildings within the city. The wall thickness is varied between 0.60 to 0.70 m. The timber beams of the floor structure are either circular or rectangular cross section with diameter ranging from 0.15 to 0.30m. The timber beams in the Central building are perpendicular to the façade, resting on transversal timber plates. Other timber beams in the building pierce into the stone masonry walls. Nonetheless, these timber elements were in good condition. For the partition walls in the Central Building, the used materials do not seem to be original and functioned as intended. Some of the current conditions are shown in Figure 35 and Figure 36 (Ortega et al. 2009a). The current state of the Alfândega block exhibits the loss of seismic awareness in the region, which might further lead to compromise or abandonment of the Local Seismic Culture the region, has developed.

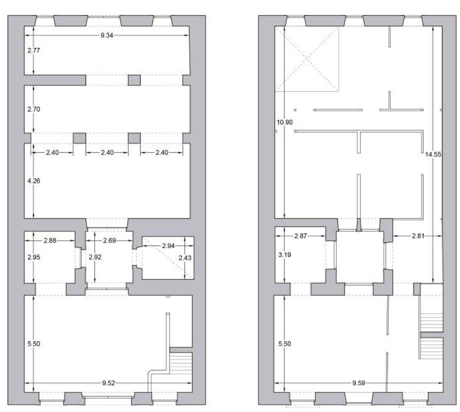


Figure 33: Current plan of the Central Building; (left) ground floor; (right) first floor. Dimension in meters (credits: J. Ortega).

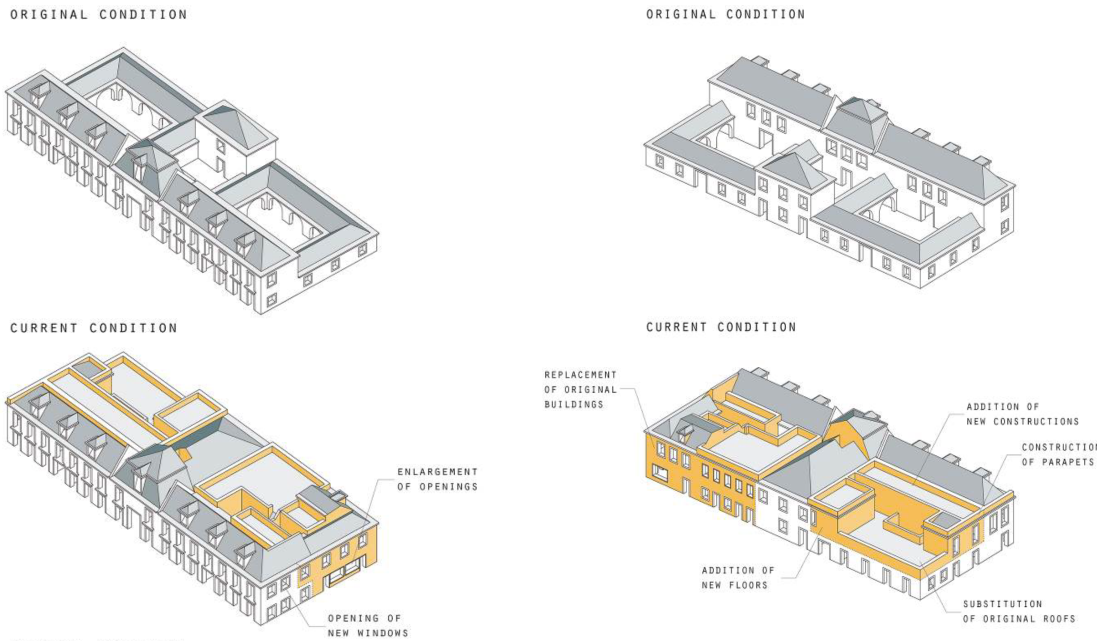


Figure 34: Original and Current State of Alfandega Block; alternations are highlighted in orange (credits: J. Ortega).

Figure 35: Current Condition. (From left to right) timber partition wall; new gable at the back of Central Building; and reconstruction of mansard roof (credits: J. Ortega).

Figure 36: Current state of Central Building. (Left) Morphology of stone masonry wall; (right) timber floor beams on transversal timber plate (credits: J. Ortega).

4.0 Numerical Analysis of Seismic Behavior of the *Alfândega* Block

This chapter deals with the numerical analysis of the *Alfândega* block in its original geometric configuration, aiming to achieve a better understanding of its seismic behavior, assess the major vulnerabilities and provide insights for seismic risk mitigation measures. For these reasons, it was first decided to assess the behavior of three individual buildings that constituted the building aggregate, then followed by an evaluation of the aggregate in a global behavior. Total of four numerical models were constructed based on the historical documentation and a recent material survey (Ortega et al. 2009b; Mascarenhas, 2004).

The first three models are, the Waterfront Building, the Central Building, and the Salting Factory. However, in block assemblages these different buildings interact and their seismic behaviors are influenced by the interrelationship between the adjoining buildings. The fourth model was intended to simulate a more realistic behavior of the *Alfândega* Block as an aggregate. Modal analysis was performed initially to determine the mode shapes and natural frequencies of each model. Pushover analysis was carried on for assessing the seismic vulnerability that associated within each structure. A comparison between the individual buildings and the complete block as an assemblage of the different buildings is provided at the end with respect to their seismic capacity.

4.1 Numerical Model of Alfândega Block

The numerical models were constructed with DIANA software (TNO 2009) taking into account the geometric details of the original drawings and the loads associated to its normal use. Three types of loading were applied in the models in addition to the self – weight of the materials: live load of 2 kN/m^2; floor load of 1 kN/m^2; and roof load of 0.5 kN/m^2.

The structural systems such as the exterior walls, window and door frames, and arcade were modelled using three-dimensional finite elements (CTE30 ten–node, three–side isoparametric solid tetrahedron element with a four–point integration scheme over volume). The timber beams simulating the flexible floors were modeled with beam elements (CL18B element, which is a three–node, three dimensional Class III beam element with a two–point Gauss integration scheme along the bar axis). As aforementioned, four numerical models were built: (1) Salting factory (with 136 639 nodes and 78 398 elements); (2) Central Building (with 200325 nodes and 121 103 elements); (3) Waterfront Building (with 169 318 nodes and 102 311 elements); and (4) the aggregate block (with 172 453 nodes and 92 201 elements).

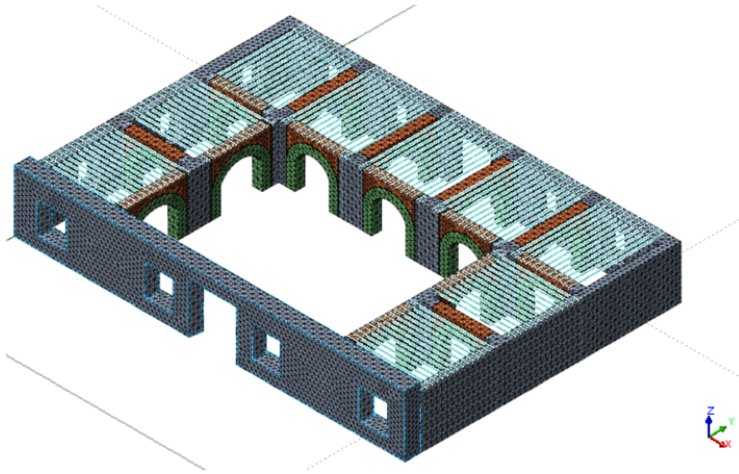


Figure 37: Numerical model of isolated building - Salting Factory.

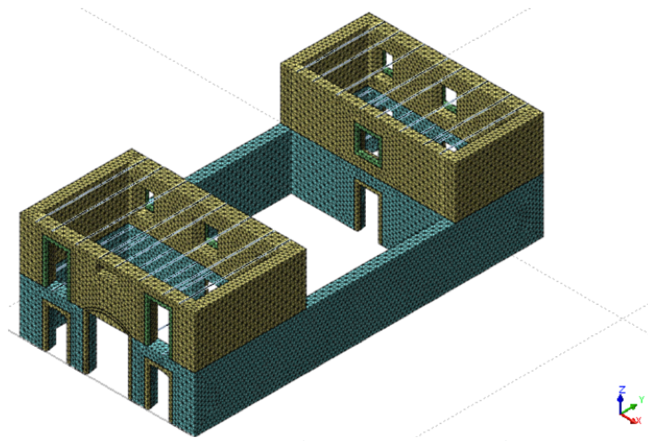


Figure 38: Numerical model of isolated building - Central Building.

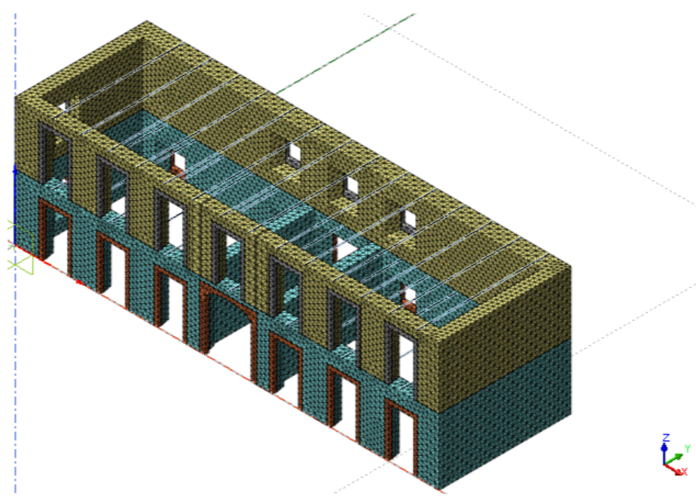


Figure 39: Numerical model of isolated building - Waterfront Building.

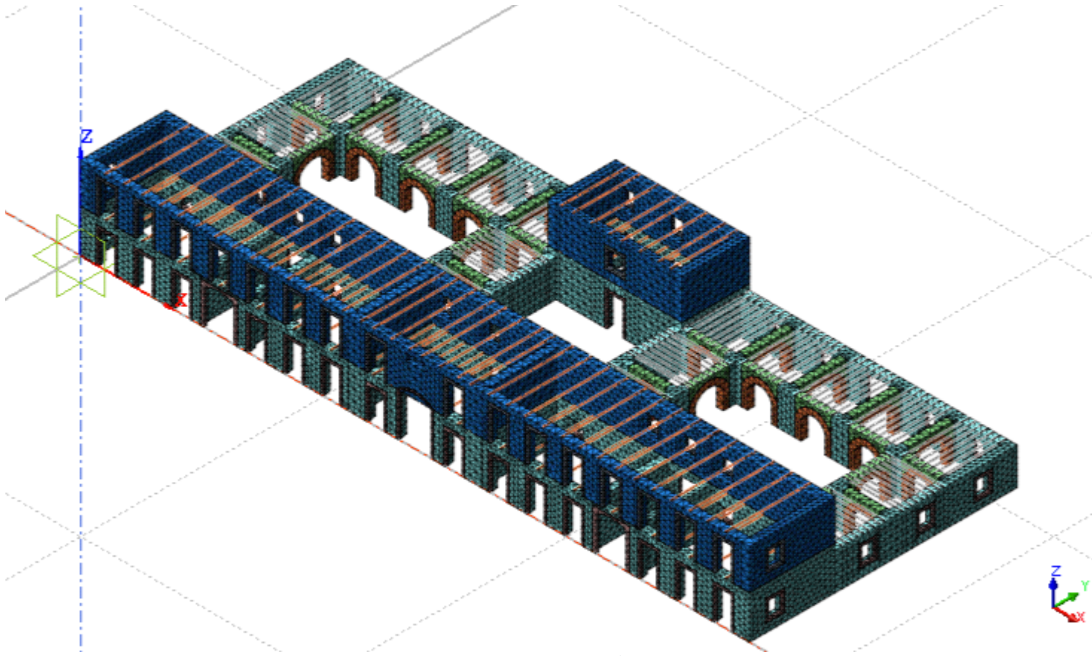


Figure 40: Numerical model of aggregate block.

The average element size for each isolated building is 0.25 m, while the aggregate block has an average element size of 0.45 m, as it was considered reasonable given the dimension of the complete building. For each model, three directional translations were restrained at the base assuming the base completely fixed. The shared walls in each isolated building were constructed with the full thickness. The locations of opening at the back of the Waterfront Buildings were assumed due to the variations in the available documentation. Regarding the interior partition walls, there were no enough evidence about the locations and mechanical properties of the materials. Although it is conservative, the partition walls were not included. The number and the size of the beams in the Salting Factory are more and smaller than other buildings. Different cross sections were observed on site, but for the simplicity of the numerical models, all the beams are assumed to have a rectangular cross section. The overall dimensions are 0.2 m x 0.2 m and 0.25 m x 0.25 m in the Salting Factory and the other buildings, respectively. Even though the effectiveness of the beam connections after two and a half centuries is questionable, any analyses without the floor system is conservative (Ramos & Lourenço 1999). However, from a recent evaluation and assessment on the current state of Central Building based on the dynamic identification and experiment modal analysis, it was observed that the floor system should be taken into account in terms of the numerical and experimental frequencies (Ortega et al. 2009b).

4.2 Simplification of the *Alfândega* Block Model

One of the major issues discussed at the beginning of this work was the type or size of elements to be used in the numerical simulation of the aggregate block due to its dimension. When using an average element size of 0.25 m, the numerical model for the aggregate block was large and expected to require extended amount of computation time. Therefore, different possibilities were initially considered: (1) use of shell elements; (2) with the consideration of the different interactions among the adjoining buildings, simplify the model by replacing the Salting Factory as simple supports; (3) by taking advantage of the symmetry of the aggregate block, reduce half of the complete model; and (4) reduction of the number of elements through wall thickness.

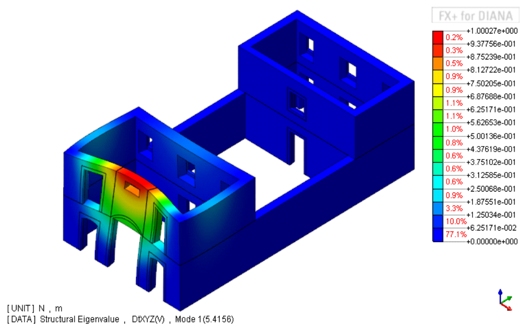
Among the different possibilities, it was decided to perform a sensitive analysis on the reduced number of elements through the thickness of the walls, aiming to maintain the true boundary conditions and avoid to produce new geometrical models (in the case of shell elements). The Central Building was selected, with element sizes of 0.30 m, 0.45 m, and 0.85m. The comparison is based on the modal analysis of each element size, and the criteria are related to the discrepancy in mode shape and frequency corresponding to the first mode.

For each element size, the absolute maximum displacement was 1.00 m, and the first mode was associated with the out–of–plane movement of the façade. As expected, the difference in frequency increases when the number of elements in thickness is reduced. Yet, each variation is relatively small; the percentage of error is within 5.0%. Among the three element sizes, the minimum difference to the reference element size was found in the model with element size of 0.30 m, but it only reduces the amount of elements by 40%. On the other hand, when one element is used within the thickness, the amount of elements reduces by about 90%, which is the most desirable in terms of computation time. Nevertheless, considering the reduced amount of elements and the associated accuracy in the analysis, it was decided to use two elements within the wall thickness for modelling the aggregate block.

Table 3: Element size comparison with respect to frequency and number of elements

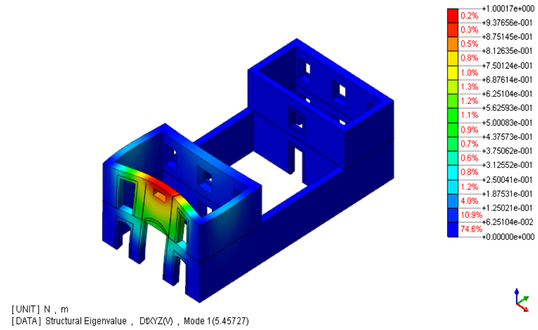
Element Size [m]	Number of Element in thickness	Frequency - Error [%]	Element – Reduced Amount [%]
0.30	3	0.74	39.5
0.45	2	1.48	78.6
0.85	1	3.69	89.6

Size – 0.25 m (4 elements in thickness)



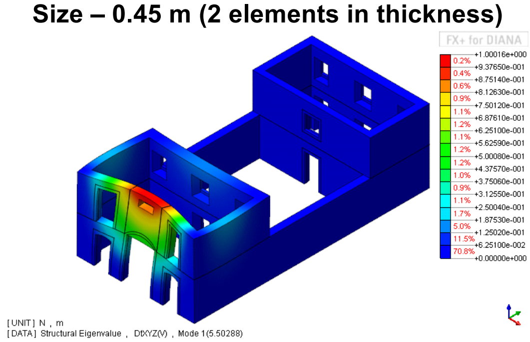
First mode: 5.42 Hz
Absolute maximum displacement: 1.00 m
Elements: 127 343
Nodes: 208 990

Size – 0.30 m (3 elements in thickness)



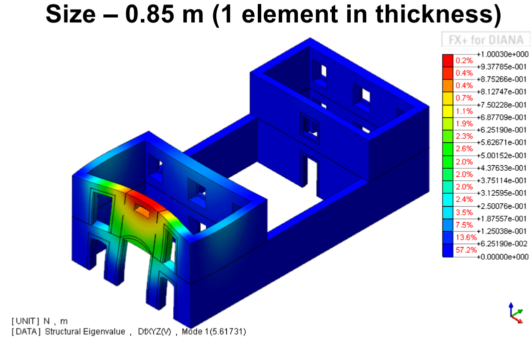
First mode: 5.46 Hz
Absolute maximum displacement: 1.00 m
Elements: 77 096
Nodes: 131 147

Size – 0.45 m (2 elements in thickness)



First mode: 5.50 Hz
Absolute maximum displacement: 1.00 m
Elements: 27 296
Nodes: 49 422

Size – 0.85 m (1 element in thickness)



First mode: 5.62 Hz
Absolute maximum displacement: 1.00m
Elements: 13 254
Nodes: 25 703

Figure 41: Comparison between different element sizes in Central Building.

4.3 Material Models and Mechanical Properties

In terms of the material properties, good quality of stone masonry was considered for the exterior walls, with a stronger material assumed to be used for frames. During the in–situ survey (Ortega et al. 2009b). it was observed that arches were made of brick masonry. The infill material was modeled as coarse rubble masonry, since it is common to be found in historical structures. In relation to the timber floors, there was no available information about the used species. Oak wood was considered because it was often used in the Pombaline buildings. The elastic material properties for the materials of the different constructive elements are presented in Table 4. These values were adapted from the available literature (Ortega et al. 2009b).

Table 4: Material Elastic Properties

Material	Young Modulus, E [MPa]	Poisson's Ratio, ν	Specific Weight, γ [kN/m^3]
Stone Masonry	2000	0.2	20
Frame	2300	0.2	20
Brick Masonry (Arch)	3600	0.2	18
Infill Masonry	300	0.2	20
Timber	10 000	0.2	6

For the nonlinear static analysis, the total strain rotating crack model was adopted for all masonry types by assuming homogeneous and isotropic materials. The adopted material behavior models under tension and compression are shown in Figure *42*, in which exponential softening behavior in tension and parabolic softening in compression. The chosen material properties for the nonlinear analysis are summarized in Table 5, based on the available literature (Ortega et al. 2009b).

Table 5: Material Non-linear Properties

Material	f_c [MPa]	G_{fc} [MPa]	f_t [MPa]	G_{f1} [N/mm]	β
Stone Masonry	2.0	3.2	0.2	0.012	0.05
Frame	2.3	3.68	0.23	0.012	0.05
Brick Masonry (Arch)	3.6	5.76	0.36	0.012	0.05
Infill Masonry	1.0	1.6	0.1	0.012	0.05

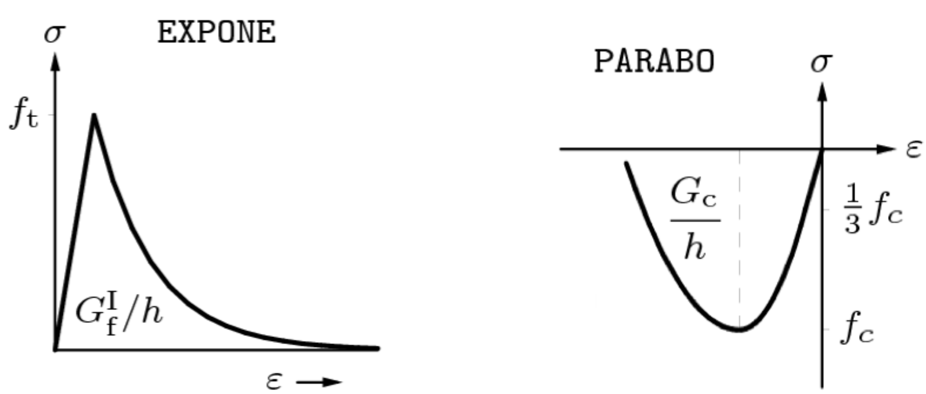


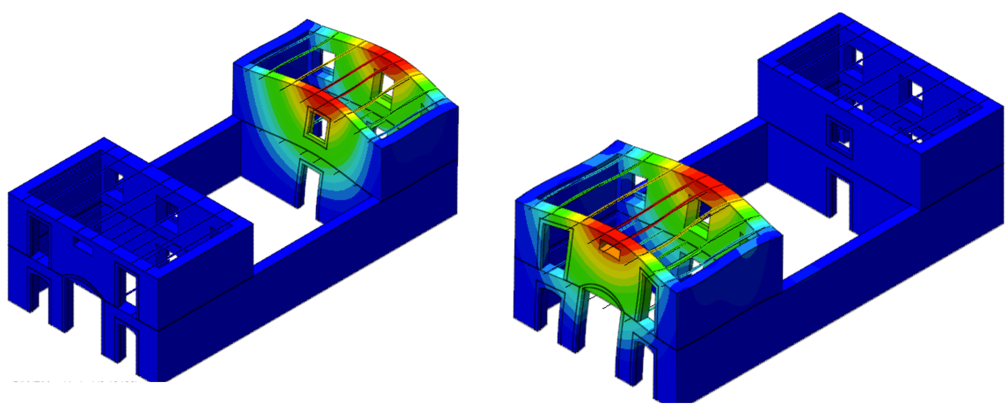
Figure 42: Nonlinear material laws for masonry: (left) exponential in tension; (right) parabolic in compression (credits: MIDAS FEA).

4.4 Modal Analysis

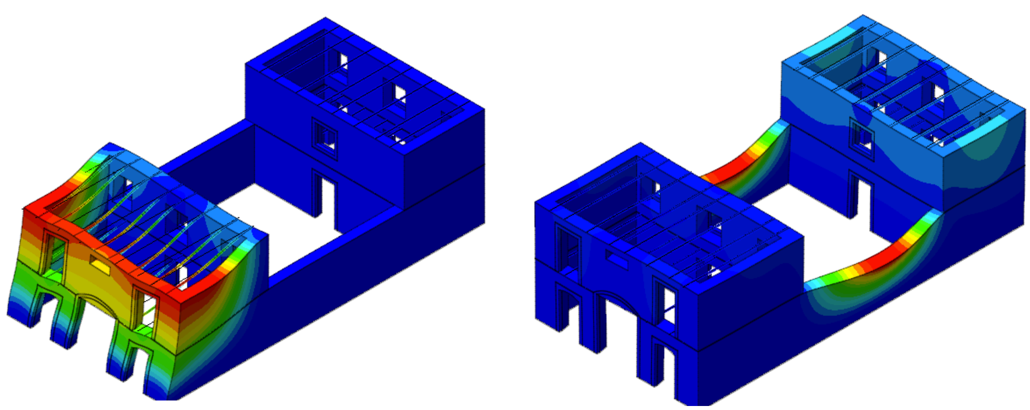
As a preliminary analysis of the individual buildings, modal analyses were considered for the aims of evaluating the dynamic behavior of the buildings and anticipating probable locations of the vulnerable elements.

4.4.1 The Central Building

The obtained frequencies and mode shapes of the Central Building are presented in Figure 43. The first and second mode shapes have similar frequencies and both undergo translation along the longitudinal direction. The displacements are dominant either on the front or rear block of the Central Building. When the structure vibrates in either of these modes, the timber beams do not prevent the out-of-plane translation but enables the connection between two parallel walls, causing out–of–plane deformations on both the front and back walls. The third mode exhibits a torsional effect by the transversal deformation on the front block. The beam elements restrain some of the horizontal movement. The fifth mode has a frequency of nearly double of the first or second mode, and the mechanism is governed by the out–of–plane movement of the walls connecting the front and the rear block. Some displacements are also experienced at the upper level of the rear block. Nonetheless, as these connecting walls have no transversal resistance, they exhibit local deformations when the structure is subjected to this frequency.



Mode 1 (f = 6.424 Hz) Mode 2 (f = 6.748 Hz)

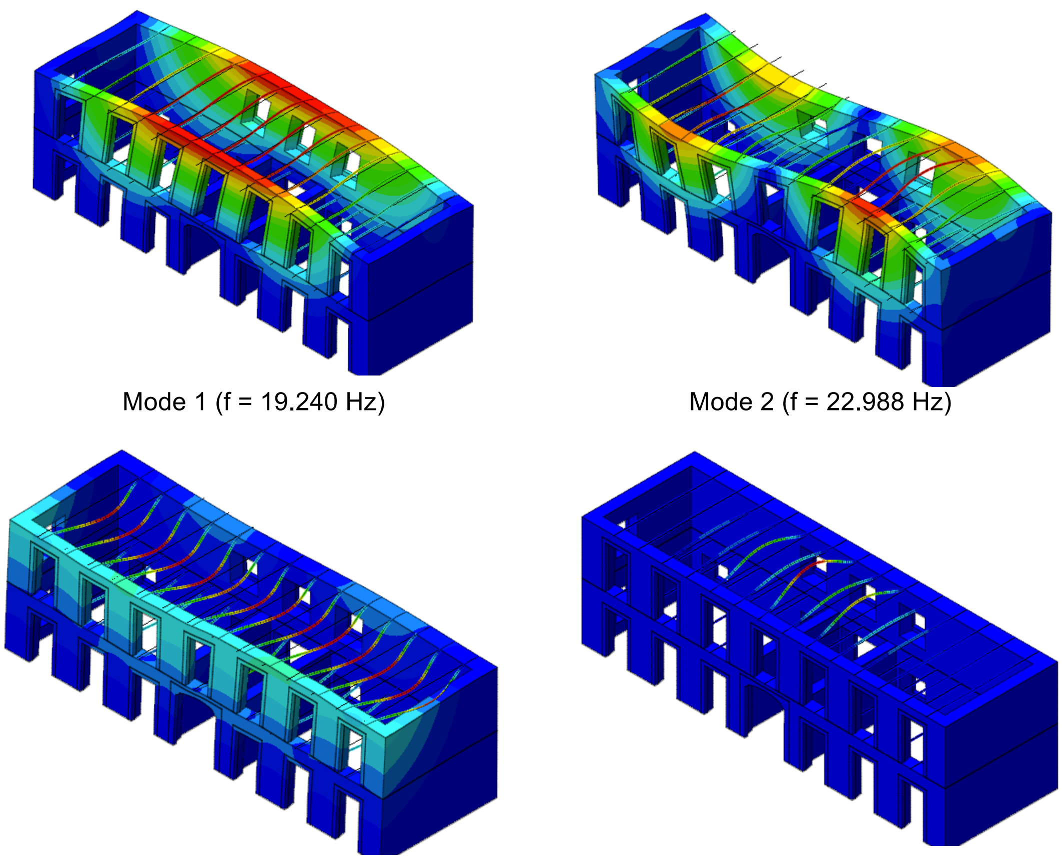


Mode 3 (f = 8.370 Hz) Mode 5 (f = 12.203 Hz)

Figure 43: Modal analysis of Central Building.

4.4.2 The Waterfront Building

The first mode of the Waterfront Building has a frequency of 19.2 Hz. The translation in the transversal direction is not restrained by the timber beams, as illustrated in Figure 44. The first mode shape corresponds to out-of-plane translation of the façades, with larger deformation at the mid span. As there are three openings in the middle of the back wall, the stiffness of the back wall is not uniform. The second mode has a frequency of 23.0 Hz. This mode is associated with out–of–plane deformations in the transversal direction on both façade and back walls. The two interior masonry walls at the ground floor restrain the movement in the middle, and thus the walls at this location remain unaffected on the upper level. This, in turn, causes different directions of the out – of – plane movement on the rest of walls; on the left side of the building, the facade exhibits an out–of–plane deformation outward, while on the right side, the wall deforms inward. The fourth and fifth mode are associated with high frequencies, 24.4 and 26.0 Hz respectively. The fourth mode is dominated by the translation in the longitudinal direction. The structure does not exhibit any significant torsional effect in the lower modes.



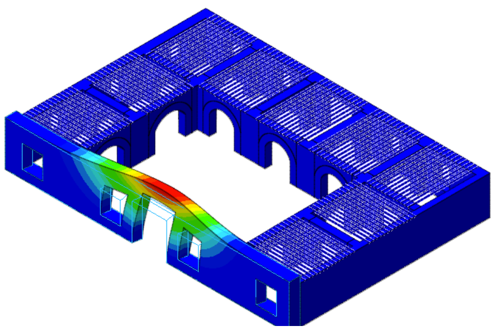
Mode 1 (f = 19.240 Hz) Mode 2 (f = 22.988 Hz)

Mode 4 (f = 24.357 Hz) Mode 5 (f = 25.962 Hz)

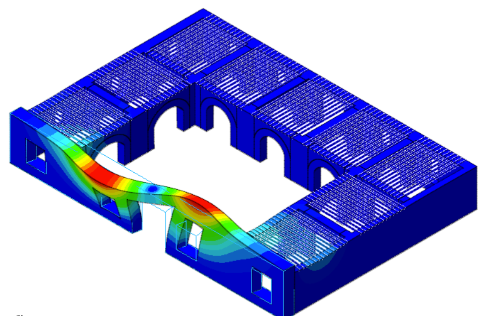
Figure 44: Modal analysis of Waterfront Building.

4.4.3 The Salting Factory

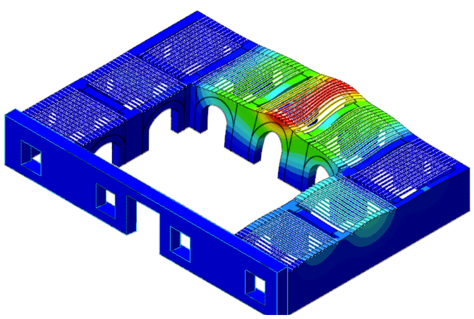
From the modal analysis, it was observed that the first three modes of the Salting Factory are associated with local deformations, as shown in Figure 45. The effective mass in each of these modes is relatively small, involving only specific structural elements at a time. The fourth mode with a frequency of 17.4 Hz is dominated by deformation in the direction perpendicular to the shared wall of the Waterfront Building. Although there is some deformation at the side, the middle bays in the arcade are highly affected by this vibration mode since there are no structural elements to restrain the movement. The fifth mode, with a frequency of 17.8 Hz, is governed by the deformation at the right side in the direction parallel to the shared wall. Similarly, no elements restrain the movement in this direction, these two bays exhibit significant out–of–plane deformations compared to the rest of the structure.



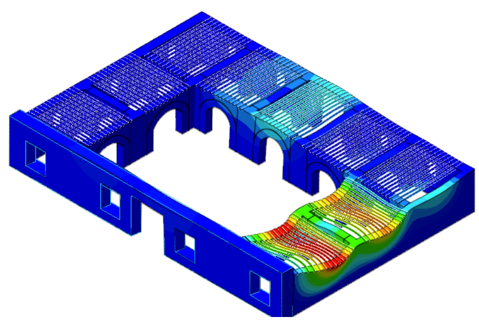
Mode 1 (f = 10.314 Hz)



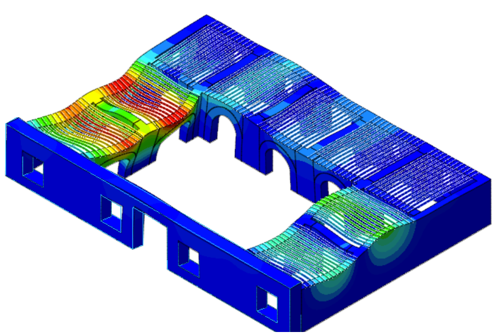
Mode 2 (f = 15.439 Hz)



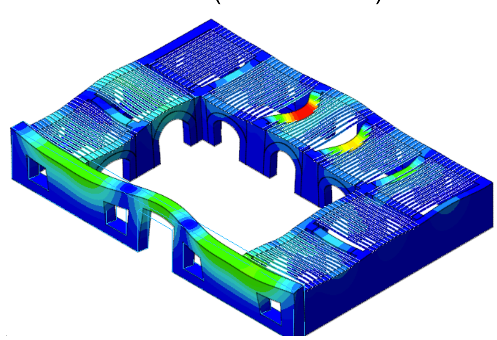
Mode 4 (f = 17.442 Hz)



Mode 5 (f = 17.760 Hz)



Mode 7 (f = 18.846 Hz)



Mode 10 (f = 20.564 Hz)

Figure 45: Modal analysis of Salting Factory.

The seventh mode is also controlled by the movement parallel to the shared wall. However, due to the openings on the left perimeter wall, the structure is more affected than the fifth mode. The tenth mode is mostly associated with the out–of–plane deformation at the top of the shared wall, and the infill materials in the central bay due to the door opening at the rear.

4.4.4 The Aggregate Block

The first mode of the aggregate block is associated with the out – of – plane deformations in the Waterfront and Central Building. The rear block of the Central Building and the Salting Factory remain intact. The fourth mode, with a frequency of 9.10 Hz, involves lateral deformation in the +Y direction and shows that the Central Building is more affected. Notice that in this mode the masonry walls at the ground level of the Waterfront Buildings influence the mode shape by resulting two directional out–of–plane displacement similar to the analysis of the isolated building. The seventh mode of the structure is related to the translation in the +X direction, combined with a certain torsional effect on the part of the structure on the façade, at both lower and upper stories. With the different geometry and high concentration of openings at the façade, the corners of the Waterfront Building have larger movements compared to the middle of the aggregate block.

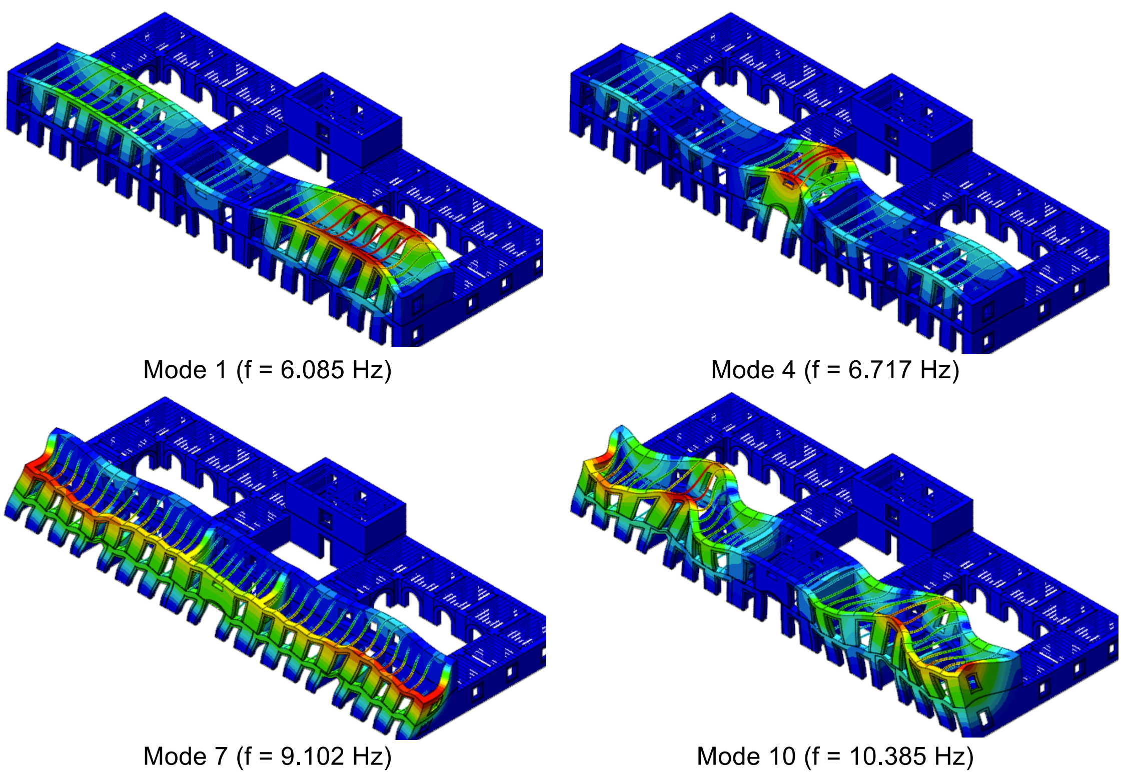


Figure 46: Modal analysis of aggregate block.

Moreover, the tenth mode with a frequency of 10.4 Hz is associated with the part of the structure on the façade and exhibits a torsional effect on both sides of the Waterfront Building, while the Central Building remains unaffected.

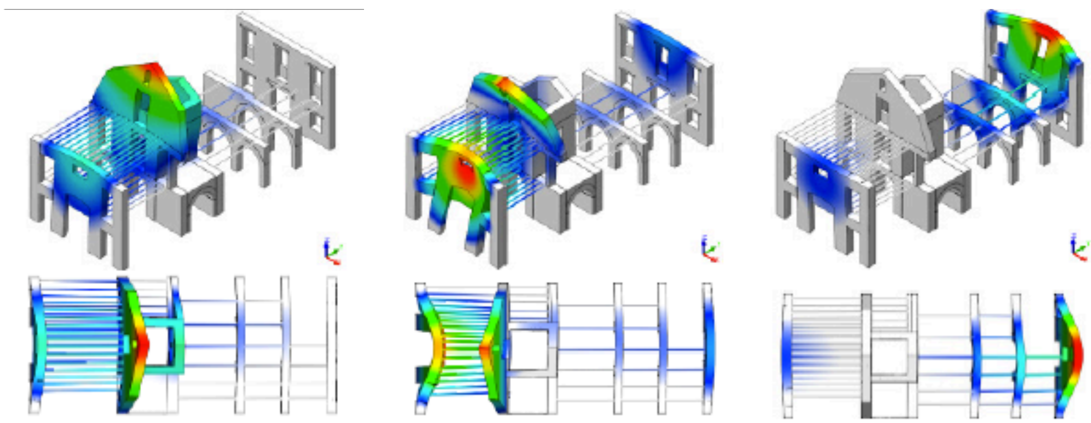
4.4.5 Discussion of Results

The mode shapes of the aggregate block are contributed by the interaction between each adjoining building, namely the Waterfront and the Central Building. The first, fourth and seventh modes of the aggregate block exemplify the interaction by combining the mode shapes that have been seen in the isolated modal analyses. In the first mode, the deformed shape on the façade is similar to the second mode of Central Building (with a frequency of 6.75 Hz) and the first mode of the Waterfront Building (with a frequency of 19.24 Hz). However, the frequency of this mode, 6.09 Hz, is lower than the frequencies of the associated mode shapes. The difference suggests that the change in geometry and thus the aspect ratio in the horizontal directions predominate the dynamic response of the structure. In the +X or the longitudinal direction of the Waterfront Building, the length to width ratio increases dramatically and thus the structure subjected to the same mode shape at a lower frequency. On the other hand, since the ratio in the Central Building remains unchanged, the frequency of the associated mode shape is similar to the frequency of the aggregate block. This is also demonstrated by the fourth mode of the aggregate block. The mode shape is a combination of the second mode of the Central and Waterfront Buildings, with a frequency of 6.75 and 23.0 Hz, respectively. The fourth mode of the aggregate block has a frequency of 6.72 Hz, which is close to the obtained frequency in Central Building.

The seventh mode of the aggregate block illustrates this correspondence, but the resultant frequency (9.10 Hz) is higher than the frequency of the associated mode shape in the Central Building. The associated frequency of this mode shape is 8.37 Hz in the Central Building, while in the Waterfront Building, the frequency is 24.4 Hz. As the part of the structure on the façade deforms in the +X direction and creates a torsional effect, the longitudinal direction of the aggregate block has more resistance to restrain the movement than in the transversal direction of the Central Building. Moreover, the intermediate walls between the Waterfront and Central Building enhance the structural response in the middle. However, the most severe displacements are now located near and at the corners.

Nonetheless, the mode shapes of the aggregate block are associated mainly in the part of the façade. The Salting Factory, in the modal analysis of the aggregate block, remains unaffected, since the structure at the base is symmetrical in plan and closer toward the ground level. Higher frequencies are required to trigger the vibration on this part of the structure. Notice that the location of the Salting Factory within the aggregate, its movements are restrained by the Central Building in the X direction and the Waterfront Building in the Y direction. Therefore, the behavior of the aggregate block is influenced mainly by the interaction between the Waterfront and Central Building.

When comparing the mode shapes and frequencies of the current state to the original construction of the Central Building with the effect from the adjoining buildings, higher frequencies and different mode shapes were obtained due to the past interventions (Ortega et al. 2009b). The first three modes of the current condition are with a frequency of 7.13, 10.96, and 11.50 Hz, respectively. The failures are associated with newly introduced structural elements and materials such as the gable wall and reinforced concrete elements. The modifications highly alter the natural frequencies and vibration modes of the original structure, which can lead to an increase in seismic vulnerability, depending on the dominant frequency in an earthquake.



Mode 1 (7.12 Hz) Mode 2 (10.94 Hz) Mode 3 (11.50 Hz)

Figure 47: Numerical mode shape and frequency after calibration in current geometric configuration of Central Building (Ortega et al. 2009b).

4.5 Nonlinear Static (Pushover) Analysis

Aiming to assess the seismic vulnerability of the *Alfândega* Block and analyze the interaction among the the constituted buildings of the aggregate, a nonlinear static (pushover) analysis was carried out in both individual buildings and in the complete Block. This method of analysis has been widely accepted in estimating the peak seismic demands of different structural systems. Unreinforced masonry buildings can also use nonlinear time history analysis as an alternative due to computational efforts and lack of accepted material constitutive properties (Magenes and Pena, 2009). However, some limitations have been discussed on this topic given that many unreinforced masonry buildings contain flexible timber floor and roof diaphragms, and the responses of such buildings are known to be governed by multiple dominant modes (Magenes et al, 2016).

In the pushover analysis, a horizontal loading pattern proportional to the mass was considered in each independent direction, for both the positive and negative direction. The first phase of the analysis involved the individual buildings and then compared to the global response of the *Alfândega* Block.

4.5.1 The Central Building

For the Central Building, pushover analysis was performed in three directions: +X, since the building is symmetrical with respect to the X direction, +Y and –Y. The selected control points for defining the pushover capacity curves of the buildings are shown in Figure 48. This points were selected based on the deformation of the structure and correspond to the structural elements where higher deformations occurred as most vulnerable to seismic actions.

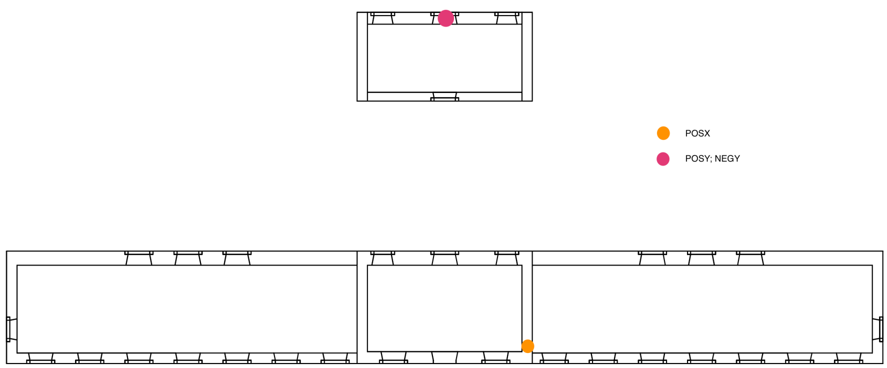


Figure 48: Selected control points for Central Building.

According to the capacity curve obtained for the +X direction, the maximum load coefficient is 0.30g at a displacement of 8.4 mm. The failure mode is associated with the torsional effect at the corner of the first floor in the front block. Since the stiffness of the wall is highly dependent on the opening dimension, it is expected that the front block have less capacity than the rear block. At the facades, the damages are located above the openings and at the floor level across the wall thickness. The back wall of the front block also has some minor cracking at the corners of opening.

The maximum seismic load coefficients achieved in the +Y and -Y direction are 0.30g and 0.33g at displacements of 4.0 mm and 24.3 mm, respectively. In the +Y direction, the structure exhibits out–of-plane deformation of the façade walls, mainly at the center of the first floor. As expected, the corners of the perpendicular walls are vulnerable to any horizontal movement. Cracks develop at the corners of the back walls of both block, due to the out-of-plane resisting mechanism of the walls, resulting in higher tensile stresses than the ones admissible for the masonry. In the –Y direction, the out–of–plane deformation is concentrated at the rear block.

Cracking is also present at the connections of the perpendicular walls, and at the opening edges, resulting from the flexural resisting mechanism.

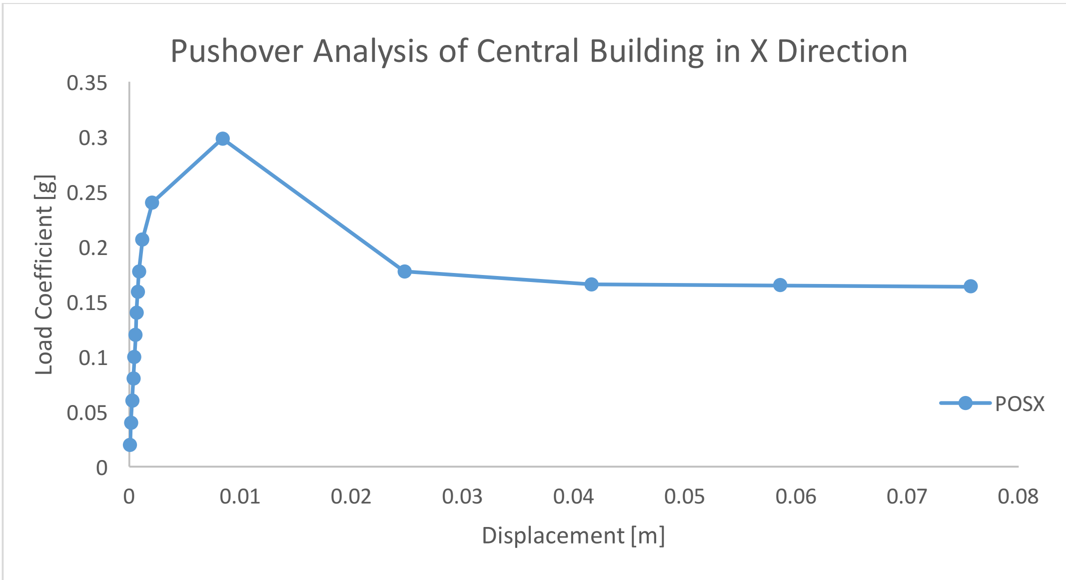


Figure 49: Capacity curve of Central Building in +X direction.

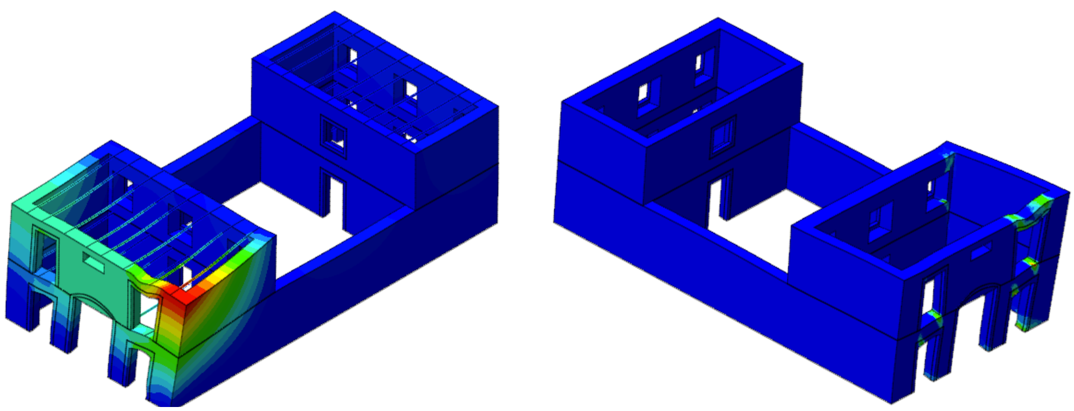


Figure 50: Deformed shape (left) and maximum principal strain (right) of the Central Building at the peak load in +X direction.

The failure mechanism is triggered at the rear block rather than in both blocks at the upper level as seen in the +Y direction. Compared to the +Y direction, the structure has slight higher capacity and considerably higher deformation ability in the –Y direction.

The peak loads, or the capacities are similar in both X and Y directions but the peak load in the +X direction is slight lower than the Y directions. This is due to the failure mode developed in the +X direction. Instead of having out–of–plane rotation of the walls in the Y directions, the failure mode in +X direction occurs because of the torsional effect at the corner. However, notice that the deformation of the peak load in the +X direction is in between +Y and –Y direction. The structure has low capacity to withstand a torsional effect, but it requires the walls move more in order to initiate the local mechanism.

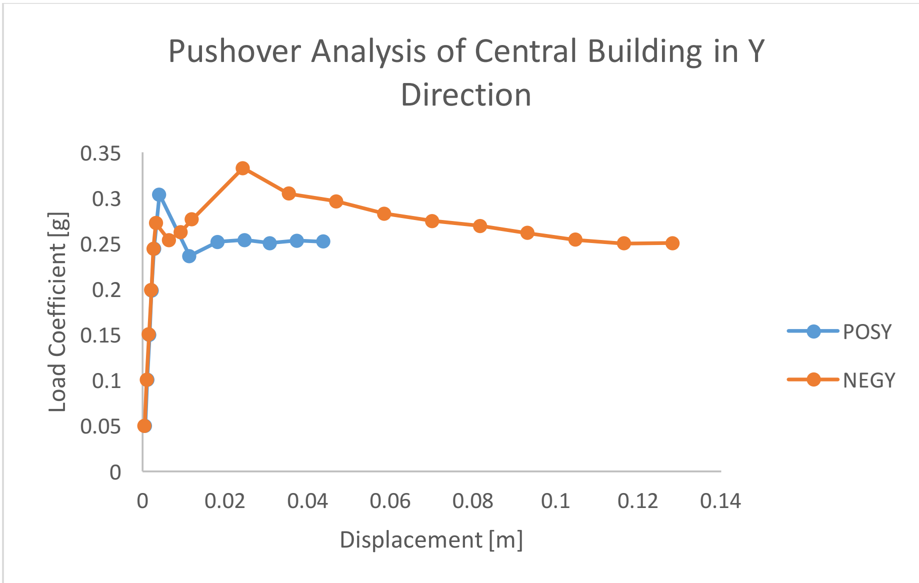


Figure 51: Capacity curve of Central Building in Y directions.

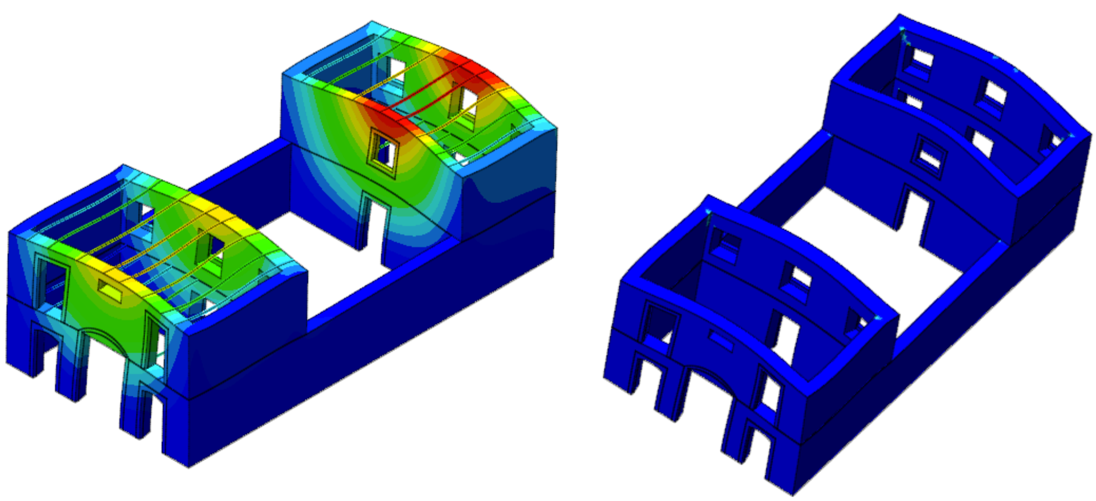


Figure 52: Deformed shape (left) and maximum principal strain (right) of the Central Building at the peak load in +Y direction.

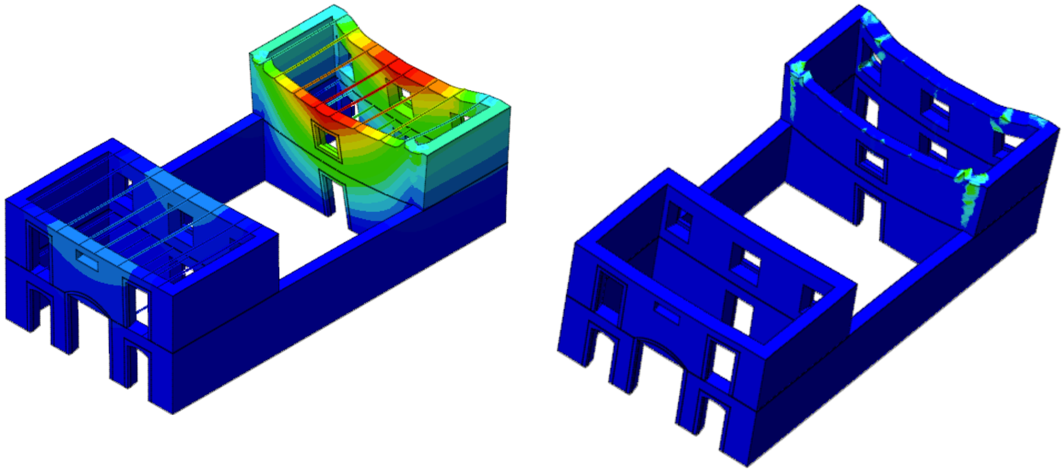


Figure 53: Deformed shape (left) and maximum principal strain (right) of the Central Building at the peak load in -Y direction.

4.5.2 The Waterfront Building

The pushover analysis of the Waterfront Building was performed in both +X, -X, +Y, and –Y directions. The selected control point for the +X direction is located at the top right corner of the first floor and for –X direction, the control point is chosen to be the top left corner of the first floor wall. For the Y direction, the chosen control point is at the mid span on top of the back wall.

The capacity curve obtained for the longitudinal direction is shown in Figure 55. The maximum load coefficient in the +X direction is 0.29g at a displacement of 13.7 mm. In the –X direction, the maximum load coefficient is 0.28g at a displacement of 7.5 mm. The capacities and deformed shapes in both directions are similar, but the deformation in the +X direction is slightly higher of the –X direction. The window opening near the corner of the left walls has influence on the deformation at which the maximum seismic capacity is achieved. When subjected to movement from the –X direction, the structure is more vulnerable. The window also affects the damage pattern at the peak load. At the peak load of the +X direction, the damages are located above the opening edges, and at one of the corners of the perpendicular walls. Unlike the +X direction, the damages are mainly located above the openings at the floor level in the –X direction. This demonstrates that the asymmetry in plan and stiffness results slight differences in terms of capacity.

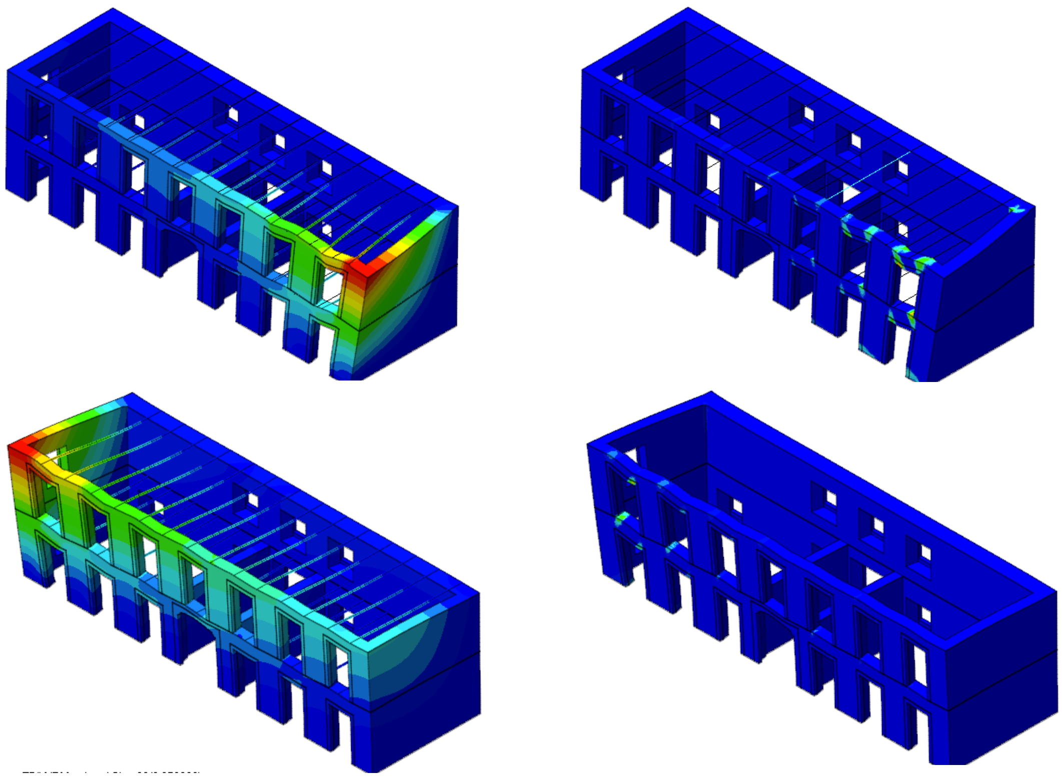


Figure 54: Pushover analysis of the Waterfront Building. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +X direction. (Bottom left) Deformed shape and (bottom right) maximum principal strain at the peak load in –X direction.

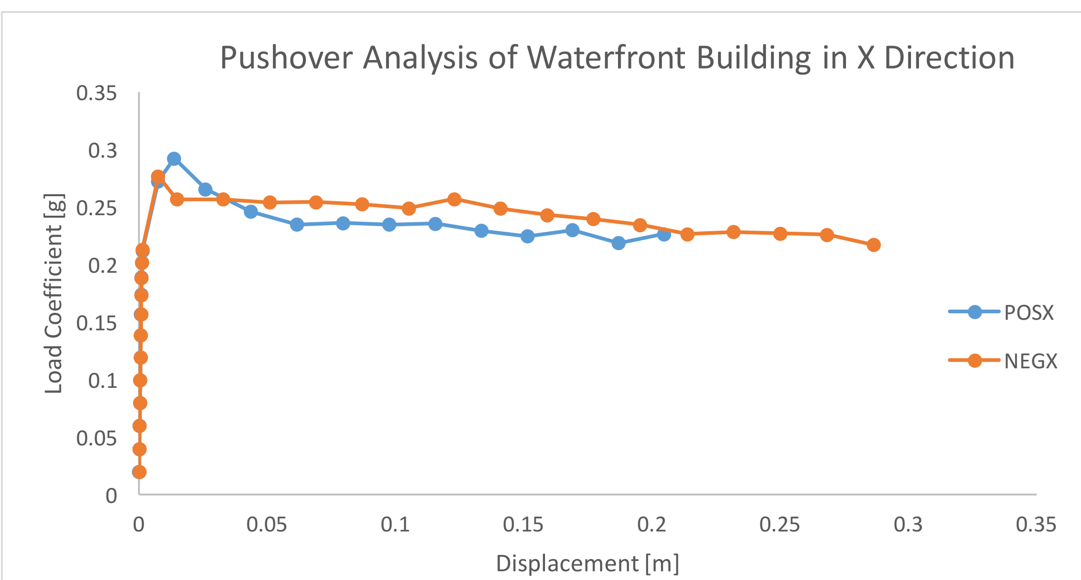


Figure 55: Capacity curve of Waterfront Building in X directions.

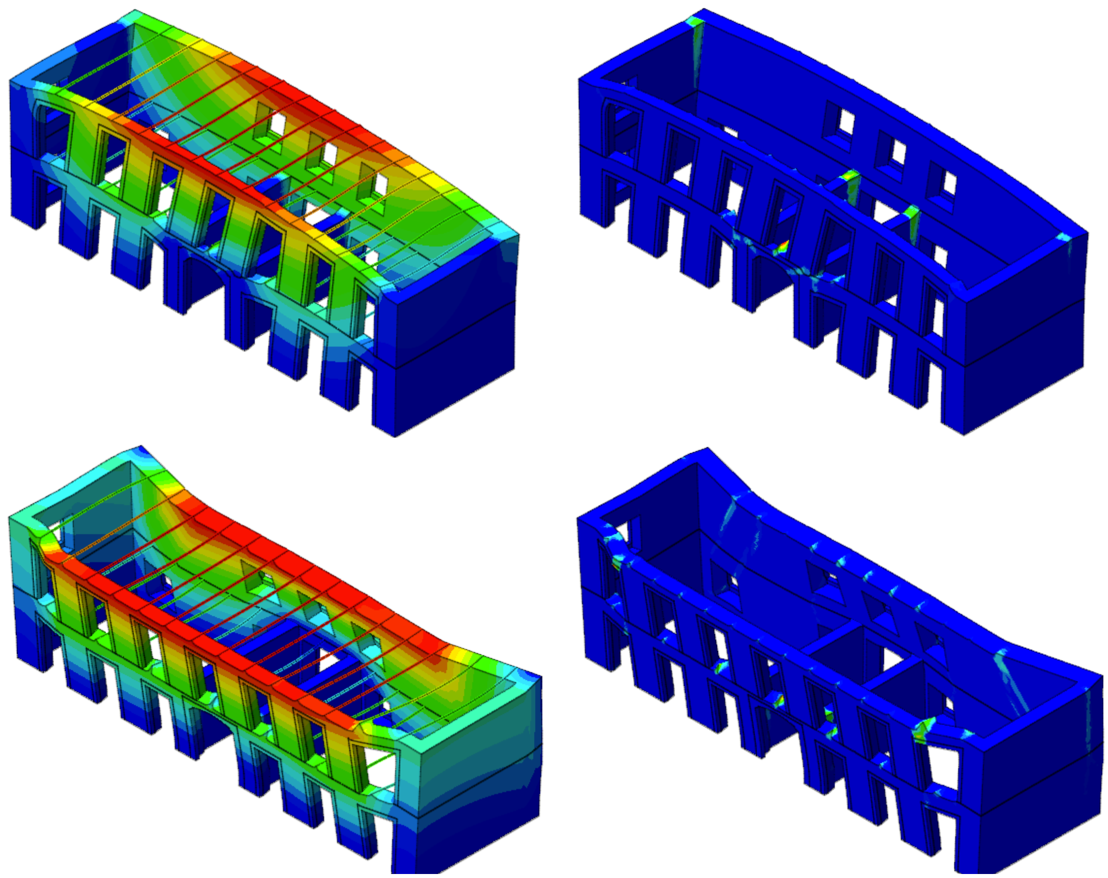


Figure 56: Pushover analysis of the Waterfront Building. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +Y direction. (Bottom left) Deformed shape and (bottom right) maximum principal strain at the peak load in –Y direction.

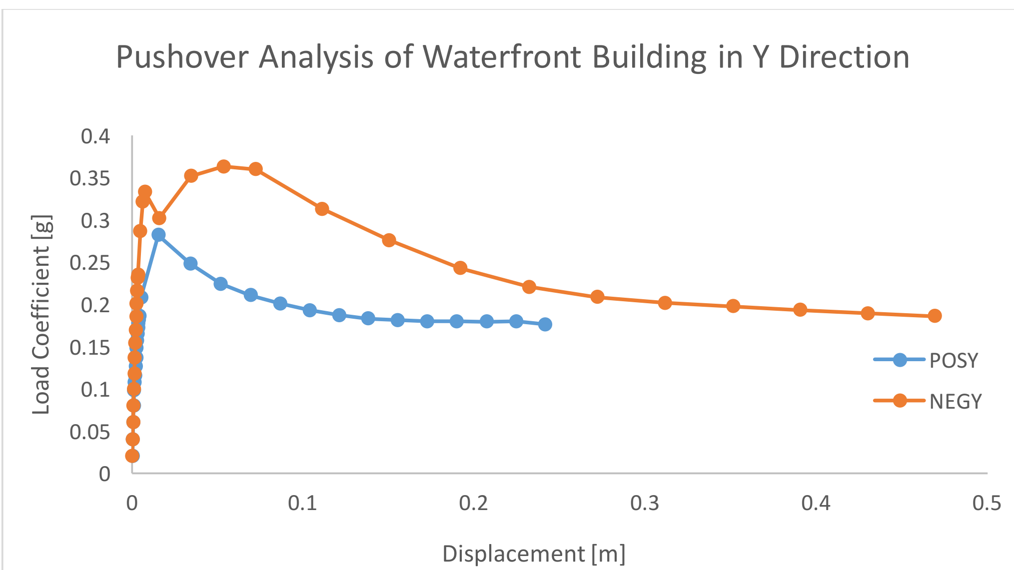


Figure 57: Capacity curve of Waterfront Building in Y directions.

In the +Y and -Y directions, the maximum load coefficients are 0.28g (at displacement of 15 mm) and 0.36g (at displacement of 53 mm), respectively. The +Y direction has a lower capacity and acceptable level of deformation. By comparing the crack patterns in both direction, it is seen that in case of +Y direction, the back wall detached completely from the perpendicular walls due to the cracking at the connections. This appears to indicate that the back wall rotates (close to a rigid body) around the base. On the other hand, in the –Y direction, the cracking develops mostly inside the masonry walls and not at the connections. This is attributed to the fact that the resisting mechanism is predominantly out-of-plane bending, which the oblique cracks at the back walls and in the façade can be seen in Figure 56. This distinct failure mechanism results also in a higher deformation capacity of the building in the –Y direction.

4.5.3 The Salting Factory

The pushover analysis of the Salting Factory was also performed in 4 directions: +X, -X, +Y, and –Y. The control points for both directions are selected on the shared walls with other buildings. For the X direction, the control points are located on the shared wall with the Central Building: In the Y direction, the control points are located on top of the shared wall with the Waterfront Building (Figure 58).

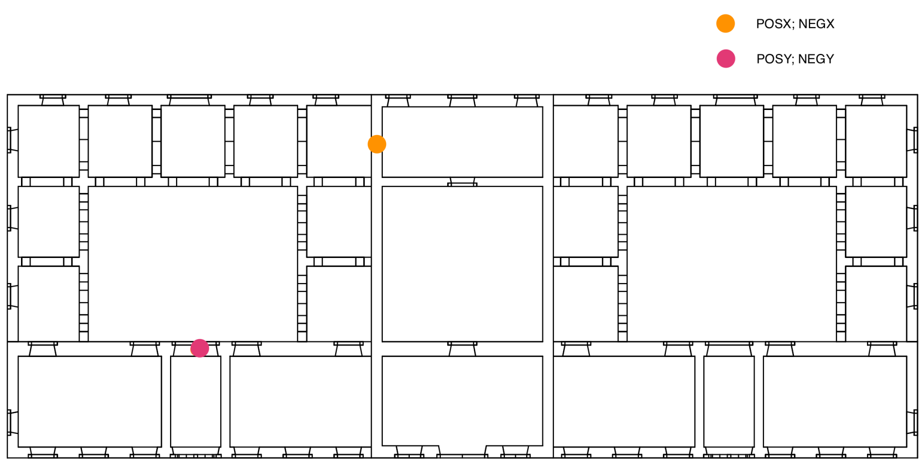


Figure 58: Locations of selected control point for pushover analysis of Salting Factory.

In the +X and –X directions, the maximum load coefficients are 0.72g and 0.75g (Figure 59). In the +X direction, deformation occurs on the perimeter walls, beams, arches and infill. At the peak load, the wall on the right side has the most prominent out–of–plane displacement, due to the cracking at the interface between the pillar and the wall, as well as the cracking on top of the infill. In the –X direction, cracking is also present on top of an infill and at an interface with a pillar. Although the behaviors are similar, the capacity in the – X direction is higher than in the +X direction. This is because of the openings on the left wall, which decrease the in–plane stiffness.

On the other hand, the deformation demand is associated with the failure mode. Since the failure mode in the +X direction occurs at the interface between the pillar and the masonry wall, these materials are stronger than the material used for infill. Hence, the structure fails with a lower deformation in the –X direction. Notice that the capacity curve for the –X direction is incomplete, but it demonstrates a higher capacity than the +X direction. This incompleteness is related to the location of the control points. In order to compare the capacity in both positive and negative X directions, the location was chosen to be the maximum displacement in the +X direction. However, as illustrated by the failure modes, the chosen control point does not have the same response in the –X direction (Figure 60).

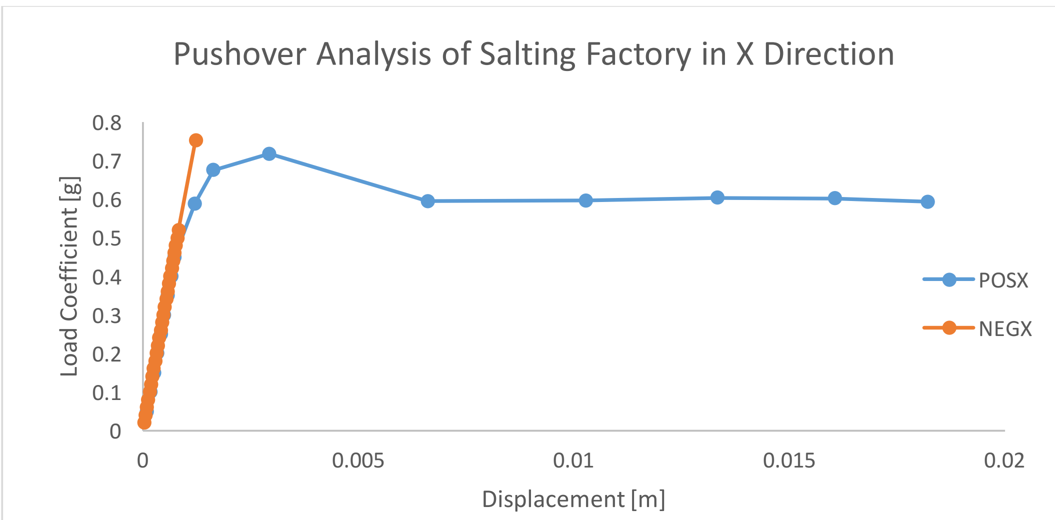


Figure 59: Pushover analysis of the Salting Factory in X directions.

The maximum load coefficients in the +Y and –Y directions are 0.62g and 0.47g, respectively (Figure 61). Both of the directions show the highest displacement occurs at the mid–span of the shared wall with the Waterfront Building, see Figure 62.

However, due to the different failure modes, the resulted capacities have a great discrepancy. In the +Y direction, the wall rotation originates at the location of the door opening, and extends between the windows. In the –Y direction, the wall rotation causes the failure at the interfaces between the pillar and the infill. As a result, the capacity in the –Y direction is lower than the +Y direction.

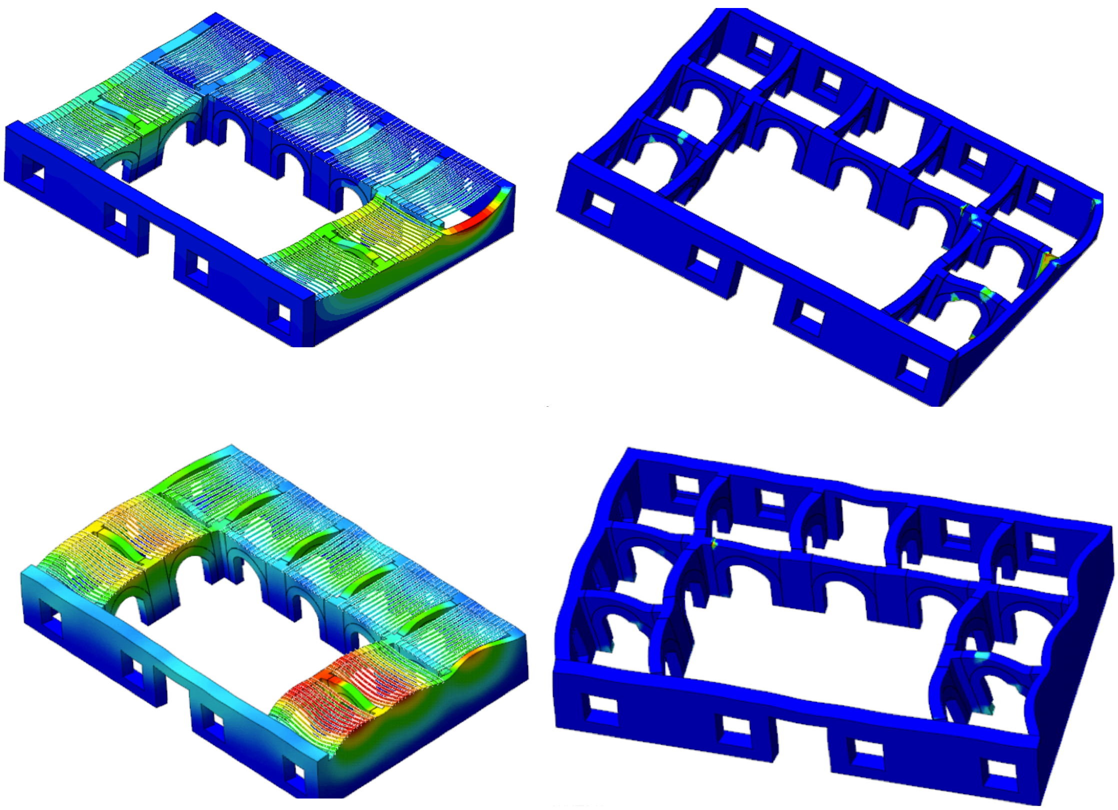


Figure 60: (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +X direction. (Bottom left) Deformed shape and (bottom right) maximum principal strain at the peak load in -X direction.

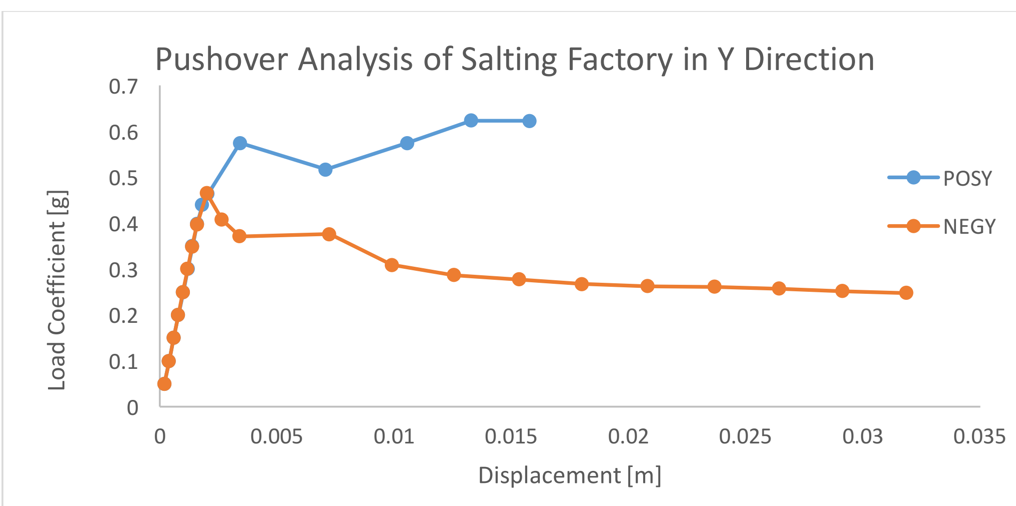


Figure 61: Capacity curve of Salting Factory in Y Directions.

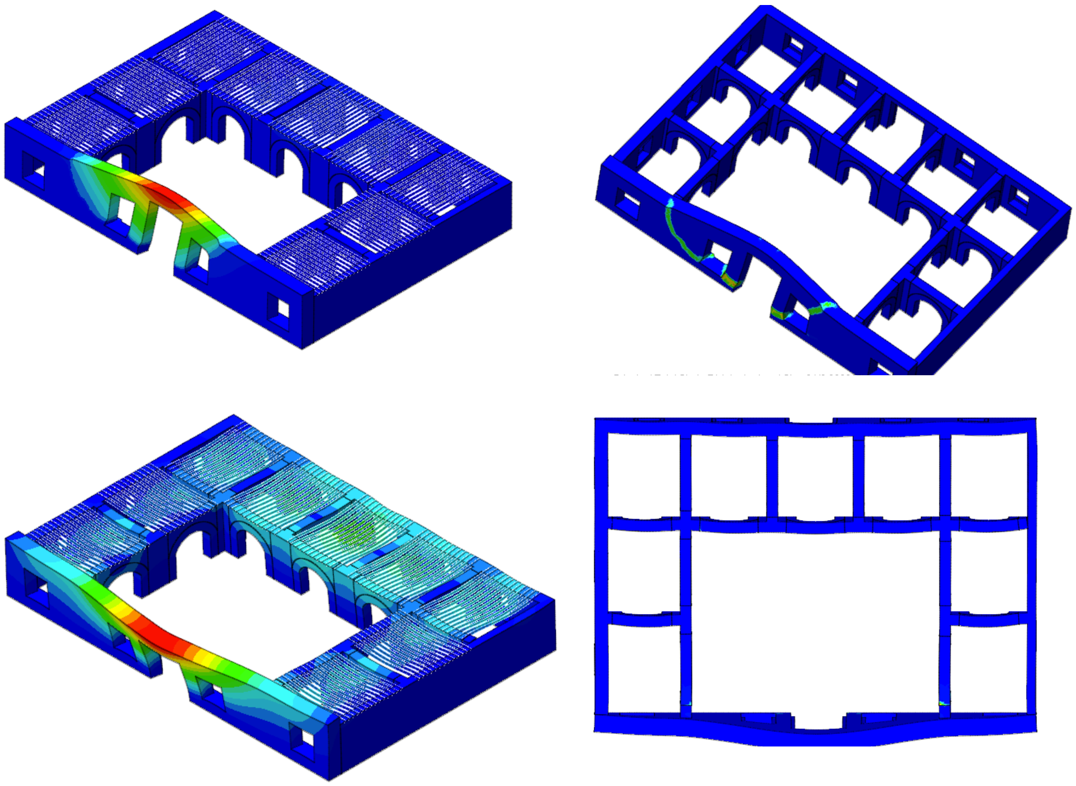


Figure 62: Pushover analysis of the Salting Factory. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +Y direction. (Bottom left) Deformed shape and (bottom right) maximum principal strain at the peak load in –Y direction

Nonetheless, the capacities in both the X directions are higher than the Y directions. Although all of the failure modes are associated with the shared walls with other buildings, the shared wall with the Central Building is supported by the longitudinal walls and series of arches in the X directions. As the most vulnerable element in the Y directions, the shared wall with the Waterfront Building does not have any resistant element. Therefore, the Y directions have a lower capacity.

4.5.4 The Aggregate Block

The pushover analysis of the aggregated block was considered in three directions: +X, +Y, and –Y. In order to compare the capacity of the isolated buildings, similar locations were selected for the control points, as illustrated in Figure 63.

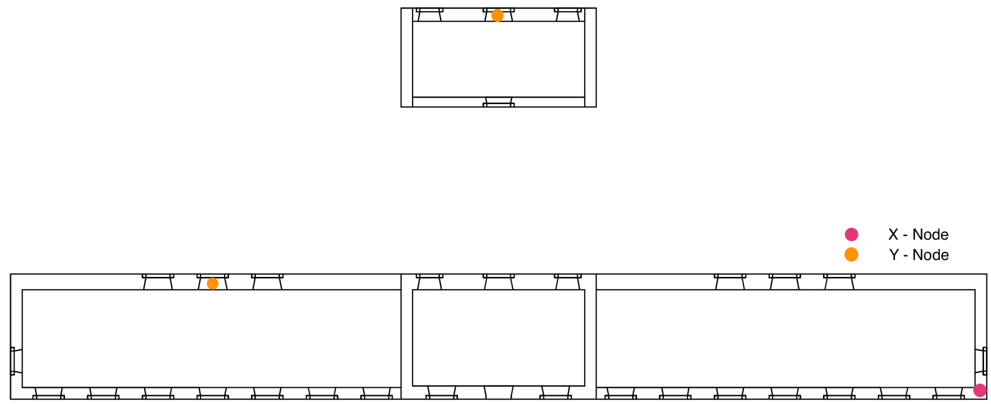


Figure 63: Selected control points for pushover analysis of the Aggregate Block.

The maximum load coefficient in the +X direction is 0.36g at a displacement of 12.7mm, see Figure 64. At the peak load, the deformed shape is associated with the torsional effect at the right corner of the Waterfront Building (Figure 65). According to Figure 66, the damages are located at the opening edges and the connection between the perpendicular walls on the right side.

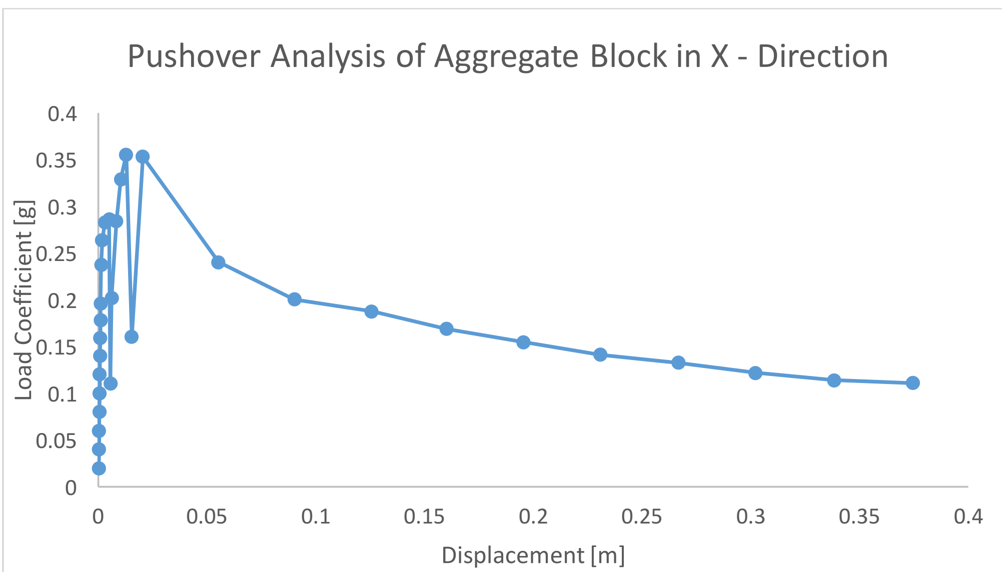


Figure 64: Capacity curve of Aggregate Block in X direction.

In the +Y direction, the maximum load coefficient is 0.39g, but occurs at different displacements in the Waterfront and the Central Building (see Figure 67). The structure experiences out–of–plane deformations on all upper level walls. The largest displacement occurs at the mid–span of the Waterfront and the rear block of the Central Building (Figure 68). The Waterfront Building on the left undergoes a displacement of 10.7 mm while the Central Building undergoes a displacement of 11.0 mm. Although the difference is very small, this indicates that the Waterfront Building is more vulnerable than the Central Building in the global behavior. This can also be demonstrated in the damage pattern illustrated in Figure 69.

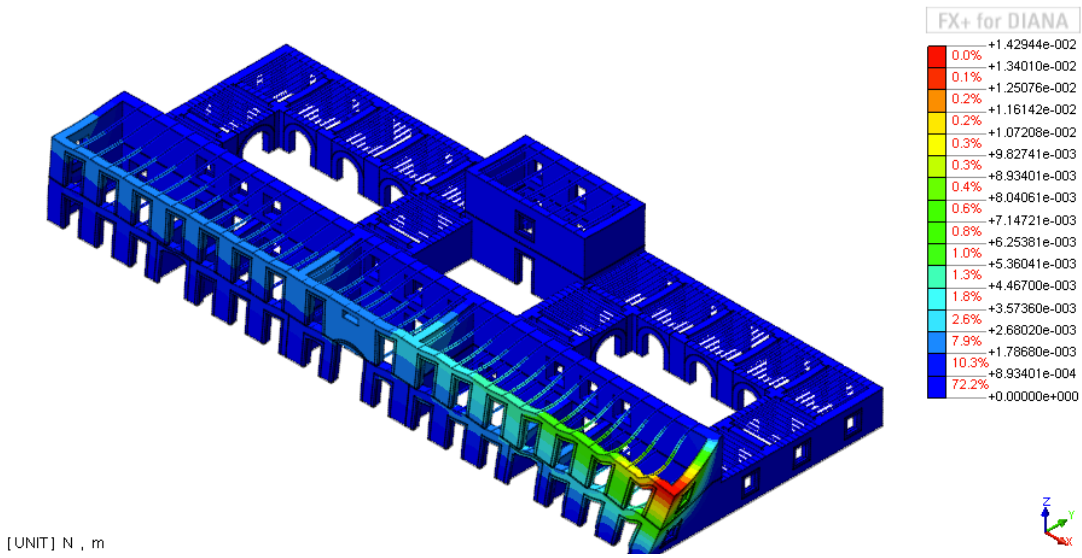


Figure 65: Deformed shape of Aggregate Block at peak load in +X direction.

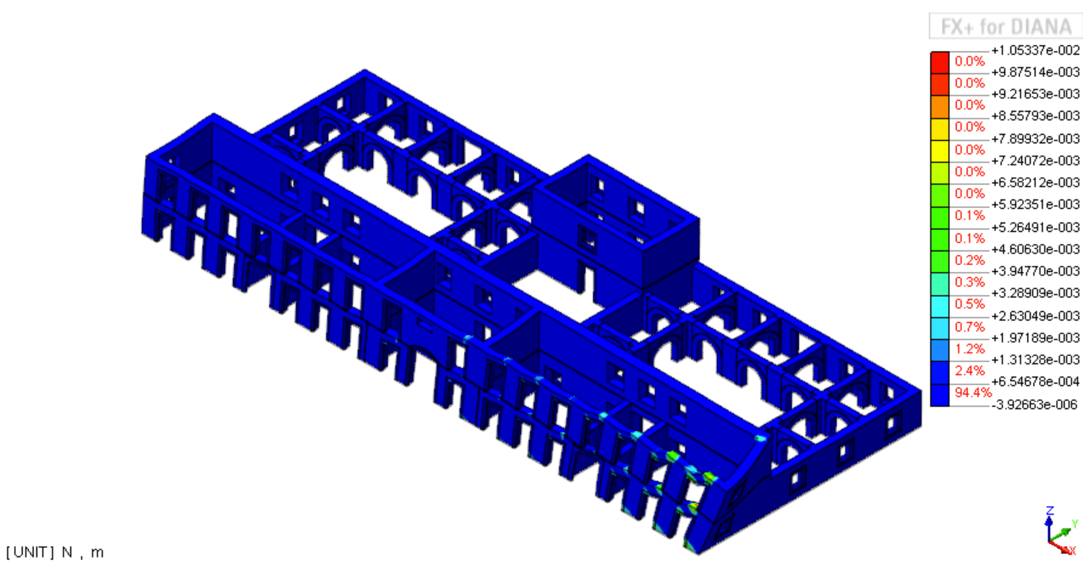


Figure 66: Maximum principal strain of Aggregate Block at peak load in +X direction.

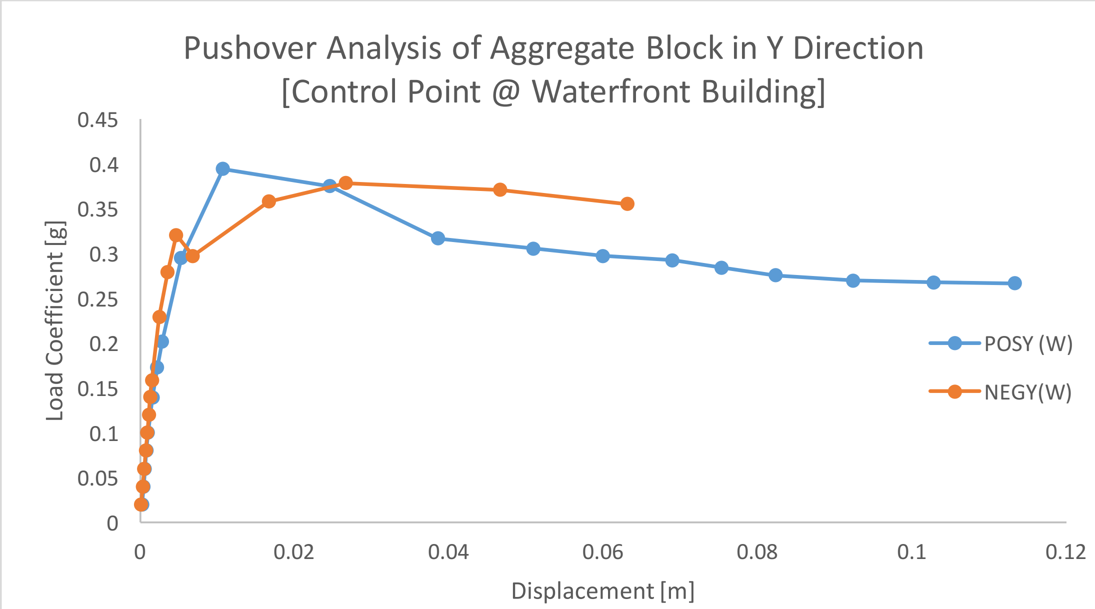


Figure 67: Capacity curve of Aggregate Block in Y directions, with control points at Waterfront Building.

The rotation of the back wall of the Waterfront Building occurs due to the cracking along the corners of the perpendicular walls. Although cracks are also present at the corners of the rear block of the Central Building, the magnitude of tensile strain is lower than those observed in the Waterfront Building. Since the walls have the same material properties, the failure mode is triggered by the deformation in the Waterfront Building.

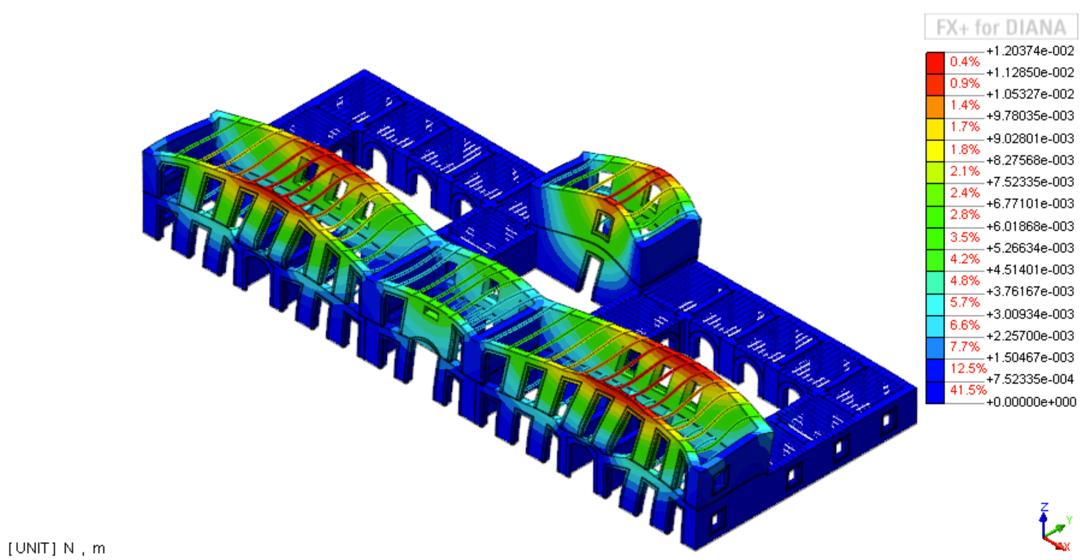


Figure 68: Deformed shape of Aggregate Block at peak load in +Y direction.

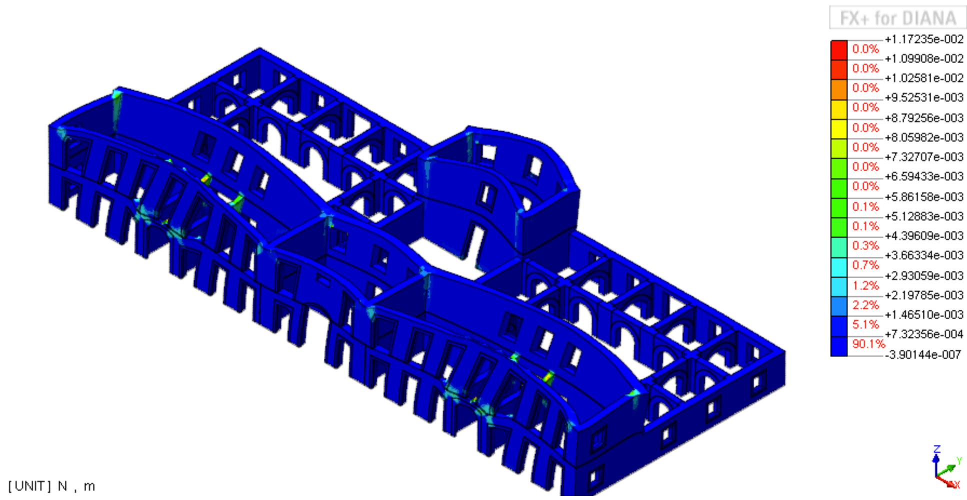


Figure 69: Maximum principal strain of Aggregate Block at peak load in +Y direction.

In the –Y direction, the maximum load coefficient is 0.38g (Figure 67). At the peak load, the structure has the largest deformation at the off–center locations of the back walls of the Waterfront Buildings. The displacement in the Waterfront Building is 26.7 mm and a displacement of 16 mm in the rear block of the Central Building. As the Waterfront Buildings deform more than the Central Building, higher magnitude and larger area of tensile strain is developed in the Waterfront Building. Hence, the Waterfront Building is more vulnerable than the Central Building as in the +Y direction. In fact, the failure patterns of the block is governed by the failure of the façades of the Waterfront buildings as it can be seen in Figure 71 and Figure 72. The damage level developed in the rear block of the Central Building is considerably lower than in the Waterfront Building. It can be confirmed by the capacity curve obtained at the control point of the rear block as shown in Figure 73.

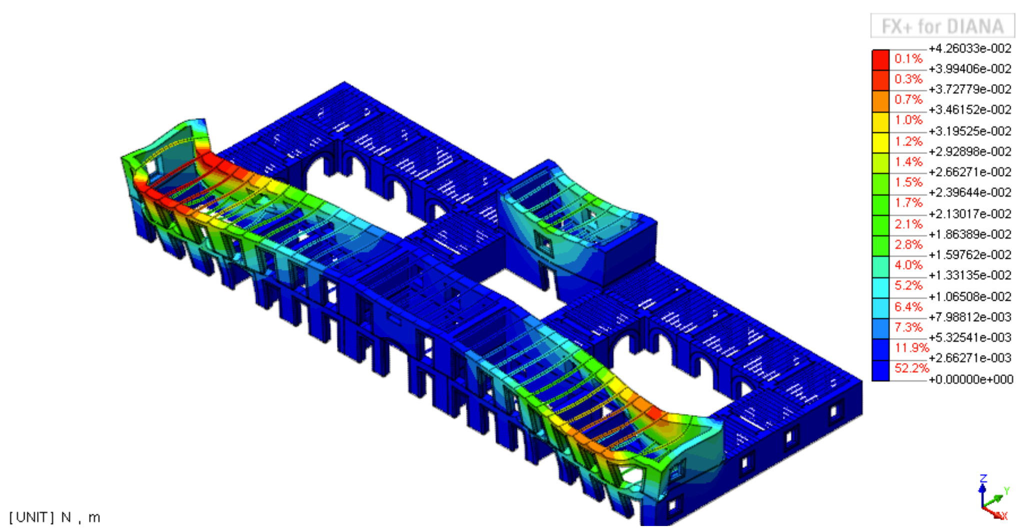


Figure 70: Deformed shape of Aggregate Block at peak load in -Y direction.

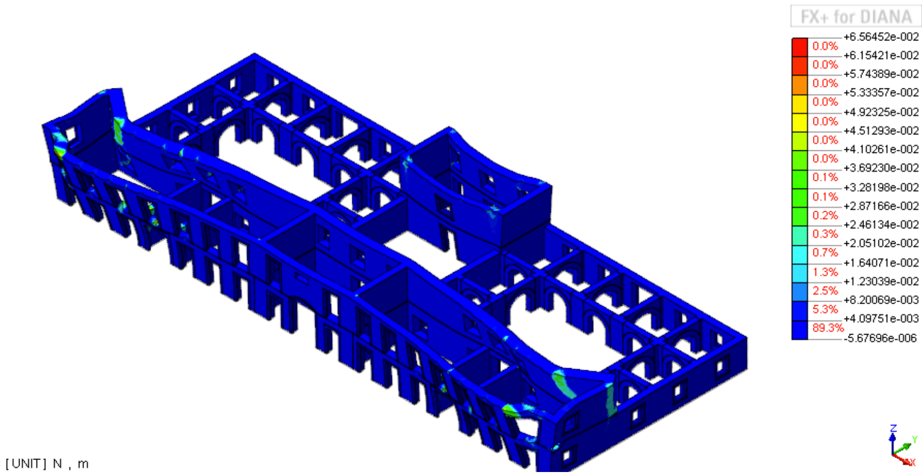


Figure 71: Maximum principal strain of Aggregate Block at peak load in -Y direction.

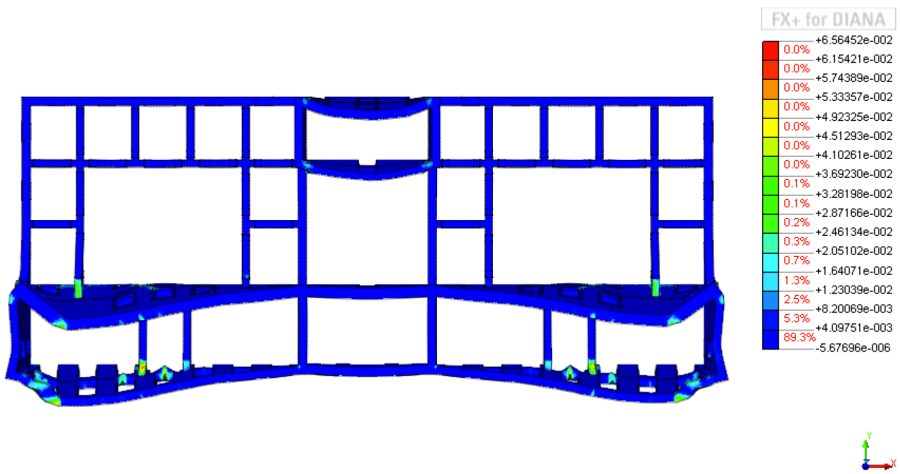


Figure 72: Plan view of maximum principal strain of Aggregate Block at peak load in -Y direction.

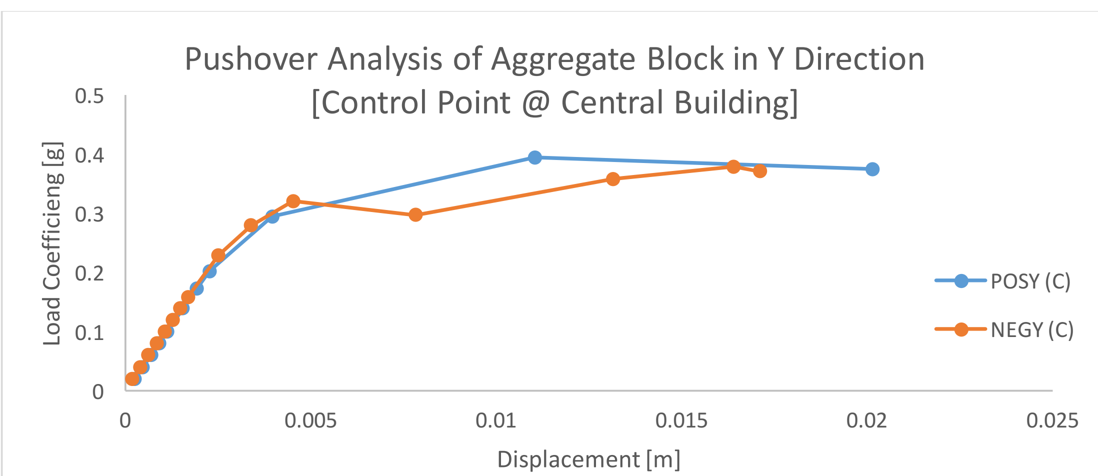


Figure 73: Capacity curve of Aggregate Block in Y directions, with control points at Central Building.

4.5.5 Discussion of Results

Table 6 summarizes the peak load coefficients of the isolated buildings and the Aggregate Block in both directions. The capacities in the Aggregate Block are higher than the capacities in the Central and Waterfront Building, but lower than the Salting Factory's, which should be expected taking into account the geometric configuration of the latter building. It highlights the importance to the global aggregate analysis in the assessment of the buildings behavior.

In the +X direction, the failure mode of the Aggregate Block involves the façade at the right corner where some torsional effects develop. This behavior is very similar to the failure mode of the isolated Waterfront Building when loading in the same direction. As the damages at the peak loads are concentrated in the right Waterfront Building with a minor crack at one of the corner openings in the Central Building, it demonstrates that there is an interaction between the Waterfront and the Central Building. The Central Building restrains the movements of the Waterfront Building on the left (Waterfront Building at right) but has no influences to the Waterfront Building on the right corner. Nonetheless, as located between two, Waterfront Building, the Central Building connects them together to form a single longitudinal block. The capacity of the Aggregate Block, hence, obtains a higher capacity than in the isolated Central or Waterfront Building. Note that the isolated Salting Factory has a higher capacity than the Aggregate Block. As an isolated building, the failure mode in the Salting Factory is associated with the deformation of the shared wall of the Central Building causing the failure in materials. This failure mode, however, in the global scale of the structure requires a stronger movement compared to the forces triggered a failure mechanism at the upper level of the Aggregate Block. Notice that global the behavior of the Aggregate Block is controlled by the behavior of the Waterfront building, the Salting Factory has a minor role. This indicates that the interaction between the adjoining buildings in an aggregate block is more complex rather than improving the global behavior as a summation.

Besides, the complexity is emphasized in the Y directions. Similar to the +X direction, the global structural capacities in the Y directions are slightly improved compared to the isolated buildings due to the interaction between the adjoining buildings. Yet, each failure mode of the Aggregate Block can be seen as a combination of the failures modes in the isolated buildings. In either +Y or –Y direction, the out–of–plane deformation occurs both at the Waterfront and the Central Building, but the Waterfront Building in this direction controls the failure of the aggregate. It shows that the Waterfront Building is the most vulnerable part within the aggregate. These parts in the Aggregate Block seem to behave independently with their own failure mechanism resulted in the isolated buildings. Notice that however, the shared walls between the Waterfront Building and the Salting Factory at the ground floor, retrains the out-of-plane movements at this level, which result in the reduction on the magnitude of the tensile strain obtained in each isolated building and thus increase the global capacity. The detachment of back walls along its height as in the isolated building is no is no longer possible. Nonetheless, the capacities of the Aggregate Block are very similar in both X and Y directions.

Finally, it should be mentioned that in spite of the timber floors are modeled by the longitudinal beams, they are considered to be flexible since they are perfectly connected to the walls. The interaction between the parallel walls also contribute for the seismic resistance.

Table 6: Summary of peak load coefficient in different buildings

Direction	Central Building	Waterfront Building	Salting Factory	Aggregate Block
+X	0.30g	0.29g	0.72g	0.36g
-X	N/A	0.28g	0.75g	N/A
+Y	0.30g	0.28g	0.62g	0.39g
-Y	0.33g	0.36g	0.47g	0.38g

5.0 Numerical Parametric Analysis

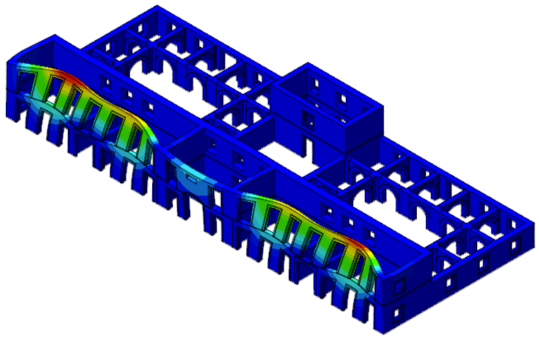
The seismic vulnerability of masonry buildings, as previously mentioned, is often associated with poor material quality and construction workmanship, structural configuration, and typological heterogeneity between adjoining buildings. One major issue is the interaction between the different structural elements, namely connection of intersecting walls and those between walls and floors. In the absence of good connections, seismic vulnerability increases due to premature collapse of walls in the out – of – plane direction. Therefore, it is essential to prevent such development by employing adequate in-plane resisting mechanism that enables the global stability of the buildings. This is often known as a “box–behavior”.

In spite of knowing the dependency of seismic vulnerability in masonry buildings on the above mentioned parameters, this work focus on the effect of the flexible timber floors in the global seismic behavior of the aggregate building. It was decided to vary the stiffness of the timber beams as a simplified way to assess the influence of the floor stiffness, taking into account that the floors are adequately connected to the masonry walls by the evidences from a previous in-situ inspection to the Central Building. Three scenarios are evaluated: (1) no beam elements, which corresponding to the absence of floors as the most unfavorable scenario; (2) timber beams with a double of the original beam stiffness (E= 20 000 MPa); and (3) timber beams with a stiffness five times more than the original beam stiffness (E=50 000 MPa). The main idea associated with each scenario is to consider the absent of floor slab, introduce a new flexible unidirectional slab, and a new unidirectional rigid slab, respectively.

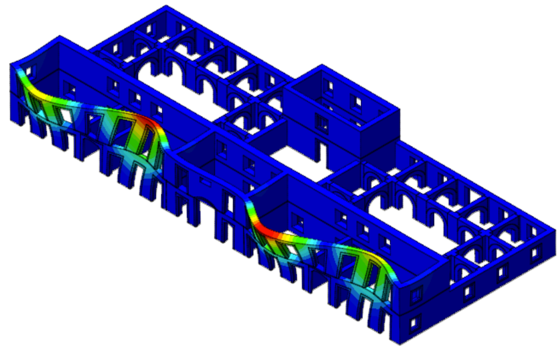
5.1 Modal Analysis

As a first step, the natural frequencies and corresponding mode shapes were obtained and compared among the different stiffness of timber beams. As illustrated in Figure 74, the mode shapes of the Aggregate Block without timber beams are mostly dominated by local modes occurring in the facade and back walls of the Waterfront Building along the Y direction, which corresponds to the direction of more flexibility. The first mode (f = 4.65 Hz) and the eighth mode (f = 6.50 Hz), are associated with out–of–plane deformation in the Y direction. Since there are no beams present, there is no interaction between the two longitudinal walls and the structural behaviors are contrary to the behaviors seen in the previous chapter. Notice that in the eighth mode, the front block of the Central Building is deformed on both sides. Based on this observation, it is expected that the most vulnerable structural elements are the longitudinal walls of the Waterfront Building, and given the potential for the façade of the Central Building develops an out–of–plane mechanism. The ninth mode, is also subjected to deformation of the Waterfront Building mainly in the Y direction. It shows that at this frequency, 6.53 Hz, the Central Building is not affected but the back walls of the Waterfront Building which rotate toward different directions.

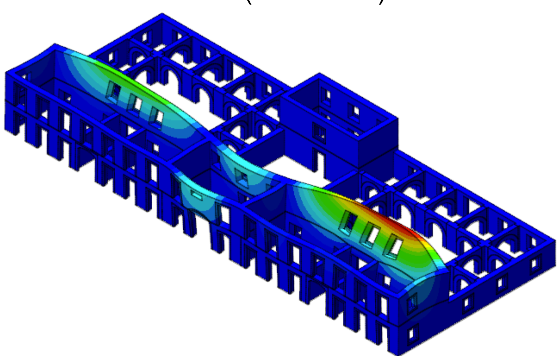
The fourth mode, with a frequency of 5.18 Hz, is governed by the translation along the X direction. However, the interior masonry walls at the ground floor restrain the movement and result in a sinusoidal shape of the façade walls.



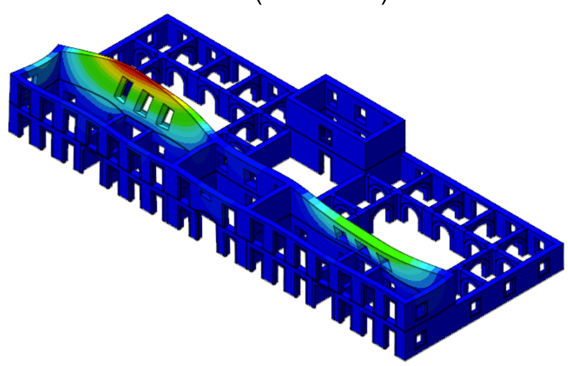
Mode 1 (f = 4.65 Hz)



Mode 4 (f =5.18 Hz)



Mode 8 (f = 6.50 Hz)



Mode 9 (f = 6.53 Hz)

Figure 74: Modal analysis of Aggregate Block without beam.

Some of the mode shapes with the beam stiffness of 2E are shown in Figure 75. The first mode, with a frequency of 6.30 Hz, shows that the façade and the back walls are deformed together due to the coupling effect of the beams. This mode is similar to the eighth mode shape (f= 6.50 Hz) of the structure without beams. The fourth mode, with a frequency of 6.80 Hz, exhibit a deformation in the Central Building and parts of the Waterfront Building. The seventh mode (f = 9.27 Hz) is associated with the deformation in the X direction, as well as the ninth mode (f = 10.86 Hz), which exhibits a torsional effect in both of the Waterfront Building. In the modal analysis with the beam stiffness of 5E, the mode shapes are identical but corresponding to slightly higher frequency levels.

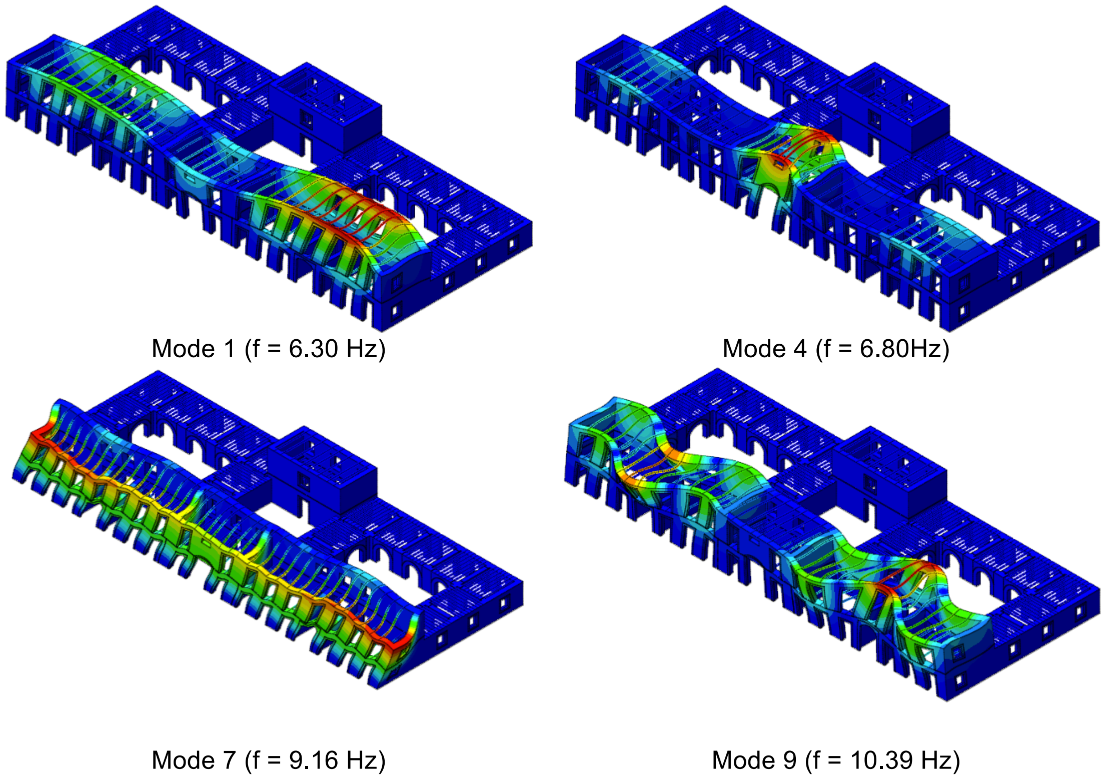


Figure 75: Modal analysis of Aggregate Block with beam stiffness of 2E.

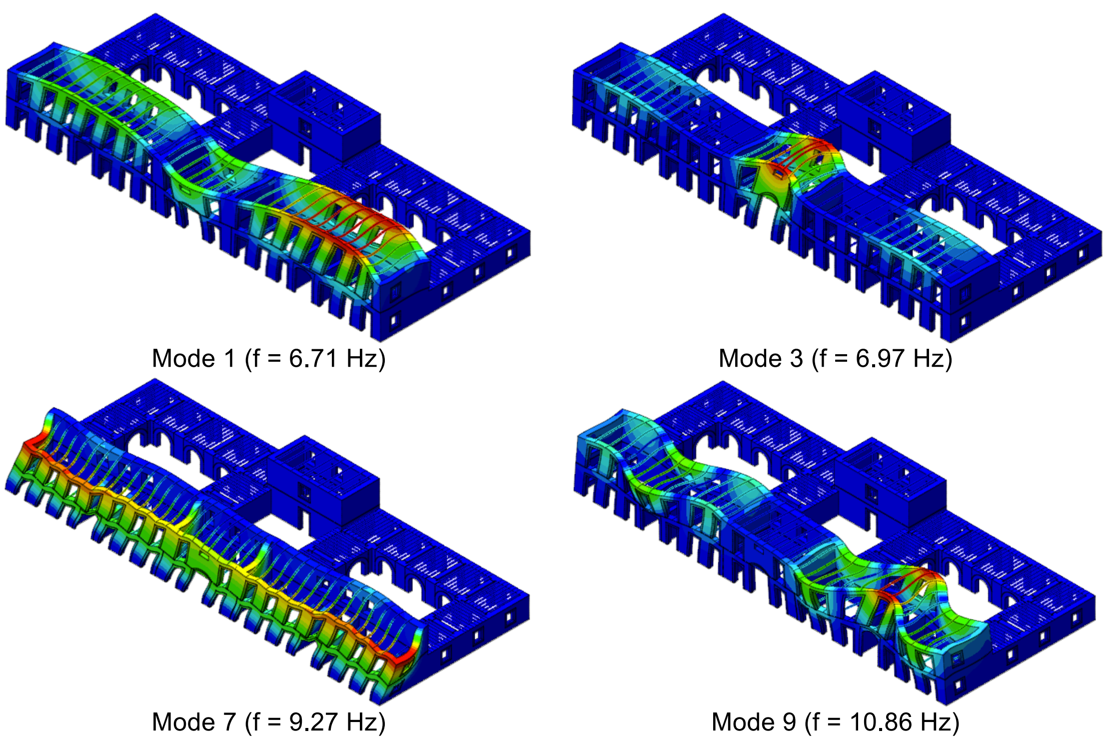


Figure 76: Modal analysis of Aggregate Block with beam stiffness of 5E.

5.2 Nonlinear Static (Pushover) Analysis

Following the same procedure as described in the previous chapter, the nonlinear static (pushover) analysis was carried out in the +Y and –Y direction with a horizontal load pattern proportional to the mass. As the most vulnerable directions are associated with the orientation of the beams seen in previous analysis, these two directions were chosen. The selected control point for three scenario is located at the façade of the Waterfront Building (Figure 77).

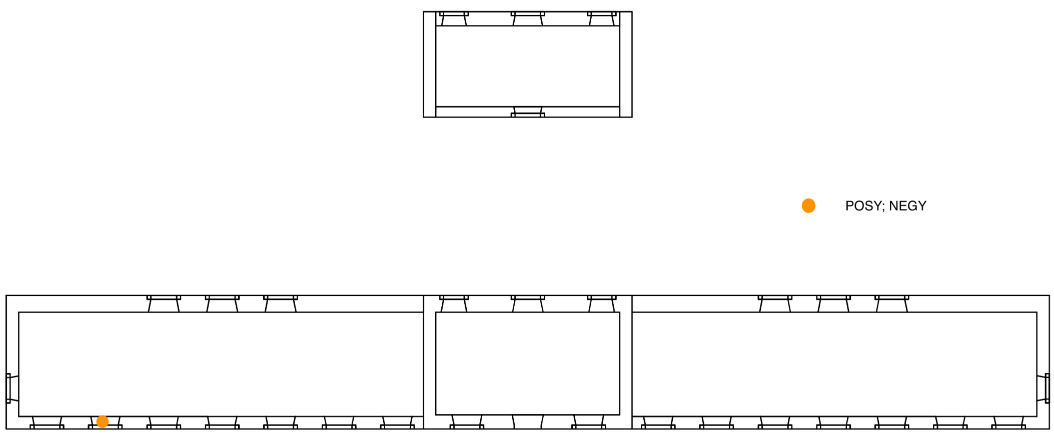


Figure 77: Selected control point for numerical parametric study in pushover analysis.

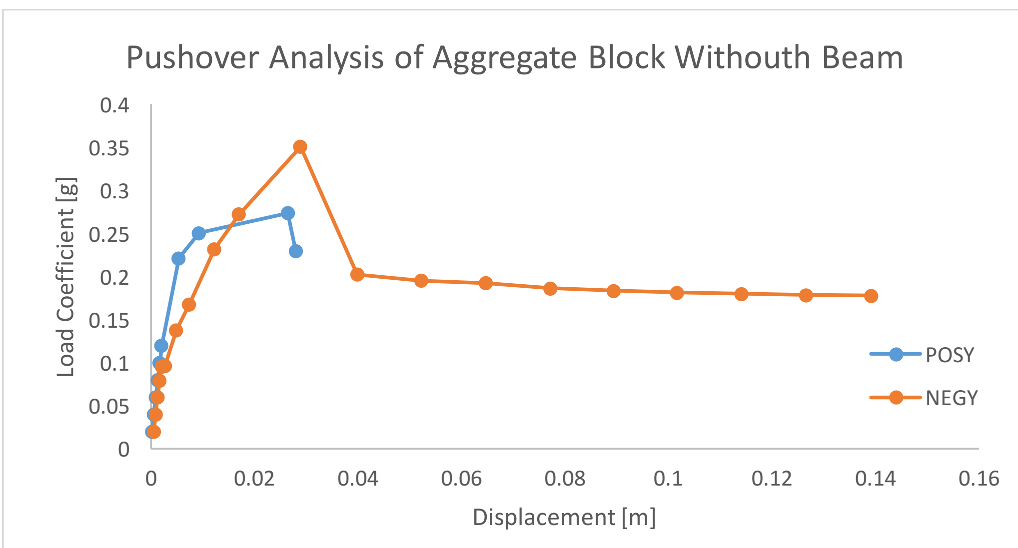


Figure 78: Capacity curve of Aggregate Block without beams in Y directions.

The maximum load coefficients of the Aggregate Block without beams are 0.27g and 0.35g in the +Y and –Y direction, respectively. In the +Y direction, the failure mode occurs due to the out–of–plane rotation of the back walls of the Waterfront Building, and is accelerated by the cracking developed at the connections between the perpendicular walls. The front walls of the Waterfront Building are also suffered important out–of–plane deformations, leading to severe cracking between the front façades and the transversal external walls and cracking at the base of the central masonry piers between the door openings. Following the field of displacements, no damages develops in the front block of the Central Buildings and small cracks appeared at the corner of the rear block of the Central Building as the result of the trend for detachment of the back walls due to out-of-plane loading. Due to the amount of damages present in the +Y direction, its capacity is lower than the capacity in –Y direction. In the –Y direction, the failure mode is associated with out –of-plane rotation of the front façade walls, particularly the façade of in the Waterfront Buildings. The Central Building present out-of-plane deformation of the front façades but with considerable lower magnitude (including the front and rear blocks). In this case, damages are concentrated on the front facades of the Waterfront Building, particularly at the connection with intersection walls. Since there are no beams in the structure, both failure modes occur at a similar displacement, 26 mm (in the +Y direction) and 28 mm (in the –Y direction). It is clear that the absence of transversal walls reducing the span of the façades of the Waterfront buildings becomes these elements very vulnerable to out-of-plane loading, thus controlling the collapse of the building under seismic actions.

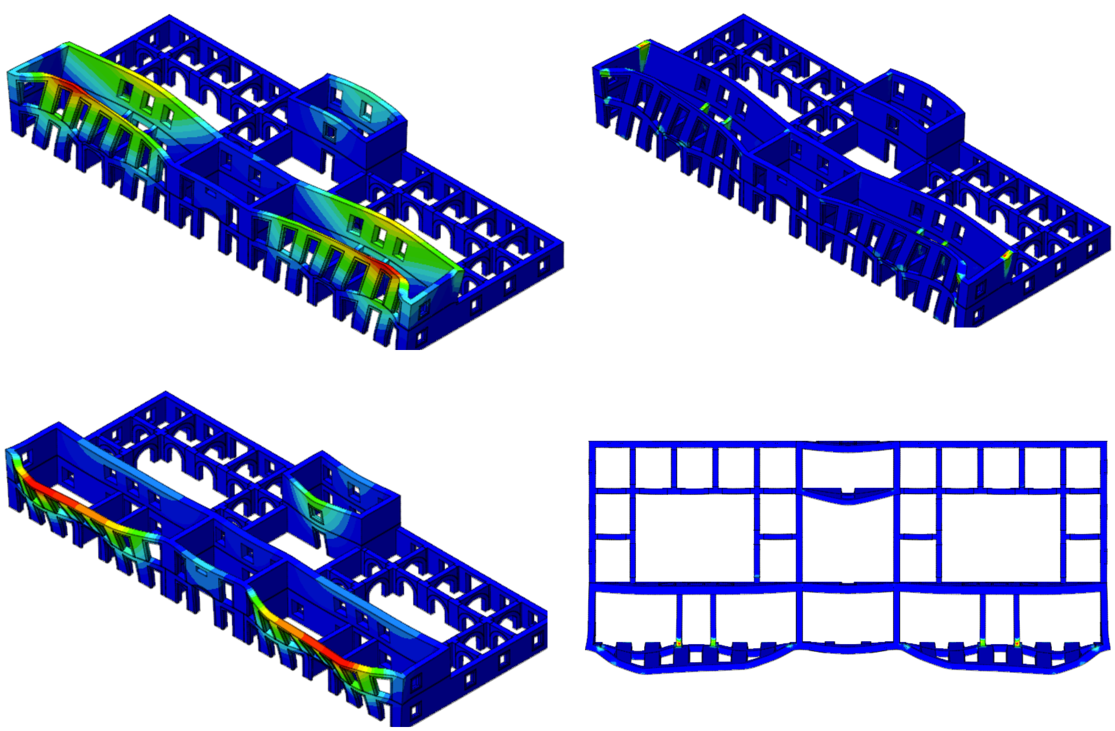


Figure 79: Pushover analysis of Aggregate Block without beams. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +Y direction. (Bottom left) Deformed shape and (bottom right) top view of maximum principal strain at the peak load in –Y direction.

In the Aggregate Block with beam stiffness of 2E, the displacement in +Y direction is 14.0 mm, and the displacement in –Y direction is 70 mm at peak load (Figure 80). In the +Y direction, the out– of–plane rotation of the back walls is accelerated by the coupling effect of the beams as both the façade and back walls in the Waterfront Building deform, which result in a large amount of tensile strain at the corners of the perpendicular walls. In the – Y direction, same failure mode develops, corresponding to similar damage pattern. However, higher capacity and deformation characterize the response of the building, as illustrated in Figure 82 and Figure 82. In fact, the maximum load coefficients of the Aggregate Block with beam stiffness of 2E are 0.32g and 0.42g in the +Y and –Y direction, respectively.

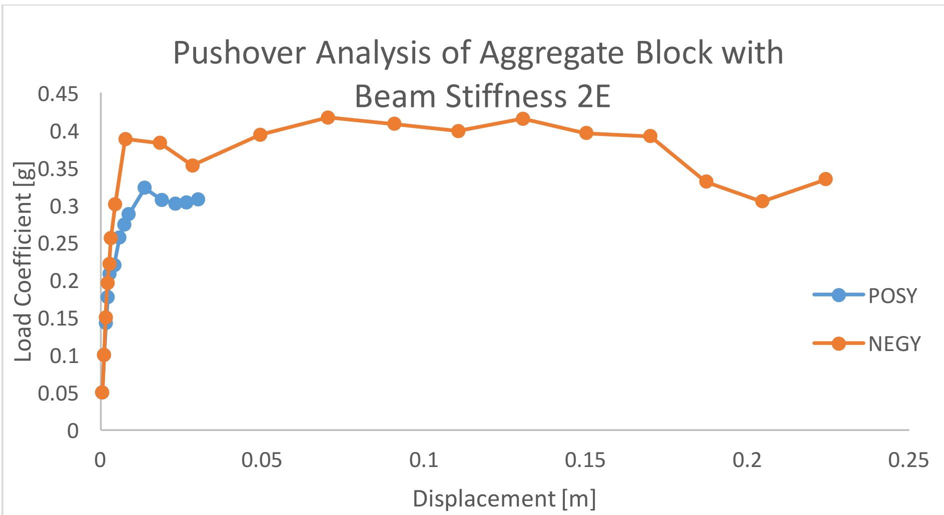


Figure 80: Capacity curve of Aggregate Block with beam stiffness of 2E in Y directions.

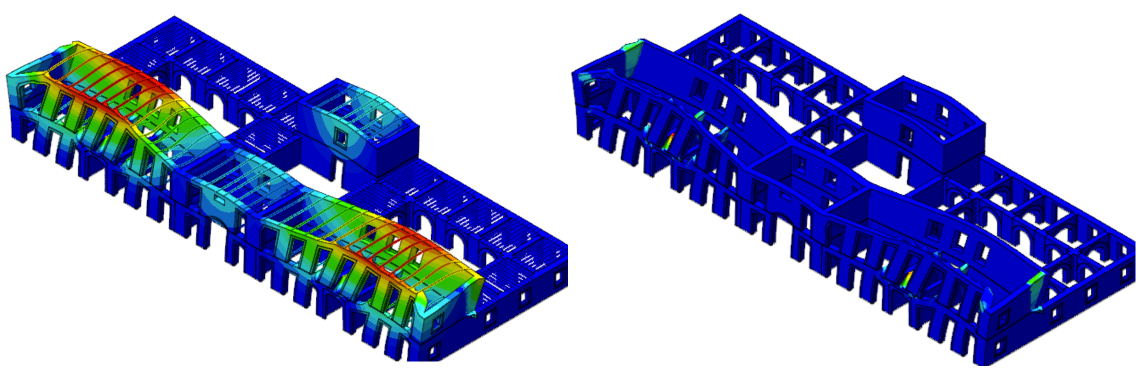


Figure 81: Pushover analysis of Aggregate Block with Beam Stiffness of 2E. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +Y direction.

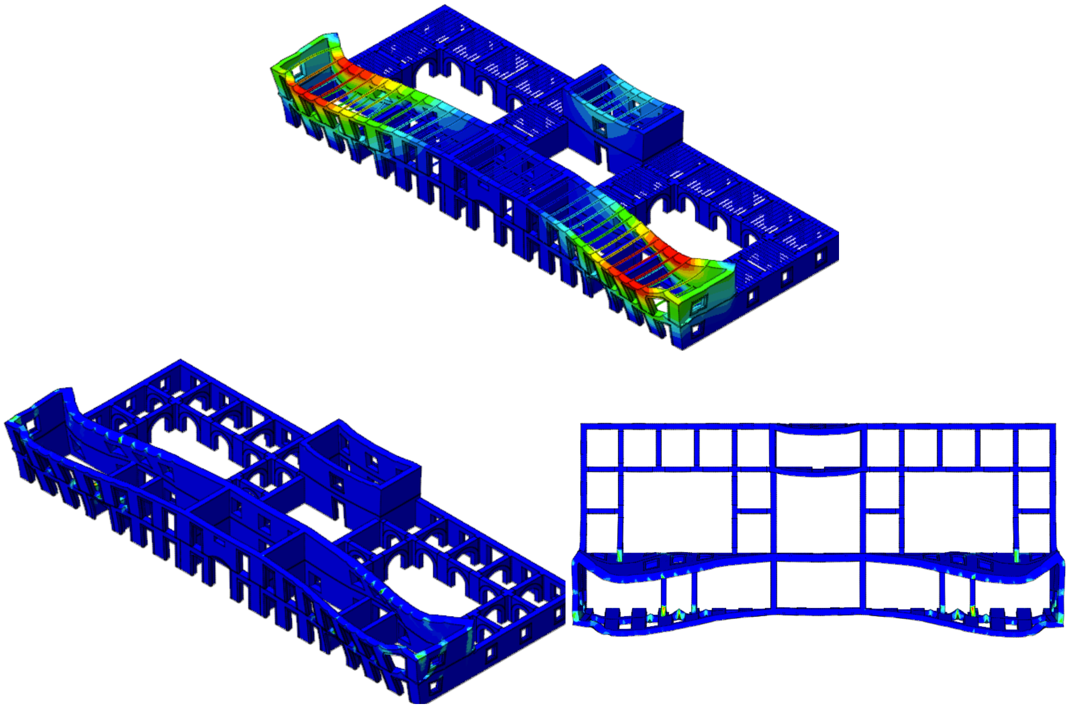


Figure 82: Pushover analysis of Aggregate Block with Beam Stiffness of 2E. (Top) Deformed shape (bottom) isometric and plan view of the maximum principal strain at the peak load in –Y direction.

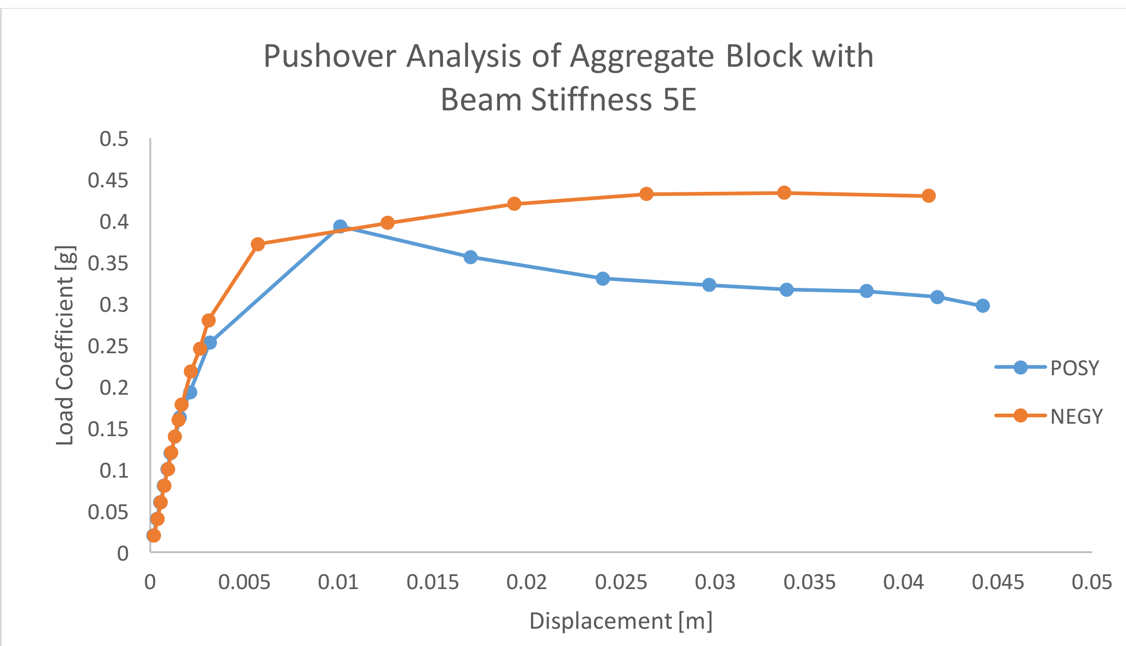


Figure 83: Capacity curve of Aggregate Block with beam stiffness of 5E in Y directions.

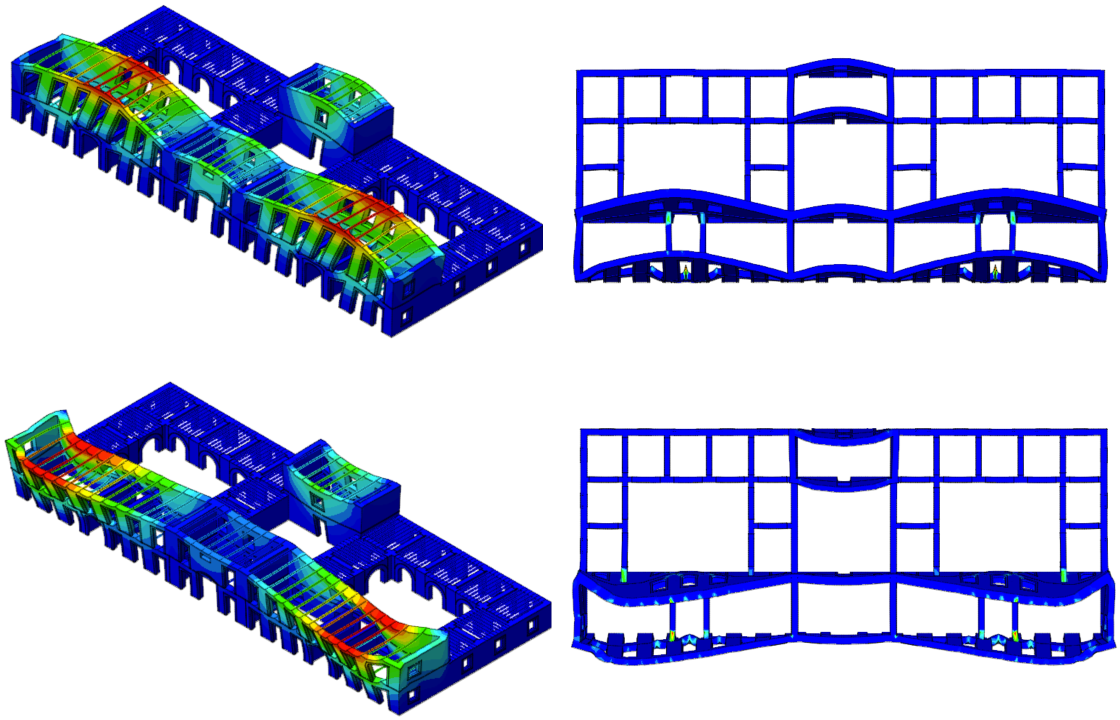


Figure 84: Pushover analysis of Aggregate Block with Beam Stiffness of 5E. (Top left) Deformed shape and (top right) maximum principal strain at the peak load in +Y direction. (Bottom left) Deformed shape and (bottom right) maximum principal strain at the peak load in –Y direction.

The maximum load coefficients of the Aggregate Block with timber beams with an elastic modulus of 5E are 0.39g and 0.43g in the +Y and –Y direction, respectively (Figure 83). In the +Y direction, the out–of–plane rotation of the façade and back walls of the Waterfront buildings cause small mild cracks at the intersection of perpendicular walls. In the –Y direction, minor cracks are occurred above the opening while the failure is triggered by the tensile damages between the façade and interior masonry walls at the ground floor, as well as between the pillars in the Salting Factory. Nonetheless, at the peak load, the displacement in the +Y direction is 10 mm, which lower than the 33 mm in the –Y direction.

Table 7: Comparison of peak load coefficient of the Aggregate Block in Y directions.

Direction	No beams	Beams with 2E	Beams with 5E
+Y	0.27g	0.32g	0.39g
-Y	0.35g	0.42g	0.43g

The failure modes and damage patterns observed in the Aggregate Block with timber floors of different stiffness are in some extent similar. However, due to the coupling effect of the beams and the associated stiffness, the results confirm the differences in terms of capacity and deformation ability. As expected, the increase in beam stiffness results a higher capacity for the aggregate under seismic action. Regarding the deformation ability, some differences were found but it would be important to further perform a more detailed analysis for defining limit states associated to certain levels of damage so that a more comparative analysis can be carried out. As illustrated in Figure 85 and Figure 86, the coupling effect provided by the beams results in lower displacement corresponding to the peak strength, which should be associated to a lower level of damage, as seen in the cases where the beams have higher stiffness (2E and 5E). This coupling effect also results in a higher mobilization of the transversal walls for the resisting mechanism, especially revealed by the higher displacements in the outermost transversal walls. It is clear when the deformation of the Waterfront Building with and without timber floors are compared and in particular when the seismic action is applied in +Y direction, which is associated to a lower capacity of the building.

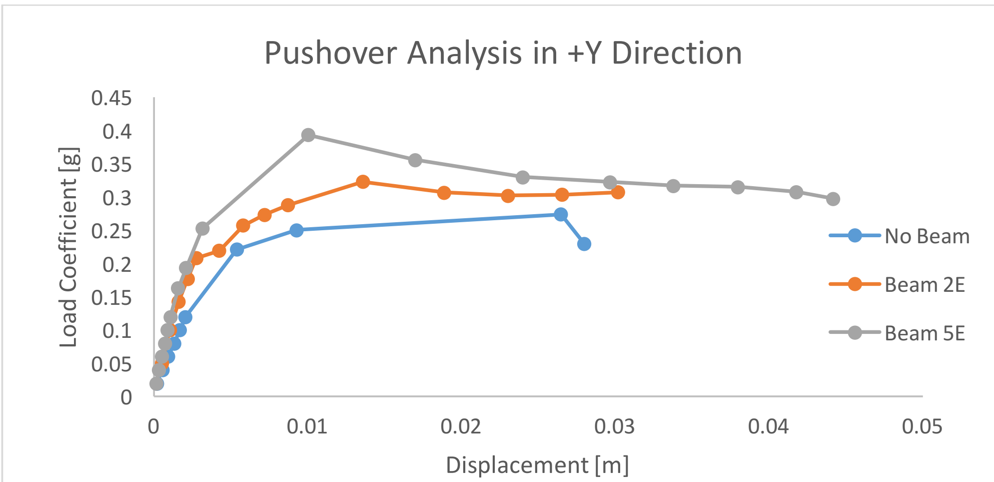


Figure 85: Pushover Analysis of different beam stiffness in +Y direction.

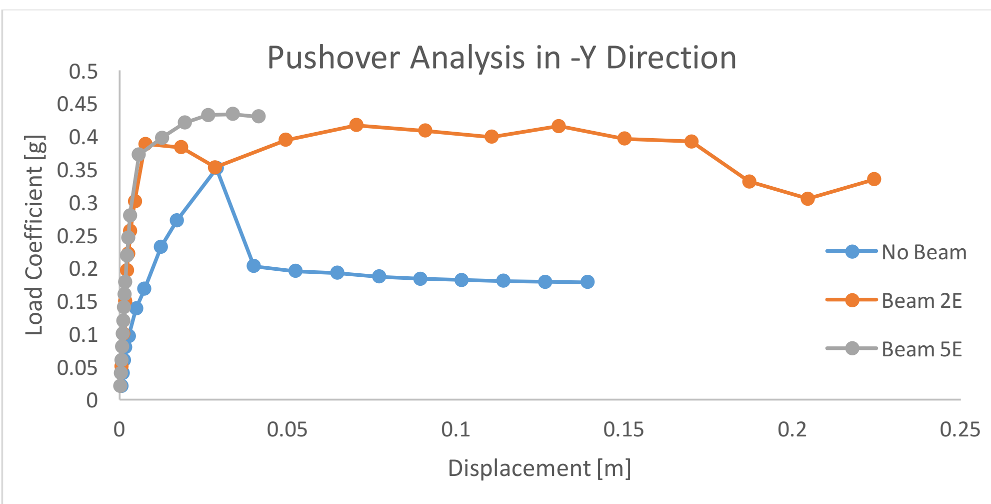


Figure 86: Pushover Analysis of different beam stiffness in -Y direction.

6.0 Conclusion

As vernacular buildings are non – engineering designed structures that evolved from previous models, its seismic vulnerability is often associated with inadequate interventions. Particularly for seismic – prone regions, where earthquake – resistant measures were already considered in the structural system at the time of construction. The change in building geometry, material properties and constructive techniques are the most influential aspects toward the seismic response of a structure, such as the change in mass and stiffness distribution, center of gravity and connection details. The seismic behavior of the type of structure that consists of different buildings, known as building aggregate, is further taking these considerations into a more interactive level. Each constituted building contributes to the global structural response, affecting and being affected by other adjoining buildings. Furthermore, the located urban environment should be taken into considerations since the neighboring structures can have a potential impact on the global behavior. The urban environment can be classified into four main types: seismic – oriented arrangement; historical center with organic matrix; scattered or rural environment; and conglomerated urban context.

In addition, for historical masonry buildings, it is of paramount important that the structure behaves as a "box" in seismic events. "Box – behavior" refers to the diaphragmatic action provided by the floor system, in which horizontal forces exerted by earthquake are able to be distributed into vertical elements with respect to their corresponding location and lateral stiffness. Timber floor structure in historical masonry buildings is usually considered as a flexible diaphragm, which exhibits a significant bending and shear deformation under horizontal forces. Although the presence of diaphragmatic action improves the structural behavior, it is difficult to generalize its effectiveness especially in the case of building aggregates due to the interdependent structural systems.

Nevertheless, there are two approaches for seismic mitigation: morphological and tectonic approach. Morphological approach pertains to the building geometry and stiffness distribution of a structure. On the other hand, tectonic approach incorporates different strategies or systems as seismic reinforcement, including the use of constructive techniques, perimeter seismic – resistant elements, utilization of arch systems, and combination of various reinforcing elements. These approaches can be divided into preventive or reactive measure, or as a combination of the two. Preventive measure refers to the resistant actions that anticipate future seismic events, where reactive measure aims to mitigate the already made damages and prohibits the structure from destruction. It is common that the Local Seismic Culture around the world embraces and adopts these approaches into their own construction details. In the Mediterranean region, one of the perimeter seismic – resistant measures, *loggia* is also used as an exterior architectural feature at the same time. The other prominent seismic resistant measure in this region is the use of timber frame at the upper stories or as a secondary structural system aside from masonry. The use of timber elements can also be seen in the Asian and Himalayan Local Seismic Culture. In Latin American, natural flexible materials such as plant fibers are often used in the mesh systems for walls, floors, and roofing structures.

The seismic – resistant system *Pombaline* building and its associated city planning are considered as the milestone of Portuguese Local Seismic Culture. This system was used across Portugal along with other seismic mitigation approaches. The city of Vila Real de Santo Antonio built after the 1755 Earthquake, exemplifies the emphasis on the practice of Local Seismic Culture. However, due to the low frequency of seismic activities in the region, the seismic – resistant measures have been erased from their intended functions.

The numerical analyses of the original Alfândega block demonstrated that the effectiveness of the traditional construction system in seismic events. As a building aggregate, its structural performance is highly dependent on the constituted buildings and their interaction. It is conservative to consider the constituted buildings in isolation. The global behavior of the building aggregate has a higher capacity to resist horizontal movements, but it should be noted that the behavior is greatly governed by the vulnerable parts within the structure as seen in the isolated cases.

Furthermore, the parametric study on the timber beam stiffness reveals a significant influence of the presence of floor system in structural behavior. The damage patterns and the deformation ability are dependent on the beam stiffness.

6.1 Future Works

The Alfândega block has been undergone numerous interventions in the last two centuries. Its current configuration is completely different than the original's. These modifications disregard the anticipated seismic – resistant systems and proposed a potential increase of vulnerability. In order to have a holistic understanding of various influences from different elements and structural systems of the present condition, parametric study regarding the numbers and layout of openings, position and relative height of the constituted and new additional buildings, and alternation of floor system should be studied in the future.

7.0 Reference

Amos, L.F. & Lourenço, P.B., 1999. Seismic Analysis of a Heritage Building Compound in the Old Town of Lisbon.

Calvi, G.M. et al., 2006. Development of Seismic Vulnerability Assessment Methodologies Over the Past 30 Years. *ISET Journal of Earthquake Technology*, 43(472), pp.75–104. Available at: http://home.iitk.ac.in/~vinaykg/Iset472.pdf.

Cardoso, R., Lopes, M. & Bento, R., 2005. Seismic evaluation of old masonry buildings. Part I: Method description and application to a case-study. *Engineering Structures*, 27(14), pp.2024–2035.

Carlos, G.D., Correia, M.R., Sousa, G., et al., 2015. Lisbon: Downtown's reconstruction after the 1755 earthquake. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 169–172.

Carlos, G.D., Correia, M.R., Rocha, S., et al., 2015. Vernacular architecture? In M. R. Correia, P. B. Lourenco, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 11–16.

Conservation, A., 2014. THE STAVE CHURCH AS A MEDIUM FOR THE.

Correia, M.R. & Carlos, G.D., 2015. Recognising local seismic culture in Portugal, the SEISMIC-V research. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 123–129.

Correia, M.R., Varum, H. & Lourenço, P.B., 2015. Common damages and recommendations for the seismic retrofitting of vernacular dwellings. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 241–244.

Dipasquale, L. & Mecca, S., 2015. Local seismic culture in the Mediterranean region. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 67–76.

Ferrigni, F., 2015a. The central and eastern Asian local seismic culture: Three approaches. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 77–81.

Ferrigni, F., 2015b. Vernacular architecture: A paradigm of the local seismic culture. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 3–9.

Fonseca, J.F.B.D. & Vilanova, S.P., 2015. Seismic hazard analysis: An overview. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 131–136.

Guerrero Beca, L.F. & Vargas Neumann, J., 2015. Local seismic culture in Latin America. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 61–66.

Lang, K., 1982. Seismic Vulnerability of Existing Buildings. *Meteorological Society*, 12(12), pp.1309–1313.

Langenbach, R., 2015. The earthquake resistant vernacular architecture in the Himalayas. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 83–92.

Lima, A., Viana, D., et al., 2015. *Local Seismic Culture in Portugal* M. R. Correia & G. D. Carlos, eds., Lisboa: Argumentum, Edicoes.

Lima, A., Correia, M.R., et al., 2015. Systematisation of seismic retrofitting in vernacular architecture. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 235–239.

Lourenço, P.B. et al., 2011. Analysis of Masonry Structures Without Box Behavior. *International Journal of Architectural Heritage*, 5(4–5), pp.369–382.

Mallardo, V. et al., 2008. Seismic vulnerability of historical masonry buildings: A case study in Ferrara. *Engineering Structures*, 30(8), pp.2223–2241.

Mendes, N. & Lourenço, P.B., 2013. Sensitivity Analysis of the Seismic performance of ancient masonry buildings. *4th International Conference on Computational Methods in Structural*

Dynamics and Earthquake Engineering, Kos Island, Greece, (June), pp.12–14. Available at: C:\Users\Usuario\Desktop\Paper\Literature\Mendes N., Louren?o P.B. - Seismic performance of ancient masonry buildings - a sensitivity analysis.pdf.
Ortega, J. et al., 2009a. Seismic behavior of an old masonry building in Vila Real de Santo António , Portugal. , 1774(Figure 1).
Ortega, J. et al., 2009b. Seismic behavior of an old masonry building in Vila Real de Santo António , Portugal. , 1774(Figure 1).
Ortega, J. et al., 2015. Seismic vulnerability of vernacular buildings in urban centres — the case of Vila Real de Santo António. *Seismic retrofitting: learning from vernacular architecture*, pp.219–226.
Ortega, J. & Vasconcelos, G., 2015. Seismic-resistant building practices resulting from Local Seismic Culture. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 17–22.
Ortega, J., Vasconcelos, G. & Lourenço, P.B., 2015. Seismic behaviour assessment of vernacular isolated buildings. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 203–212.
Paula, R.F. & Coias, V., 2015. Practices resulting from seismic performance improvement on heritage intervention. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architectures*. London: CRC Press, pp. 23–28.
Ramos, L.F. & Lourenço, P.B., 2004. Modeling and vulnerability of historical city centers in seismic areas: A case study in Lisbon. *Engineering Structures*, 26(9), pp.1295–1310.
Ramos, L.F. & Lourenço, P.B., 1999. Seismic Analysis of a Heritage Building Compound in the Old Town of Lisbon.
Shearer, P., A Brief History of Seismology.
Sousa, G., 2015. Portuguese historical seismicity. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 143–150.
Tobriner, S., 1983. La Casa Baraccata: Earthquake - Resistant Construction in 18th - Century Calabria. *Society of Architectural Historians*, 42, pp.131–138.
Varum, H. et al., 2015. Seismic behaviour of vernacular architecture. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 151–156.
Viana, D.L. et al., 2015. Systematisation of seismic mitigation planning at urban scale. In M. R. Correia, P. B. Lourenço, & H. Varum, eds. *Seismic retrofitting: learning from vernacular architecture*. London: CRC Press, pp. 229–234.
Vicente, R. et al., 2011. Evaluation of Strengthening Techniques of Traditional Masonry Buildings: Case Study of a Four-Building Aggregate. *Journal of Performance of Constructed Facilities*, 25(3), pp.202–216.

8.0 Appendix A - Modal Analysis

Appendix A.1 Modal Analysis of Central Building

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	6.42	0.00	0.00	0.00	95516.00	15.51	15.51
2	6.75	0.07	0.00	0.00	102000.00	16.56	32.07
3	8.37	127340.00	20.68	20.68	0.02	0.00	32.07
4	11.67	47851.00	7.77	28.45	0.00	0.00	32.07
5	12.20	167410.00	27.19	55.63	0.00	0.00	32.07
6	12.46	0.70	0.00	55.63	71.89	0.01	32.09
7	13.04	14815.00	2.41	58.04	0.00	0.00	32.09
8	13.66	5498.20	0.89	58.93	108.35	0.02	32.10
9	13.68	19588.00	3.18	62.11	26.72	0.00	32.11
10	14.89	9390.30	1.52	63.64	0.10	0.00	32.11
11	16.26	0.06	0.00	63.64	6791.10	1.10	33.21
12	16.79	6296.00	1.02	64.66	0.02	0.00	33.21
13	18.10	1.77	0.00	64.66	6982.70	1.13	34.35
14	18.25	0.07	0.00	64.66	68549.00	11.13	45.48
15	18.82	0.02	0.00	64.66	35172.00	5.71	51.19
16	19.30	30519.00	4.96	69.62	1.49	0.00	51.19
17	19.58	2.05	0.00	69.62	85552.00	13.89	65.08
18	21.36	90.49	0.01	69.63	6.86	0.00	65.08
19	21.86	1221.20	0.20	69.83	8.67	0.00	65.08
20	22.14	0.09	0.00	69.83	3476.50	0.56	65.65

Appendix A.2 Modal Analysis of Waterfront Building

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	19.24	0.04	0.00	0.00	9466.90	45.18	45.18
2	22.99	556.06	2.65	2.65	13.57	0.06	45.24
3	24.27	10.34	0.05	2.70	1.87	0.01	45.25
4	24.36	7816.00	37.30	40.00	0.73	0.00	45.25
5	25.96	3.09	0.01	40.02	21.10	0.10	45.35
6	26.10	3.65	0.02	40.03	0.12	0.00	45.35
7	26.13	12.03	0.06	40.09	0.04	0.00	45.36
8	26.22	4.51	0.02	40.11	1.26	0.01	45.36
9	26.27	0.31	0.00	40.11	0.47	0.00	45.36
10	26.32	1.24	0.01	40.12	0.01	0.00	45.36
11	26.42	2.42	0.01	40.13	0.99	0.00	45.37
12	26.61	9.62	0.05	40.18	0.46	0.00	45.37
13	26.70	0.25	0.00	40.18	0.00	0.00	45.37
14	26.77	1.49	0.01	40.19	0.38	0.00	45.37
15	26.81	3.72	0.02	40.20	0.35	0.00	45.37
16	26.83	2.92	0.01	40.22	0.00	0.00	45.37
17	26.84	7.89	0.04	40.25	0.00	0.00	45.37
18	26.87	0.13	0.00	40.26	0.08	0.00	45.37
19	26.93	0.31	0.00	40.26	0.00	0.00	45.37
20	26.94	3.05	0.01	40.27	0.11	0.00	45.37

Appendix A.3 Modal Analysis of Salting Factory

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	10.31	0.53	0.00	0.00	28661.00	6.93	6.93
2	15.44	750.20	0.18	0.18	1.24	0.00	6.93
3	16.94	6014.00	1.45	1.64	860.23	0.21	7.14
4	17.44	1915.90	0.46	2.10	53896.00	13.03	20.17
5	17.76	50718.00	12.27	14.36	3527.20	0.85	21.03
6	18.62	6857.90	1.66	16.02	1.39	0.00	21.03
7	18.85	42485.00	10.27	26.30	476.69	0.12	21.14
8	19.39	6385.20	1.54	27.84	39.22	0.01	21.15
9	20.29	22487.00	5.44	33.28	15625.00	3.78	24.93
10	20.56	4081.60	0.99	34.27	51995.00	12.57	37.50
11	20.75	126.14	0.03	34.30	3020.20	0.73	38.23
12	21.05	251.36	0.06	34.36	11.78	0.00	38.24
13	21.19	10.18	0.00	34.36	9433.40	2.28	40.52
14	21.54	3.23	0.00	34.36	13152.00	3.18	43.70
15	21.65	6798.80	1.64	36.01	389.09	0.09	43.79
16	22.31	267.83	0.06	36.07	384.09	0.09	43.89
17	22.78	947.62	0.23	36.30	3954.00	0.96	44.84
18	23.73	1652.10	0.40	36.70	12115.00	2.93	47.77
19	24.42	11511.00	2.78	39.48	4034.10	0.98	48.75
20	25.60	0.08	0.00	39.48	4022.60	0.97	49.72

Appendix A.4 Modal Analysis of Alfândega Block

Mode	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	6.09	19.04	0.00	0.00	249910.00	12.52	12.52
2	6.11	153.68	0.01	0.01	36585.00	1.83	14.35
3	6.51	0.00	0.00	0.01	95081.00	4.76	19.11
4	6.72	1.63	0.00	0.01	129030.00	6.46	25.57
5	7.08	2138.30	0.11	0.12	3849.50	0.19	25.76
6	7.16	178.80	0.01	0.12	42025.00	2.10	27.87
7	9.10	402530.00	20.16	20.28	0.17	0.00	27.87
8	9.98	83.53	0.00	20.29	36780.00	1.84	29.71
9	10.16	11198.00	0.56	20.85	25.10	0.00	29.71
10	10.39	23.54	0.00	20.85	37938.00	1.90	31.61
11	12.21	1630.40	0.08	20.93	0.09	0.00	31.61
12	12.81	77.87	0.00	20.94	0.02	0.00	31.61
13	12.97	1769.70	0.09	21.02	2.04	0.00	31.61
14	13.34	11.23	0.00	21.02	366.83	0.02	31.63
15	13.79	101170.00	5.07	26.09	6.23	0.00	31.63
16	13.94	259.91	0.01	26.10	5716.70	0.29	31.92
17	14.46	101850.00	5.10	31.20	2.61	0.00	31.92
18	14.50	181830.00	9.11	40.31	0.45	0.00	31.92
19	15.17	4.79	0.00	40.31	11472.00	0.57	32.49
20	15.71	3.29	0.00	40.31	70864.00	3.55	36.04

Appendix A.5 Modal Analysis of Alfândega Block without beams

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	4.65	0.08	0.00	0.00	99709.00	5.09	5.09
2	4.67	42.12	0.00	0.00	143.45	0.01	5.10
3	5.09	0.01	0.00	0.00	1354.20	0.07	5.17
4	5.18	543.84	0.03	0.03	0.00	0.00	5.17
5	5.63	0.00	0.00	0.03	15050.00	0.77	5.94
6	5.91	0.02	0.00	0.03	96677.00	4.94	10.88
7	6.33	0.00	0.00	0.03	74353.00	3.80	14.67
8	6.50	58.97	0.00	0.03	109870.00	5.61	20.28
9	6.53	671.53	0.03	0.07	13656.00	0.70	20.98
10	7.25	37.58	0.00	0.07	69404.00	3.54	24.53
11	7.64	17465.00	0.89	0.96	1185.00	0.06	24.59
12	7.83	142.55	0.01	0.97	54488.00	2.78	27.37
13	7.88	361.48	0.02	0.99	128.58	0.01	27.38
14	8.11	46.43	0.00	0.99	13662.00	0.70	28.07
15	9.19	379200.00	19.37	20.36	0.04	0.00	28.07
16	10.05	4.07	0.00	20.36	3328.80	0.17	28.24
17	10.64	359.41	0.02	20.38	14.08	0.00	28.25
18	10.77	8995.30	0.46	20.83	31.67	0.00	28.25
19	10.92	0.17	0.00	20.83	284.44	0.01	28.26
20	11.26	2.30	0.00	20.83	61105.00	3.12	31.38

Appendix A.6 Modal Analysis of Alfândega Block with beam stiffness of 2E

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	6.30	19.90	0.00	0.00	246910.00	12.37	12.37
2	6.32	187.30	0.01	0.01	31406.00	1.57	13.94
3	6.78	0.01	0.00	0.01	97439.00	4.88	18.82
4	6.80	0.55	0.00	0.01	161880.00	8.11	26.92
5	7.30	2185.70	0.11	0.12	4702.90	0.24	27.16
6	7.36	409.46	0.02	0.14	26463.00	1.33	28.49
7	9.16	408900.00	20.48	20.62	0.04	0.00	28.49
8	10.12	21.25	0.00	20.62	19640.00	0.98	29.47
9	10.39	9142.60	0.46	21.08	320.49	0.02	29.49
10	10.54	88.41	0.00	21.08	54239.00	2.72	32.20
11	12.28	1720.90	0.09	21.17	0.10	0.00	32.20
12	13.07	47.84	0.00	21.17	0.00	0.00	32.20
13	13.10	1075.60	0.05	21.22	1.60	0.00	32.20
14	13.54	11.81	0.00	21.22	425.66	0.02	32.22
15	13.98	115200.00	5.77	26.99	12.95	0.00	32.22
16	14.09	617.60	0.03	27.02	4686.40	0.23	32.46
17	14.56	188230.00	9.43	36.45	0.82	0.00	32.46
18	14.62	91810.00	4.60	41.05	3.68	0.00	32.46
19	15.35	4.02	0.00	41.05	11345.00	0.57	33.03
20	15.92	11.82	0.00	41.05	72767.00	3.64	36.67

Appendix A.7 Modal Analysis of Alfândega Block with beam stiffness of 5E

MODE	FREQUENCY	**EFF.MASS TX**	%	CUM.PERCENT.	**EFF.MASS TY**	%	CUM.PERCENT.
1	6.7081	14.774	0.00073988	0.00073988	208220.00	10.43	10.43
2	6.7407	275.54	0.013799	0.014539	14050.00	0.70	11.13
3	6.9671	0.056806	2.8448E-06	0.014542	250690.00	12.55	23.69
4	7.2948	0.019706	9.8685E-07	0.014543	103150.00	5.17	28.85
5	7.7432	2304.2	0.11539	0.12993	4905.20	0.25	29.10
6	7.8017	1076.2	0.053896	0.18383	12787.00	0.64	29.74
7	9.2702	422720	21.17	21.354	0.01	0.00	29.74
8	10.301	1.3808	0.00006915	21.354	2875.90	0.14	29.88
9	10.862	6510.4	0.32604	21.68	4300.80	0.22	30.10
10	10.938	422.32	0.021149	21.701	66924.00	3.35	33.45
11	12.341	1769	0.088589	21.789	0.08	0.00	33.45
12	13.313	107.67	0.0053921	21.795	0.50	0.00	33.45
13	13.572	9.8911	0.00049534	21.795	0.00	0.00	33.45
14	13.87	6.2819	0.00031459	21.796	840.27	0.04	33.49
15	14.261	139220	6.9721	28.768	58.62	0.00	33.49
16	14.309	4240.9	0.21238	28.98	2705.40	0.14	33.63
17	14.623	187900	9.41	38.39	1.31	0.00	33.63
18	14.891	65520	3.2812	41.671	6.93	0.00	33.63
19	15.549	2.437	0.00012204	41.671	5607.90	0.28	33.91
20	16.202	15.502	0.00077635	41.672	75488.00	3.78	37.69