

UPWARDS - VERTICAL EXTENSIONS OF MASONRY BUILT HERITAGE FOR SUSTAINABLE AND ANTIFRAGILE URBAN DENSIFICATION

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ABSTRACT

Urban densification represents one of the biggest universal challenges of contemporary cities: the increase of urban population requires new spaces to accommodate the growing demand for housing, working and tertiary activities. However, the land available for new constructions in highly urbanized areas is very limited. In this framework, the vertical extension of existing buildings is the most sustainable strategy and fascinating. Masonry structures, constituting a major part of the built stock in the historical city centres of several European countries, are particularly suitable for vertical addition of extra-floors, since they generally exhibit an adequate overstrength for bearing an increase of gravity loads, although seismic retrofit interventions are usually required. In this paper, starting from the case study of the historical town of Pozzuoli, in South Italy, an "antifragile" approach, based on the intermediate isolation system, is suggested to rise the height of the masonry building, while reducing the global seismic demand. For this aim, in this paper are analysed the architectural-functional and structural design issues related to a vast-scale application of vertical additions isolated on the top of existing structures. Further, the feasibility and effectiveness of the strategy are discussed, and benefits in terms of seismic performance are evaluated.

1. INTRODUCTION

European cities have a long history of vertical extension as obvious strategy of urban densification within the city fortification walls, as analysed in [1]. Building extra floors on top of existing buildings is an old idea, also degenerating in building abuses, unauthorized and parasite constructions, as the urban agglomerates that are heaped disorderly on top of the condominiums in Hong Kong [2].

Urban densification, as well, is one of the biggest universal challenges of the contemporary cities, particularly with reference to urban housing stock, which reveals inadequate to accommodate the increase of urban population growth. According to the United Nations [3] the world population living in urban areas, currently equal to 55%, will increase up to 68% by 2050. Human migration and global economic models push people to move to cities, since "*urban metropolises offer jobs, universities and cultural institutions …. sheer variety of things to do and see …. bring opportunities for wealth and the kind of collaborative creativity that has produced some of humanity's best ideas, including the industrial revolution and the digital age*" [4]. However, one of the cornerstones of sustainability is the containment of land consumption, formally set in Europe to zero net land take by 2050 as an explicit target in the context of the "Roadmap to a resource efficient Europe" [5] and in the document "The Future Brief: No net land take by 2050?" [6].

In this perspective, the problem of urban densification is today as urgent and constrained as in the past: large new spaces required for the increasing population - but no land available for new constructions. The solution to the problem can be found by adopting the same strategy as in the past, i.e. by looking at the building stock as a resource, by thinking at the building roofs as construction sites, and by planning the new required housing and working space as a new city above the existing city. It is the very fabric of existing buildings, especially the ones that use long lasting, durable materials, like masonry, that should be regarded as natural resources. In this perspective, all buildings are worth saving, not only architectural landmarks or historical monuments; also modest, vernacular and often unremarkable buildings have roles to play, and "*it would be criminal to ignore existing building inventory as an opportunity for reuse*" [7].

A sustainable urban growth therefore requires a paradigm shift towards the exploitation of the existing buildings for reuse, requalification and extension. As underlined by [1] this approach "*is no longer a question of idealism, but of resources and economics*". Within this new paradigm, vertical extension seems one the most promising strategies, with a large potential of space for building on the rooftops in cities [8, 9, 10]. Several European cities, are moving toward this direction. An exemplary case is London: on the basis of a series of studies carried out in the past few years for assessing the capacity for new rooftop homes [11, 12, 13], the Ministry of Housing, Communities and Local Government proposed changes to the National Planning Policy Framework (NPPF) to make it easier to add up to two storeys to existing buildings. The Mayor's Draft New London Plan, as well, recognised that London boroughs should be identifying sites suitable for "*delivering residential above existing commercial, social infrastructure and transport infrastructure uses*" [14].

In this paper, masonry structures are selected as the optimal candidate for vertical extension. In Italy and in several European countries, in fact, the built heritage is rich of masonry structures in the city historic centres, where, on the other hand, the high demand for real estate and the scarcity of land for new constructions would suggest the increase of square footage of existing buildings through vertical addition of extra-floors. However, some specific issues concerning structural design are worth of discussion and investigation. First, the existing structure should exhibit an adequate overstrength for bearing the increase of compression forces due to the addition gravity load. Second and foremost, in regions characterized by medium/high seismic hazard, as in Italy and all Mediterranean Europe countries, the mass of the vertical addition modifies, and generally increases the seismic demand on the existing structure. Finally, a further aspect that should be analysed is the suitable material and structural system for erecting the extension quickly, easily, efficiently and sustainably.

With reference to the first point, it should be reminded that masonry is characterised by relatively large compression capacity (and almost no tension resistance). Further, structural elements such as walls and columns are generously sized in well-designed masonry buildings and usually have compression strength demand to capacity ratio quite low, between one fifth and one tenth [e.g.: 15, 16]. Therefore, masonry buildings are more than likely able to accommodate the extra gravity loads due to the upper extension.

Concerning the seismic loads, masonry structures can exhibit a poor response and require seismic retrofit interventions; the mass addition due to the vertical extension would increase the seismic demand on the structure, thus further worsening its working conditions. This aspect may significantly affect the feasibility of vertical extensions in regions characterized by seismic hazard. However, a novel approach is here proposed for the vertical extension, taking inspiration from some recent projects utilising the technique of seismic isolation [17, 18, 19, 20]. In such cases, a vertical addition equipped with seismic isolation at its base has been erected on the roof of the existing building: this configuration has proved to serve not only for expanding the building volume, but also as a valid retrofit strategy. Indeed, the isolation system can be designed for converting the vertical addition into a huge mass damper, with remarkable reduction of the seismic demand on the structural complex, even with respect to the standalone existing building. Actual applications of this approach have been realized by adding a steel vertical addition on the top of an existing reinforced concrete building: the two-storey Musashino City Disaster Prevention and Safety Center [19, 20], in Tokyo, extended by a five-storey steel superstructure; the three-storey 185 Berry St. building, located in the China Basin area of San Francisco [17, 18].

It is worth underlining that no similar application has ever been achieved for masonry building; however, two peculiar additional advantages can be obtained by means of this retrofit strategy in the case of masonry. First, the increase of compression forces due to gravity loads gives rise to a global improvement of the building capacity, i.e. a stabilizing effect that reduces, or even neutralises, the tensile local demand that arises in the walls due to lateral load bending and shear effects. Second, exploiting the mass damper effect, the seismic global demand (in terms of bending and shear effects) on the existing building and its foundations decreases as well. Therefore, massive retrofit interventions in the lower structure can be strongly reduced, or avoided at all, and the extension works can be carried out with almost no disruption of the activity hosted in the existing building. The authors would like to emphasize that this paper does not focus on a specific case-study, rather it is a proof-of-concept study that addresses the issue of the vertical extension of masonry buildings by suggesting an innovative procedure to be applied at a district scale, or even at the city scale, by considering both architectural and structural aspects. However, once a specific building is selected, a preliminary investigation on the masonry structure is necessary to evaluate the possibility of adding the vertical extension equipped with seismic isolation. In particular, it is necessary to ensure that a rigid diaphragm is present at each floor, and that orthogonal walls are effectively connected each other, thus avoiding local modes and out-of-plane displacements in the substructure, thanks to the box-type behaviour.

With reference to the third aspect, i.e. the structural material and typology, an in-depth study, mainly consisting in steel structural solutions, is carried out in another paper [21].

This paper offers the conceptual framework for approaching the problem of vertical extension for masonry buildings in seismic zones, with design solutions that have the potential of reducing the seismic effects on the existing structure with respect to the as-is configuration. In this perspective, the proposed applications can be ascribed to the category of "antifragile" interventions. The concept of "antifragility", the antonym of fragility, was introduced by Taleb [22] with specific reference to the finance/economy world, and then extended to other contexts [23]. Antifragility is something more than robustness and resilience: it describes the quality of a system which, not only is unharmed by adverse events, but it is actually strengthened by them. In this light the vertical extension with seismic isolation can be interpreted as an antifragile intervention: the increase of building mass due to the extension, which is expected to increase the damage potential at earthquake occurrence, instead, gives rise to an improvement of the seismic behaviour of the building with respect to the as-is configuration.

In a previous paper [24], we have selected a masonry building in Gulianova (central Italy) as case study for testing the idea of a steel superstructure, equipped with seismic isolation system at its base. By adopting several design solutions, we also demonstrated that a carefully design of the isolation system and the vertical addition allows also to regularize the dynamic behaviour of the existing building, thus nullifying the detrimental effects of possible eccentricities in the lower structure.

In this paper, the same idea is proposed for Pozzuoli, a town of the province of Naples, south Italy. As will be discussed in the following section, Pozzuoli can be defined "an antifragile city", due to a long history of cyclic disasters and resurgences, thus the application of building interventions aimed to antifragility seems both desirable and suitable. Further, a wide plan for reconstructing rooftop volumes, previously demolished, has been recently issued, therefore a number of buildings can be identified as case studies.

This paper provides the analytical framework for dealing with the expansion of masonry buildings through vertical extension that activates the mass damping mechanism. The potential improvement of the buildings arising from the extension is analysed from different points of view, namely in terms of architectural language, relationship to the urban context, and seismic response.

While the city of Pozzuoli is here considered, the strategy can be applied to more general cases. Indeed, this study can be extended to other European cities, whenever the needs for vertical extension and seismic retrofit simultaneously arise for the building stock. As previously discussed, this type of intervention is particularly suitable for masonry buildings, which are usually oversized with respect to vertical loads and constitute a major part of the architectural heritage of historic European cities. Moreover, when urban regulations allow or even encourage the extension of existing buildings, the vertical additions should be conceived according to the guidelines on architectural heritage (e.g. Icomos-iscarsah [25]). To this aim, the design solutions here proposed are perfectly inserted in the surrounding urban fabric and, by using a different architectural language, ensure the legibility and the distinguishability of the new parts with respect to the existing ones. From the structural point of view, the innovative seismic retrofit technique determines a significant reduction of the conventional structural interventions that would be necessary on the substructure, thus minimizing the impact on the historical value of the architecture. Finally, the use of steel as a structural material for the extension is a sustainable choice that respects the principles of compatibility and reversibility of the intervention (the steelwork can be easily assembled and then disassembled).

Based on the above considerations, the authors think that the present study can be a catalyst for future debates on sustainable and resilient, or even antifragile, urban densification.

2. THE CITY OF POZZUOLI

2.1 Historical relationship between seismicity and urban configuration of Pozzuoli

Pozzuoli (Figure 1) is a coastal town located in the metropolitan area of Naples (Sothern Italy), founded by the Greeks with the name of *Dicearchia* (528 B.C.), and then integrated within the Roman Empire (196 B.C.) with the name of *Puteoli*. It is located in the in the area of *Campi Flegrei (Phlegraean Fields*), a complex volcanic landform (also described as a "super-volcano") which covers an area of about 400 km^2 and is characterised by the slow subsidence and uplift of the ground, registered since the Roman times. Known as a bradyseism (or “slow earthquake”), evidence of the movements is preserved in Pozzuoli, on three marble columns in the Roman marketplace called Macellum (misidentified as a temple of Serapis, Figure 2); here, bands of boreholes left by marine Lithophaga molluscs on the marble columns show the variation of the site level with respect to the sea. Due to this persistent volcanic activity, with long phases of bradyseism and some eruption events accompanied by intense seismic activity, the history of Pozzuoli represents “a rare example of the long-lasting resilience that enabled a port city to react to a long series of disasters and rebuildings” [26]. Therefore, it is interesting to retrace the main events, reported in detail by Colletta [26] and here briefly recalled, which Pozzuoli had to face during its history of antifragile city.

The ancient *Dicearchia* was a landing place and a maritime trade centre, with activity strictly related to the port system of the nearby colony Cuma. Its *Acropolis*, now *Rione Terra*, was an important public and religious point in a commanding position overlooking the bay, used as natural harbour. As *Puteoli*, in the Roman age, the town grew enormously and became the major port-town of the entire Roman Western Empire. Then, in the second century A.D. the city experienced a phase of decline, also caused by the occurrence of a period of intense *bradyseismic* activity.

Figure 1: The bay of Pozzuoli.

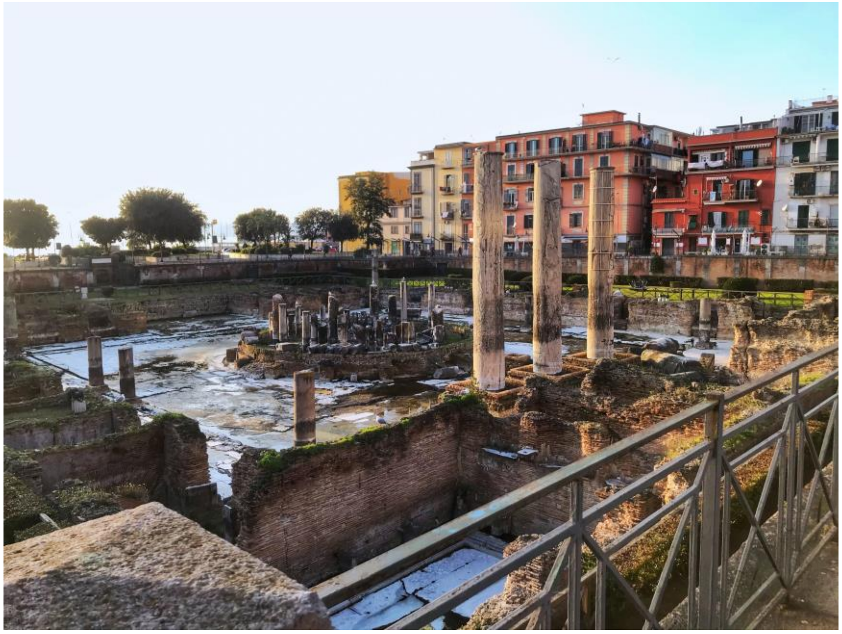

Figure 2: The ruins of the Roman building of Macellum in Pozzuoli, known as Serapeion Templum.

Between the 9th and 10th century Pozzuoli reached the maximum level of submersion, 6.30 meters at the site of *Macellum.* The natural and historical landscape of Pozzuoli and *Campi Flegrei* territory completely changed in the Medieval Age: the city gradually contracted until it was no larger than the ancient Acropolis, and became a defensive fort along the coast line of the Kingdom of Naples. The whole area cyclically underwent profound changes due to bradyseismic activities, which occasionally become more intense. Remarkable upward movements occurred starting from the beginning of the 11th century until 1456, when an earthquake completely destroyed the Rione Terra.

The most macroscopic event happened on the 28th September 1538, with a land emersion of 7.40 meters on the coast of Pozzuoli and the consequent eruption of Monte Nuovo, which, in turn, caused earthquake and seaquake. Subsequently, the Spanish viceroy Pedro de Toledo rebuilt the centre of Pozzuoli on the new land emerged from the sea, thus promoting the revival of the port city in both demographic and economic terms. This new *vice-royal district*, called "*Borgo Nuovo*", was organized in a regular plan, subdivided into three blocks, with orthogonal streets around a large rectangular square that overlooked the new port of the city.

During the 20th century, significant bradyseismic phenomena and related seismic events occurred twice, between the 1969 and 1972, and in the period from 1982 to 1985. During the 1969-72 crisis, damage and instabilities were registered both in the Rione Terra and in the vice-royal district. In particular, the extensive damage exhibited by old and already deteriorated buildings, as well as the immediate threat of collapse detected for several structures, pushed the authorities to evacuate the whole area of Rione Terra, and to transfer the population in the neighbourhood Rione Toiano, newly designed by the engineer Luigi Cosenza. During the subsequent crisis, in the years 1982-1985, the ground uplift reached 1.79 m, and several seismic events of different intensities were registered, producing new severe damage to historical and architectural heritage. In this case, a plan of emergency demolitions was issued, with the aim of decreasing density by 50% in the vice-royal district area, mainly carried out by tearing down the upper floors of several buildings rather

than whole constructions. A new neighbourhood, the Monterusciello district, was designed and built within an agreement between the City of Pozzuoli and the University of Naples Federico II. The construction work only lasted one year and 650 new properties were delivered to house the evacuated population.
It is worth underlining that the bradyseism is a phenomenon strictly related to the volcanism, and the mechanisms producing the bradyseismic crises are still under study, with conflicting interpretations highly debated in the scientific community [e.g.: 27, 28, 29, 30]. There is no doubt that volcanic, bradyseismic, and seismic activities are closely connected. As reminded by [31], seismicity occurring in volcanic area is generally characterized by earthquakes with occurrence frequency and magnitude values dependent on the state of the volcano, e.g.: the location and upward mitigation of the magma, the hydrothermal fluid circulation, the induced fracturing phenomena, the local stress variation. However, though volcanic risk is the main concern for the Campi Flegrei zone, earthquakes can represent a threat for the inhabitants and the building structures in the highly densely populated area of Pozzuoli [31], as demonstrated by the evacuation plans and demolition works made necessary in the aftermath of the crises of 1969-1972 and 1982-1985. Therefore, while a multi-hazard approach should be adopted for a thorough assessment of the risk of the city of Pozzuoli and entire zone of Campi Flegrei, in this paper a focus on seismic hazard is adopted for addressing the problem of vertical extensions aimed to reconstruct the demolished floors in the vice-royal district, according to the Land Use and Urban plans, as discussed in the following subsection.

2.2 The reconstruction (Land Use and Urban Plans)

As reported in the previous section, the partial and total demolition works issued in the 80s caused a major loss of built heritage of the city of Pozzuoli. Therefore, in 2002 the city's government drew up a general Land-Use Plan (in Italian: *Piano Regolatore Generale*, P.R.G.) [32] that established the reconstruction of the building volumes demolished in the 1983 – 84, for a total of 840 rooms, 100 for residential occupancy and 740 for tertiary activities (yellow area in Figure 3). Then, accounting for the provisions of the P.R.G., a more detailed Urban Implementation Plan (in Italian: *Piano Urbanistico Attuativo*, P.U.A.) was approved in [2015] , specifying the modality of rebuilding the demolished volumes (appointed as *re-composition*) for a limited area of Pozzuoli (area delimited by red line in Figure 3). In particular, the P.U.A. refers to three zones, also depicted in Figure 3: zone A1, Rione Terra; zone A2, Vice-royal district; zone B1, residential area around Corso Umberto I. It is worth noticing that the buildings considered for re-composition in the P.U.A. are mostly located in the historical part of the city (zones A1-A2), with only some individual elements falling in the zone B1. In most cases, the volumes to rebuild should have non-residential use. This choice is strategic: the main goal of the P.U.A. [2015] is to create a hub of activities able to promote the strengthening and the development of sectors such as tourism, trade, leisure, wellbeing and sport [33]. However, this purpose is strictly connected to the need of restoring the ancient aesthetic dignity of the building façades that even today appears injured by the emergency demolition measures.

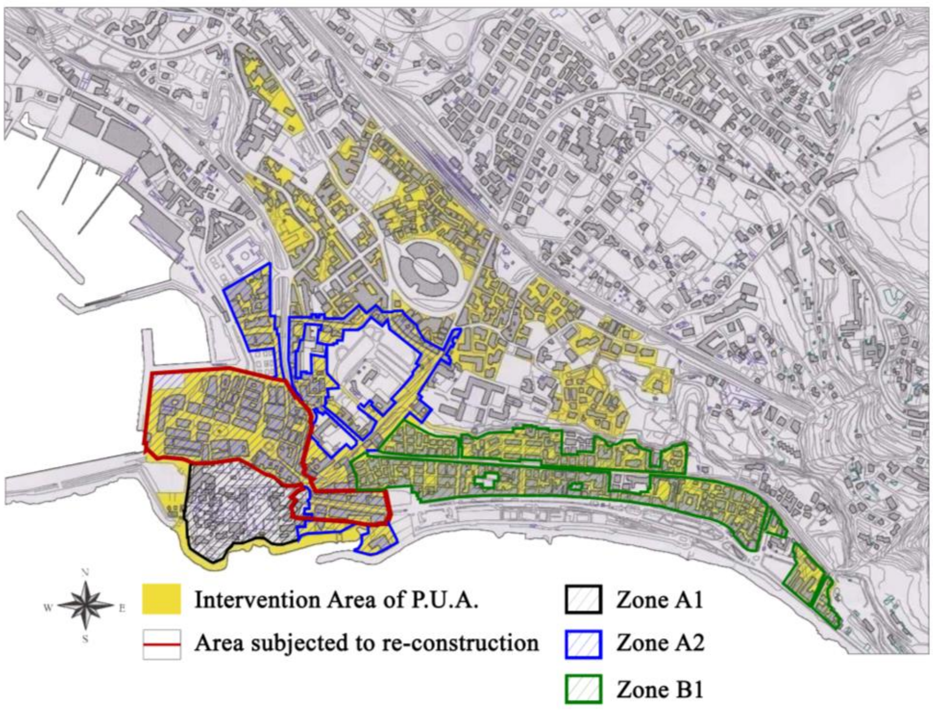

Figure 3: P.R.G. and P.U.A. areas of intervention.

3. THE INTERVENTION AREA AND THE PROPOSED APPROACH

The *vice-royal district* of Pozzuoli, area with the red outline in Figure 3, has been selected for applying the proposed approach concerning the vertical addition in seismic zones. The intervention area of the P.U.A. [34] is depicted in Figure 4 (top) with the urban profiles (sections from A-A' to E-E') identifying the buildings object of the volumetric re-composition. (Figure 4, bottom). The sections refer to: A-A' and B-B', "Corso della Repubblica"; C-C', "Cosenza street"; D-D', "Giovanni de Fraia street"; E-E', "Corso Giuseppe Garibaldi"; F-F', "Matteotti street"; G-G', "San Paolo street". Data of the relevant buildings are provided in Figure 5 in terms of: floor area, number of existing and demolished storeys, number of storeys to rebuild. From Figures 4 and 5 it can be noted that several buildings should be subjected to volumetric re-composition; the number of demolished storeys and storeys to rebuild varies between 1 and 3, and the floor area varies between 50 and 600 m^2. It is also worth observing that, for each building, the number of storeys to rebuild is generally less than the demolished ones in order to regularize the urban profiles through restored buildings with almost the same height.

The re-compositions planned in the P.U.A. provide new volumes with the same architectural style of the pre-existences, dating back to the 16^{th}-17^{th} centuries, thus conferring to the historic centre of Pozzuoli its ancient form (i.e. "how it was, where it was"). For example, it is proposed to use

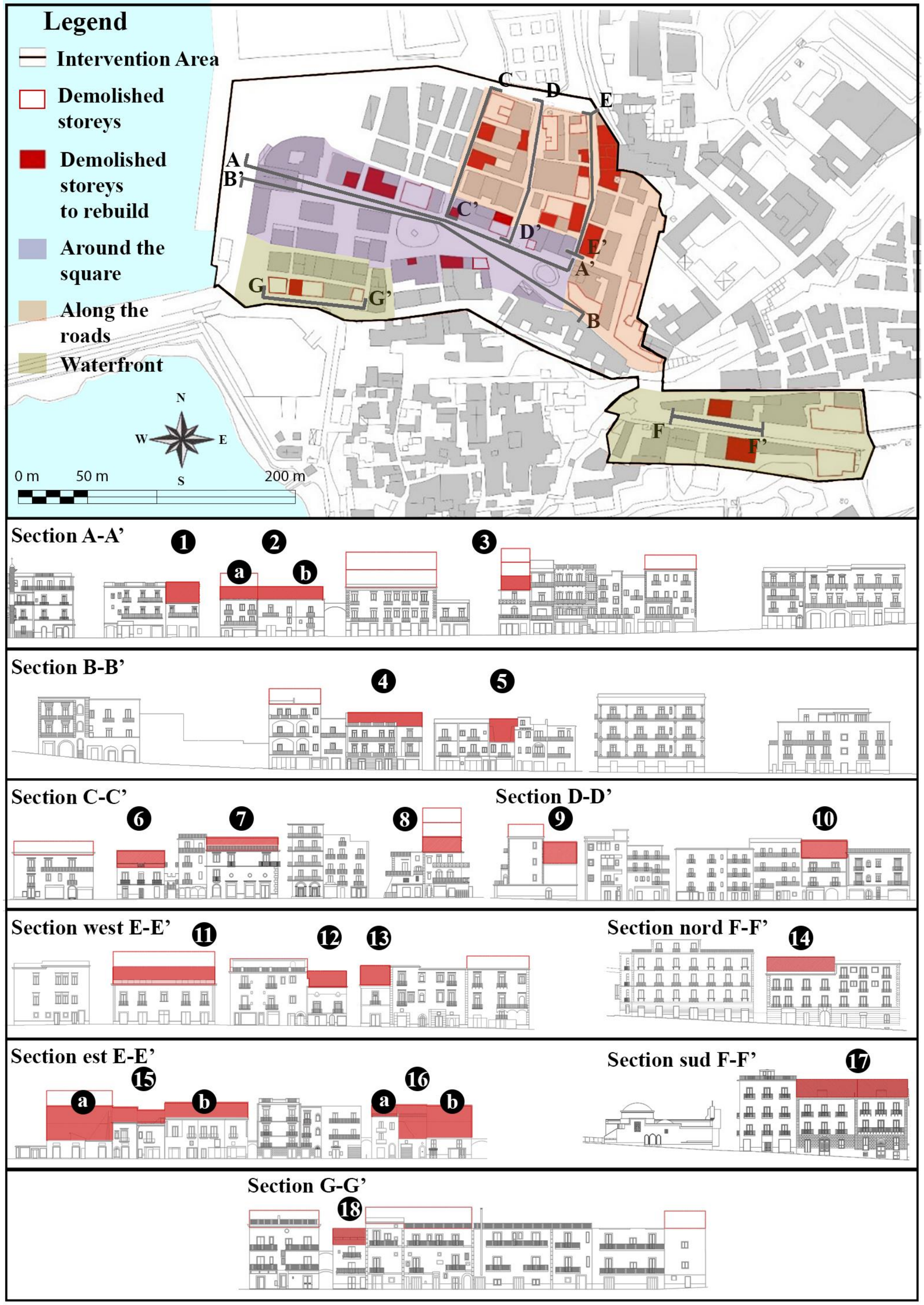

Figure 4: P.U.A. of Pozzuoli: intervention Area under investigation (top), most representative urban profiles (bottom).

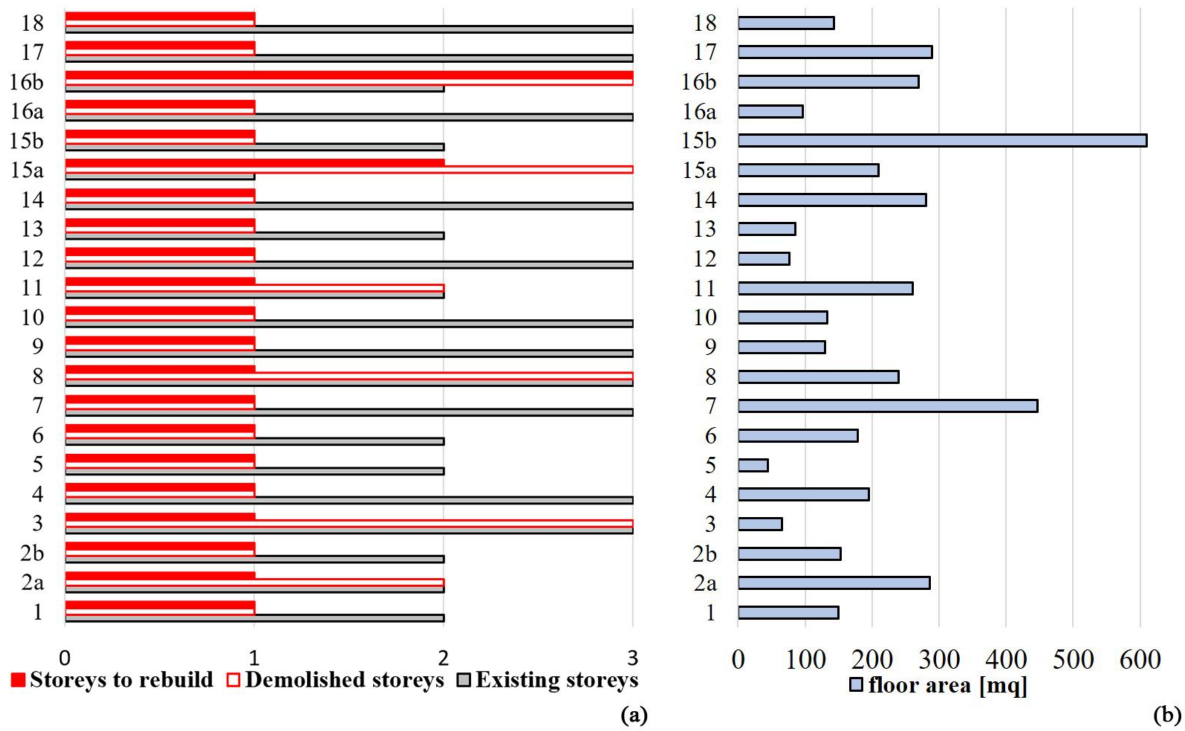

Figure 5. Data of the buildings subjected to volumetric re-compositions (see Figure 4): (a) number of storeys, (b) floor area.

the same plasters, friezes, colours, layout of façade openings, inter-storey heights. This strategy makes impossible reading the new interventions. In the present paper, an alternative strategy is proposed, with vertical extensions that are intentionally designed to be legible from both architectural and structural point of view, namely with contemporary architectural forms clearly denouncing the intervention time, and structural systems and techniques that radically (but advantageously) modify the behaviour of the extended structural complexes with respect the as-is configurations.

3.1 The architectural-design approach and the connection with the urban space

From the **architectural point of view**, starting from the idea of accounting for the location of the building within urban tissue, some composition rules are identified. In the site plan provided in Figure 4 (top) three different locations are recognised, namely *around the square*, *along the roads*, *waterfront*; i.e.:

- **Around the square** (Figure 6, part 1): the street frontage is recomposed by resuming the façade alignment, guaranteeing a *front reading* of the urban profile overlooking the large space of the square, i.e. “Piazza della Repubblica” (sections A-A’ and B-B’ in Figure 4).
- **Along the roads** (Figure 6, part 2): the vertical extensions on the narrowed axes are emptied ensuring a wider opening on the road (sections C-C’, D-D’, E-E’ in Figure 4). The re-composition is characterized by a *tangential reading*, therefore, in some cases the addition is hidden from the main street.

- **Waterfront** (Figure 6, part 3): the vertical additions overlooking the coast become a *landmark* with distinctive elements, dizzying heights and unusual forms (sections F-F', G-G' in Figure 4).

In this framework, the vertical additions on existing buildings modify the contemporary landscape. Therefore, by interpreting the latest trend in architectural design, they assume the typical attributes of "parasitic" architectures. Recently, parasitic vertical additions can be observed in the housing areas as new architectural tendency. Astonishing colours, forms and textures contrasting with the host-building allow to achieve comforting yet distinctive additions. By refusing the traditional schemes of architectonic composition, these new volumes incorporate the core of existing building as true "parasites" and define one cohesive unit with it [35].
Hence, by choosing parasitic vertical extensions able to dialogue with the urban context, three different themes are identified, namely *the urban corner*, *the completion as urban texture repair*, *the reconfiguration of the façades*; i.e.:

- **The urban corner**: it refers to corner buildings, located at the intersection of road axes. For highlighting the edge, the re-composition starts by adding the new volume, and proceeds by subtracting a part of such volume in correspondence to the adjacent existing buildings (Figure 7, part 1). This theme has been inspired by the project of the Casa Lude [36], a corner addition in Cehegìn (Spain), designed by the Grupo Aranea architects.
- **The completion as urban texture repair**: the vertical addition is designed as urban texture repair able to fill the gap between the adjacent existing buildings and the one to be extended. The added volumes, reflecting the inter-storey heights of the lower buildings, are treated with operations of subtraction and extrusion to redesign in a modern way the profiles of the street frontages (Figure 7, part 2). A similar approach has been used for the vertical expansion of a building at "viale Monte Grappa" in Milan (Italy) [37], where the designers (Westway Architects) conceived a new volume that fills the empty space between the two adjacent buildings.
- **The reconfiguration of the façades**: it takes place by adding the new volumes either *in parts* (Figure 7, part 3a), if the building heights are very different from each other, or *in continuous way* (Figure 7, part 3b), if the building heights are comparable. Therefore, the street front is relooked with a new distinctive image, as for example in Treehouses Bebelallee in Hamburg (Germany) [38], designed by the Blauraum Architects.

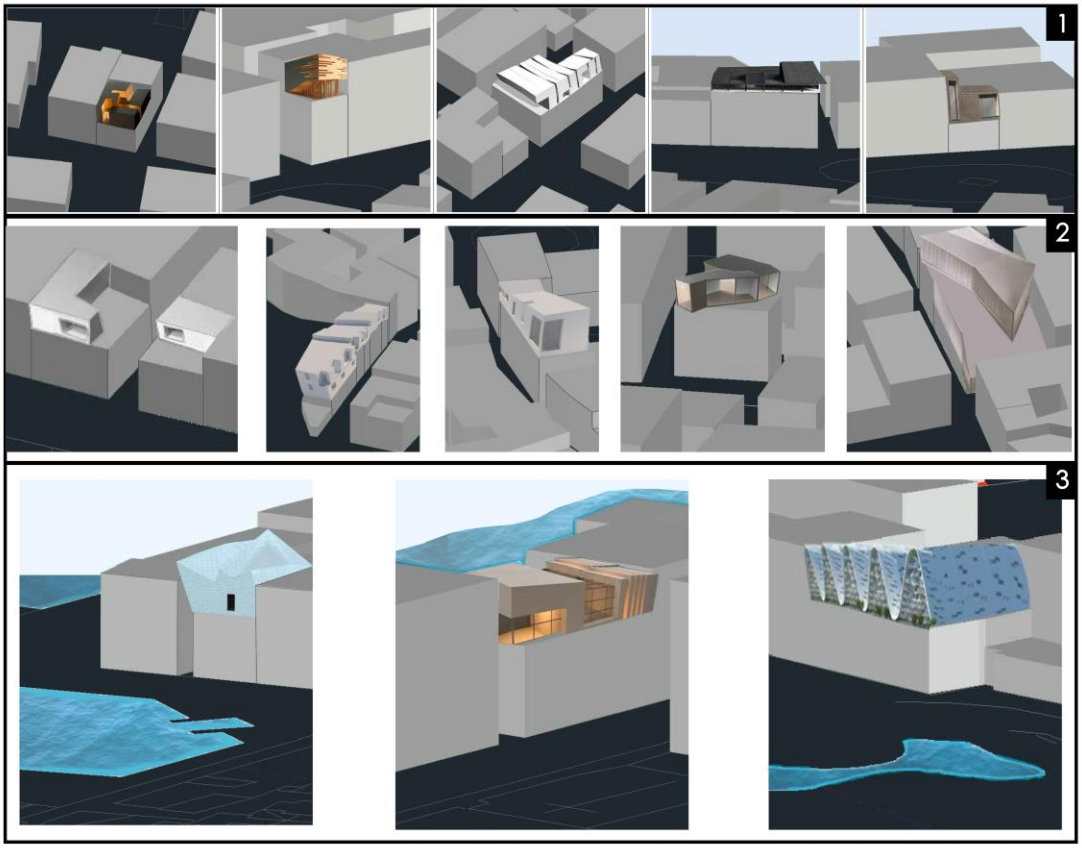

Figure 6: Design approach for the three different zones: around the square (1), along the roads (2) and the waterfront (3).

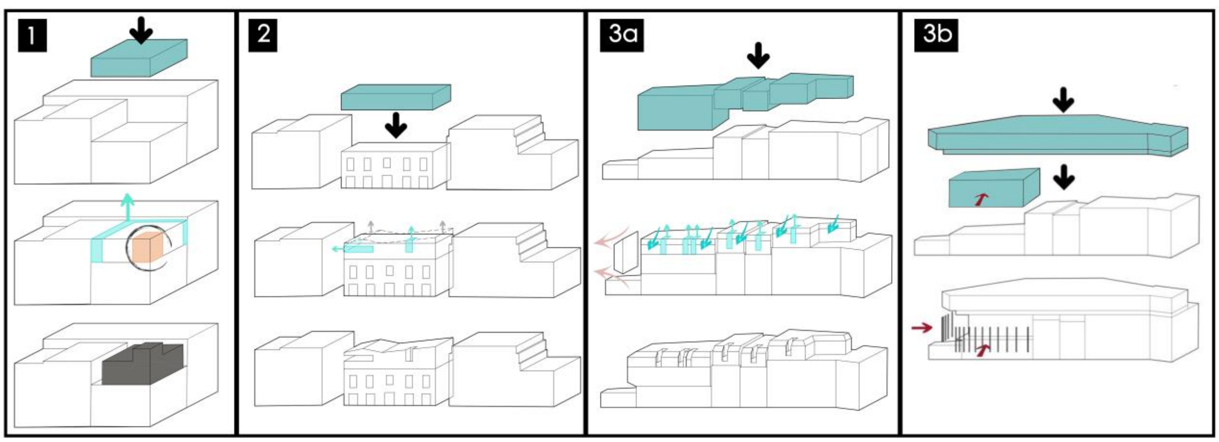

Figure 7: (1) the urban corner; (2) completion as urban texture repair; (3a) reconfiguration of the façade in parts; (3b) reconfiguration of the façade in continuous way.

3.2. The structural-design approach

From the **structural point of view**, the vertical addition is realised by erecting a steel structure, equipped with seismic isolation system at its base, on the roof of the existing masonry building (Figure 8b).

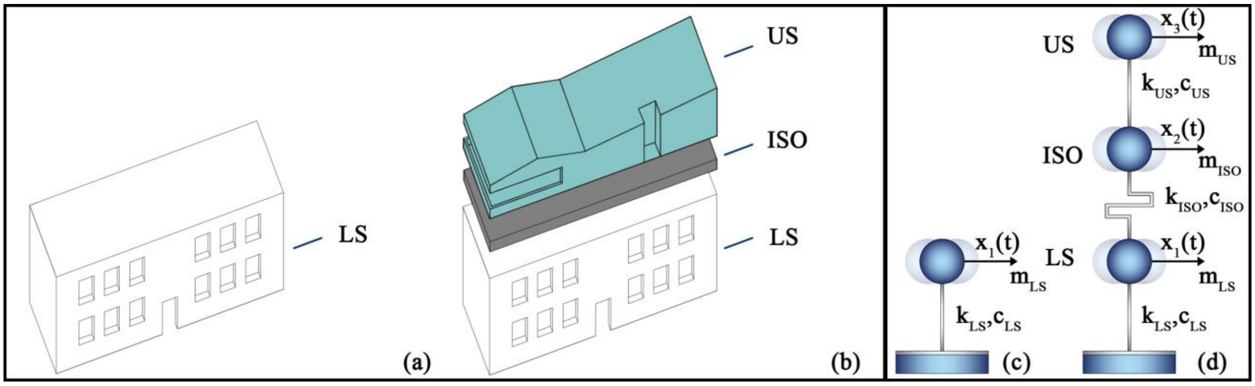

Figure 8: (a) existing masonry building, (b) retrofitted building with intermediate isolation, (c) simplified SDOF LS model, (d) simplified 3DOF IIS model.

As already mentioned in section 1, two major structural aspect should be addressed in order to check the feasibility of the vertical extension, namely: the needed bearing overstrength of the masonry walls and building foundations, and the proper account of the modified seismic demand on the existing structure.

With reference to the first problem, it is considered that: 1) the masonry buildings of the vice-royal district in Pozzuoli present walls (about 75 – 80 cm thick) and foundation structures designed for bearing all the gravity load acting before the demolition (1 to 3 demolished storeys); 2) by adopting a steel structure, the storeys to rebuild (between 1 and 3) are lighter than the demolished masonry storeys; 3) well-designed masonry buildings have compression strength demand to capacity ratio between one fifth and one tenth [e.g.: 15, 16]. These considerations imply the preliminary static feasibility of the extension, even with number of storeys greater than to the ones established in the P.U.A. [34].

For the second problem, the use of seismic isolation at the base of the new structural addition is here proposed. Properly designing the seismic isolation system, accounting for the dynamic properties of the existing lower structure and of the new upper structure, a counter-intuitive behaviour can be achieved for the structural complex, namely a non-increase or even a decrease of the seismic effects on the extended building and its foundation structures with respect to the as-is configuration (Figure 8a). In fact, the isolation interface may be designed for completely disconnecting the existing building from the new volume, thus not varying the seismic demand, or for connecting the two structural part in a controlled mode that converts the vertical addition into a huge mass damper. From the technical point of view, the isolation layer is realized as a conventional base isolation system, e.g. with high damping rubber bearing devices. More specifically, the isolation bearings are sandwiched between two steel beam-grillage, with the bearing at positions corresponding to the cross-section centroid of the underlying masonry piers.

The steel beam grillage is mounted with the aim to support the upper structure, transfer the gravity loads to the isolation bearings, and then to the masonry piers, and to make the isolators work as a parallel system under seismic actions.

For evaluating the amount of such added mass, the ratio between the new and existing mass can be utilised. In particular, by assuming as mean values of gravity loads 20 kN/m^2 and 7 kN/m^2 respectively for the masonry and steel buildings, and by utilising the data provided in Figure 5 (i.e. 1 – 3 storeys to rebuild with a floor area of 50 – 600 m^2), the values of the mass ratio, varying between 0.1 and 0.4, are derived.

The existing masonry structure equipped with the isolated vertical extension is configured as an intermediate (or inter-story) isolation system (IIS). The dynamic behaviour of IIS can be preliminary evaluated by utilizing simplified models. In fact, in IIS buildings the isolation interface ideally divides the structure into three parts: the masonry lower structure (LS), the isolation system (ISO), the steel upper structure (US) (Figure 8b). Each part of the structure can be described by a lumped mass, thus, obtaining a simplified three-lumped-mass (3DOF) model [39, 40, 41] (Figure 8d). For comparative purpose, the single degree-of-freedom model of the lower structure (SDOF LS) is also considered (Figure 8c). In particular, with regard to the 3DOF IIS model provided in Figure 8d, the mass, stiffness and damping of the degrees of freedom, are referred to respectively as *m*, *k* and *c*, while the subscripts *LS*, *ISO* and *US* indicate respectively the lower structure, the isolation layer and the superstructure. It is worth noting that the mass m_{ISO} is made up by two parts: the mass of the grillage of steel beams installed on the top of the existing structure and the mass of the isolation devices (high damping rubber bearings). Indeed, it should be underlined that the mass of the sole isolation devices represents a small percentage of m_{ISO}, while the main amount is the mass of the steel beams grillage that is mounted on the roof of existing masonry building and serves as a foundation for the new upper structure.

The natural frequencies of the three structural parts are evaluated as follows:

$$\omega_{LS} = \sqrt{\frac{k_{LS}}{m_{LS}}};\ \omega_{ISO} = \sqrt{\frac{k_{ISO}}{m_{US} + m_{ISO}}};\ \omega_{US} = \sqrt{\frac{k_{US}}{m_{US}}} \tag{1}$$

In order to evaluate the feasibility of retrofitting existing masonry structures by means of the intermediate isolation, and to define the optimal configuration of the isolated vertical addition, i.e. the one that minimizes the global seismic response, a wide parametric analysis is carried out (see section 4), by adopting the simplified 3DOF IIS model for the extended building, and, the sake of comparison, the SDOF LS model of the as-is building.

The properties of the isolation system and the upper structure are the design unknowns. The design parameters adopted for the analyses refer to mass, stiffness and period ratios, and the range of values of such parameters not only account for the constraints imposed by the P.U.A. (Figure 5), but it is also more inclusive, for extending this study to any cities located in seismic zones. For this aim, the parametric analysis carried out also consider different dynamic characteristics for the existing masonry buildings.

4. FEASIBILITY AND EFFECTIVENESS OF IIS: PARAMETRIC ANALYSIS

4.1 Definition of the design parameters

As already mentioned in the previous section 3.2, assuming for architectural and historical conservation issues not to modify the substructure in any way, the design variables are exclusively represented by the dynamic characteristics of the isolated upper structure.

For the purpose of generalizing the present application, the followings non-dimensional parameters are introduced in terms of mass, stiffness and period ratios, namely:

- mass ratio α, evaluated as the ratio of the total isolated mass, namely $M_{ISO} = m_{US} + m_{ISO}$, to the lower structure mass, m_{LS};
- stiffness ratio K, evaluated as the ratio of the upper structure stiffness, k_{US}, to the one of the lower structure, previously appointed as k_{LS};
- isolation ratio I, evaluated as the ratio of the nominal period of the isolation system, T_{ISO}, to the one of the fixed base upper structure, here named T_{US}.

In particular, the parameters are varied in the following ranges:

- $\alpha = \dfrac{M_{ISO}}{m_{LS}} \in \{0.1;\ 0.25;\ 0.5;\ 1;\ 2\}$;
- $K = \dfrac{k_{US}}{k_{LS}} \in \{0.1;\ 0.5;\ 1;\ 2\}$;
- $I = \dfrac{T_{ISO}}{T_{US}} \in [0.1,\ 10]$.

With regard to the parameter I, it should be recalled that it is also adopted for the base isolation system (BIS), for which a value greater or equal than 3 is recommended by several design codes (including Italian design codes [42]).
As discussed in the previous section 3.2, for the sample of masonry buildings of Pozzuoli subjected to volumetric re-composition, the values of mass ratio ranges between 0.1 and 0.4. However, in order to explore the behaviour of IIS structures, a wider range of values for α (between 0.1 and 2) is here adopted. In addition, it is worth underlining that the parameter α does not allow to evaluate the mass of the superstructure and the isolated system individually, but it is referring to the amount of the total isolated mass M_{ISO}. Considering the values derived from the relevant literature for simplified two degree-of freedom BIS models [43, 44], in the present paper the mass of the upper structure is assumed equal to 1.5 times the mass of the isolation system, i.e. $m_{US}/m_{ISO} = 1.5$ (value obtained by assuming the parameter γ equal to 0.6, being $\gamma = m_{US}/M_{ISO}$).
For what concerns the stiffness ratio, feasible values of K are certainly larger than 0.5. However, also in this case, the choice of a very flexible upper structure is adopted for thoroughly investigating the dynamic behaviour of IIS buildings.
Accounting for the morphology and height of the buildings to extend (from one to three story tall, see Figure 5), five different masonry lower structures are defined, by varying the fundamental period T_{LS}, namely:

- $T_{LS} \in \{0.18;\ 0.36;\ 0.55;\ 0.72;\ 0.81\}\ s$ (i.e. from stiff - one story to more flexible - three storey masonry building).

In addition, a further mass ratio parameter is considered in this paper, named the "total" mass ratio Rm, that is the total isolated mass $M_{ISO} = m_{US} + m_{ISO}$ over the total mass of the building $M_{tot} = M_{ISO} + m_{LS}$. Clearly, this parameter could also be provided as a function of the mass ratio α:

$$R_m = \frac{\alpha}{1+\alpha} \tag{2}$$

Therefore, for the values of mass ratio α equal to 0.1, 0.25, 0.5, 1, 2, the corresponding values of the total mass ratio R_m are respectively equal to: 0.1, 0.2, 0.3, 0.5, 0.7.

4.2 Non-proportional damping

With reference to the three parts of the 3DOF model for IIS, i.e. LS, ISO and US, the damping ratios are assumed respectively equal to:

$$\xi_{LS} = \frac{c_{LS}}{2m_{LS}\omega_{LS}} = 0.05; \quad \xi_{ISO} = \frac{c_{ISO}}{2M_{ISO}\omega_{ISO}} = 0.15; \quad \xi_{US} = \frac{c_{US}}{2m_{US}\omega_{US}} = 0.02 \tag{3}$$

In particular, the equivalent damping ratio of the isolation system corresponds to the adoption of high damping rubber bearings. It is worth noticing that assuming very different values of the damping ratios for the three DOFs of the system, the IIS model would be characterized by non-proportional damping. A non-proportional (or non-classical) damped system is characterized by complex-valued natural modes and does not satisfy the Caughey and O' Kelly identity: $\mathbf{CM}^{-1}\mathbf{K} = \mathbf{KM}^{-1}\mathbf{C}$ (with **M**, **K**, and **C** the mass, stiffness and damping matrices) [45], therefore the off-diagonal terms in the damping matrix **C** cannot be neglected. Furthermore, when **C** is an arbitrary symmetric positive definite matrix, the expansion in terms of the eigenvectors for the undamped system and real modal coordinates does not lead to uncoupled modal equations. Therefore, we should work with an expansion involving complex modal coordinates and complex state eigenvectors, working in the state space [46].

Hence, considering the non-classically damped IIS models, the eigenvalue problem is set as $\mathbf{A}\mathbf{u}_n = \lambda_n \mathbf{u}_n$ and is solved to obtain the *n-th* complex eigenvalue, λ_n, and eigenvector, $\mathbf{u}_n$, respectively corresponding to $f_n = 2\pi|\lambda_n|$ and $\eta_n = -\mathbf{Re}(\lambda_n)/|\lambda_n|$, where:

$\mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{I}; & -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}$ is the state space matrix (2N x 2N), and I (N x N) is the unit matrix;

$|\lambda_n| = \sqrt{\mathbf{Re}(\lambda_n)^2 + \mathbf{Im}(\lambda_n)^2}$ is the modulus of the *n-th* eigenvalue, $\mathbf{Re}(\lambda_n)$ and $\mathbf{Im}(\lambda_n)$ are the real and complex conjugate pair of λ_n.

However, for the sake of simplicity, the modal dynamic analyses, utilized for defining the dynamic behaviour of the IIS systems, are firstly carried out by assuming proportional damping. Then, the optimal configurations for the isolated vertical addition are selected by carrying out response spectrum analyses in which the modal damping values are assumed equal to the corresponding damping ratios of the excited portions (section 4.3). Finally, these simplified assumptions are removed in section 4.4 for the spectrum-compatible time history analyses carried out on the selected IIS optimal configurations.

4.3. Linear Dynamic Analyses

In order to evaluate the feasibility of retrofitting masonry buildings by means of intermediate isolation and to identify the optimal configurations for the isolated vertical addition, linear dynamic analyses are performed on both models, 3DOF IIS and SDOF LS, respectively representing the retrofitted building and the existing masonry structure.

The elastic acceleration response spectrum derived from the Italian seismic design code [42] for the site of Pozzuoli, characterised by 10% probability of exceedance in 50 year (475 years return period), has been used for the response spectrum analyses (RSA). The spectrum is depicted in Figure 9, for a_g = 0.162 g, F_0 = 2.347, T_C* = 0.333 s, S = 1.472, C_C = 1.509, ξ = 0.05.

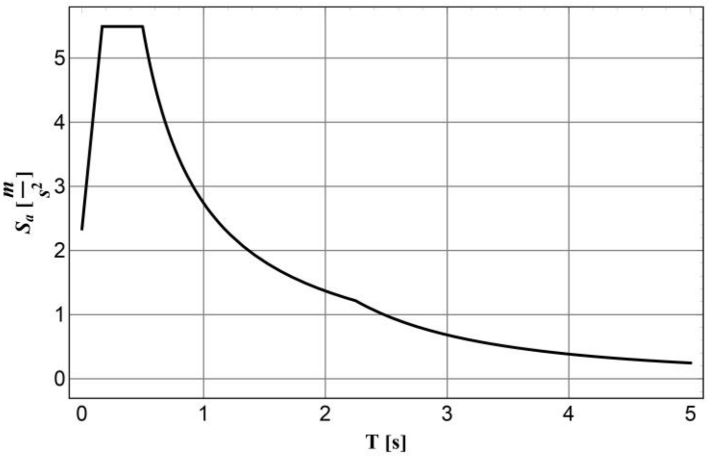

Fig. 9: (Lifesaving Limit State) LLS Elastic Spectrum Response of the site of Pozzuoli (Italy) in terms of acceleration, by assuming a structural damping of 5%

The complete quadratic combination, CQC, is adopted for evaluating the base shear force in the 3DOF IIS model, $V_{b,3DOF\ IIS}$, by assuming as modal damping ratios the structural damping ratios of the three parts, i.e.: isolation mode, $\xi = \xi_{ISO}$ = 0.15; mode of the lower structure, $\xi = \xi_{LS}$ = 0.05; mode of the upper structure, $\xi = \xi_{US}$ = 0.02. The base shear force in the SDOF LS model, $V_{b,SDOF\ LS}$, is evaluated by assuming a constant damping ratio equal to $\xi = \xi_{LS}$ = 0.05.

In order to evaluate the effectiveness of IIS system in retrofitting existing buildings, a further parameter, the base shear ratio v, being $v=V_{b,3DOF\ IIS}/V_{b,SDOF\ LS}$, is introduced. It should be noted that, when this parameter is less than one, a reduction of the seismic demand on the extended configuration IIS is observed, with respect to the as-is configuration.

In Figures 10a and 11, the base shear ratio v is depicted as a function of the isolation period T_{ISO}, respectively for the period of the lower structure T_{LS} equal to 0.36 s and to 0.18 s, 0.55 s, 0.72 s, 0.81 s. The results are provided as a cloud of points by varying the stiffness ratio *K* for a fixed value of the mass ratio *α*, and, for each mass ratio, a fitting curve is also plotted in the charts. In particular, the fitting curve is obtained by means of low-pass and mean filtering processes. It is worth observing that the lower structure identified by the period of 0.36 s and the mass ratio equal to 0.25 is representative of the dynamic characteristics of most of Pozzuoli's sample of buildings (i.e. two-storey, quite flexible structure). Therefore, in order to interpreter the results provided in the figures, for T_{LS} = 0.36 s and α = 0.25 (R_m = 0.5), in Figure 10b, c, d are respectively plotted the natural vibration modes, periods, and participating masses of three emblematic cases, namely:

(1) T_{LS} = 0.36 s, α = 0.25, K = 1, I = 0.9, with T_{ISO} = 0.125 s and R_m = 0.2 (circle marker in Figure 10a);
(2) T_{LS} = 0.36 s, α = 0.25, K = 0.5, I = 3, with T_{ISO} = 0.591 s and R_m = 0.2 (square marker in Figure 10a);
(3) T_{LS} = 0.36 s, α = 0.25, K = 0.5, I = 10, with T_{ISO} = 1.972 s and R_m = 0.2 (triangle marker in Figure 10a).

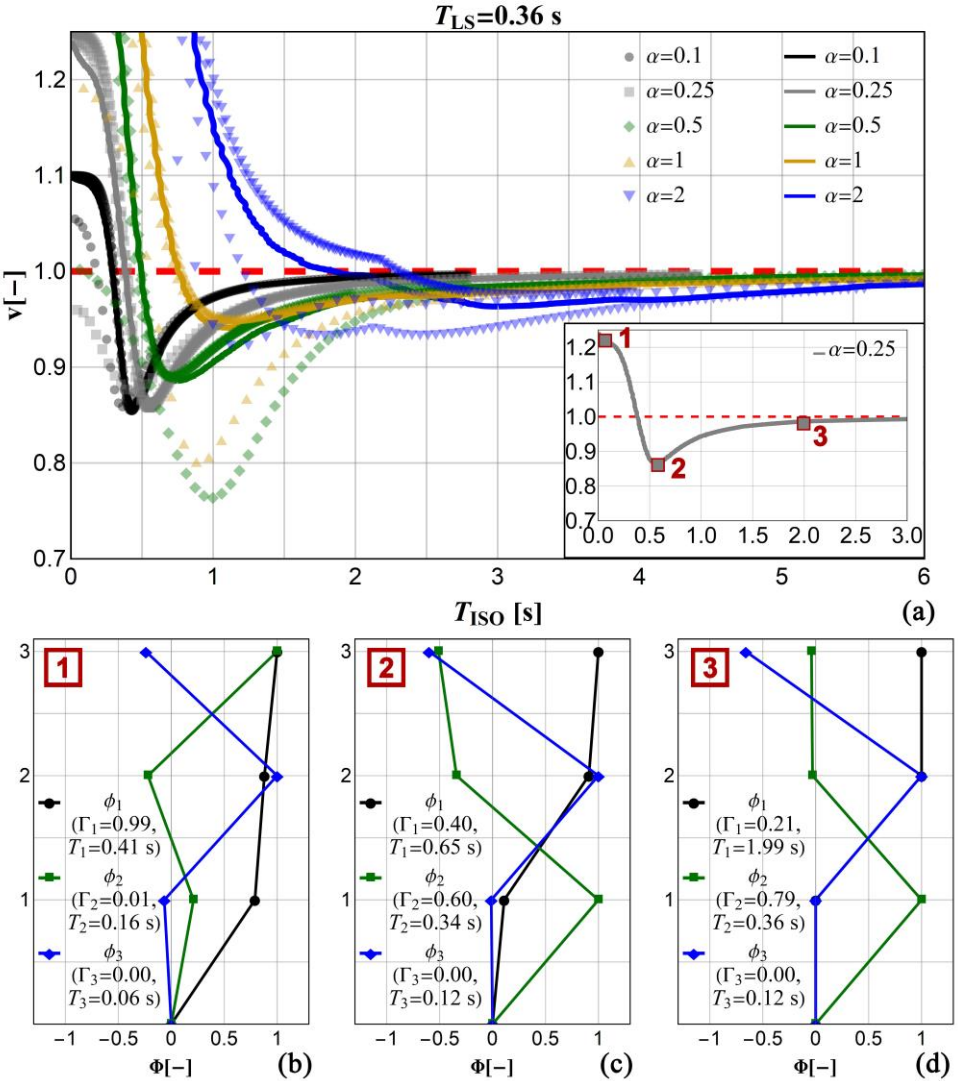

Fig. 10: (a) base shear ratio v as a function of T_{ISO} for T_{LS} = 0.36 s, vibrational mode shapes for: (b) case (1), (c) case (2), (d) case (3).

From the curves of Figure 10a, it can be observed that for very low values of T_{ISO} the 3DOF IIS structure tends to a conventional fixed-base behaviour and, consequently, the base shear ratio v is greater than one. However, for low-medium isolation periods (approximately from 0.3 s to 2 s) the base shear ratio v is less than one, while when T_{ISO} further increases up to 10 s, v tends to one.
The **case (1)** (Figure 10b) is representative of fixed-base structure in terms of vibration characteristics: the increase of mass accompanied by a decrease of stiffness leads to a longer first period (T_1 = 0.41 s larger than T_{LS} = 0.36 s).

Furthermore, while the spectral acceleration corresponding to the first period slightly decreases, the first participating mass ratio notably increases, thus implying a base shear ratio greater than one.
In the **case (3)** (Figure 10d) the first mode is the mode of the isolation layer with almost no deformations in the upper and lower structures. The first participating mass is almost equal to the

total isolated mass quantified by the parameter $R_m=M_{ISO}/M_{TOT}$. The second mode is the mode of the lower structure, with almost no deformation in the upper structure. The period and participating mass respectively correspond to the period T_{LS} and the mass of the lower structure m_{LS}/M_{TOT}. The third mode is the second mode of the isolated upper structure, with almost no deformation in the lower structure and a null participating mass. Hence, when T_{ISO} tends to infinity, or in practical terms, when it is greater than, say, 4 s, the flexible interface completely disconnects the isolated upper structure and lower structure. By emphasizing the isolation effect, this kind of behaviour is representative of IIS with *perfect isolation* (i.e. I is equal to, or larger than, 5) [24]. Accordingly, the significant modes of vibration are the pure modes of the isolation system and the lower structure. In this case, the base shear ratio is almost equal to one, in fact, the dynamics of the LS cannot be nullified, also when the flexibility of the ISO becomes very large [47].

The mode shapes of the **case (2)** (Figure 10c) shows both similarities and differences with respect to the case (3): the first mode is still the isolation mode, and the higher modes are still the first mode of the lower and the second mode of the isolated upper structures. However, considering the first mode, while in the case (3) it only involves the isolation layer, in the case (2) displacements also arise in the lower structure, and, more importantly, the participating mass is greater than the parameter R_m. Therefore, the second mode, involving the lower structure contribution, shows a participating mass smaller than the mass of the lower structure, quantified by $1-R_m$. Hence, for intermediate values of T_{ISO} (0.5 – 2 s), the isolated upper structure and the lower structure are not completely disconnected. Rather, they interact with a mechanism giving rise to the mass damping effect; therefore, the case (3) is representative of IIS with *non-perfect isolation* (i.e. I is smaller than 4) [24]. The significant modes are no more the pure modes of the isolation system and the lower structure; as already noticed in [24], "this is the main counterintuitive reason behind the advantages of IIS for retrofit. In fact, the involvement of a mass fraction of the LS in the first mode, characterised by long period and low spectral ordinates (low energy content), reduces the mass participating to the lower structure mode, characterised by much shorter period and corresponding to larger spectral ordinates (high energy content). Consequently, the base shear in the IIS configuration is reduced with respect to the existing structure configuration ($v < 1$)". In this light, the non-perfect isolation represents an *antifragile strategy* for retrofitting the masonry building by means of the new isolated vertical addition.

It is worth also observing that, the smallest values of the shear ratio v generally refer to stiffness ratios smaller than 1, and thus to upper structures excessively flexible. However, the stiffness of the steel extension should be comparable to, or larger than, the masonry counterpart ($K \geq 1$).

Moreover, it is worth underlining that among non-perfect isolation cases, not all configurations could be considered as optimal design solutions. In fact, in some cases, although the seismic demand of the LS is reduced ($v < 1$), a resonance response can occur on the superstructure, thus nullifying the beneficial filtering effect of the isolation system.

This additional aspect of the dynamic behavior of IIS, i.e. the isolation with higher modes coupling (MC) has been already investigated in [21]. In particular, the higher modes coupling effect, addressed in the recent literature [40, 41, 48, 24, 49], is related to the dynamic interaction between upper and lower structural portions, and does not depend on the isolation ratio I. It arises when the second and third modes are characterized by comparable periods, non-null participating masses, and modal deformations involving both the lower and upper structures. This effect produces a detrimental amplification of the dynamic response of the upper structure. However, the MC can be predicted and avoided by a careful design of the upper structure starting from the dynamic characteristics of the existing building, which are known. These configurations are discarded as design solutions in this paper.

By considering the other lower structures, characterised by different values of fundamental periods T_{LS} (Figure 11), a similar trend is observed, with the three zones of fixed-base behaviour, non-perfect isolation and perfect isolation clearly recognisable. Some differences emerge looking at Figure 11a, which refers to the results obtained for lower structure characterised by T_{LS}=0.18 s.

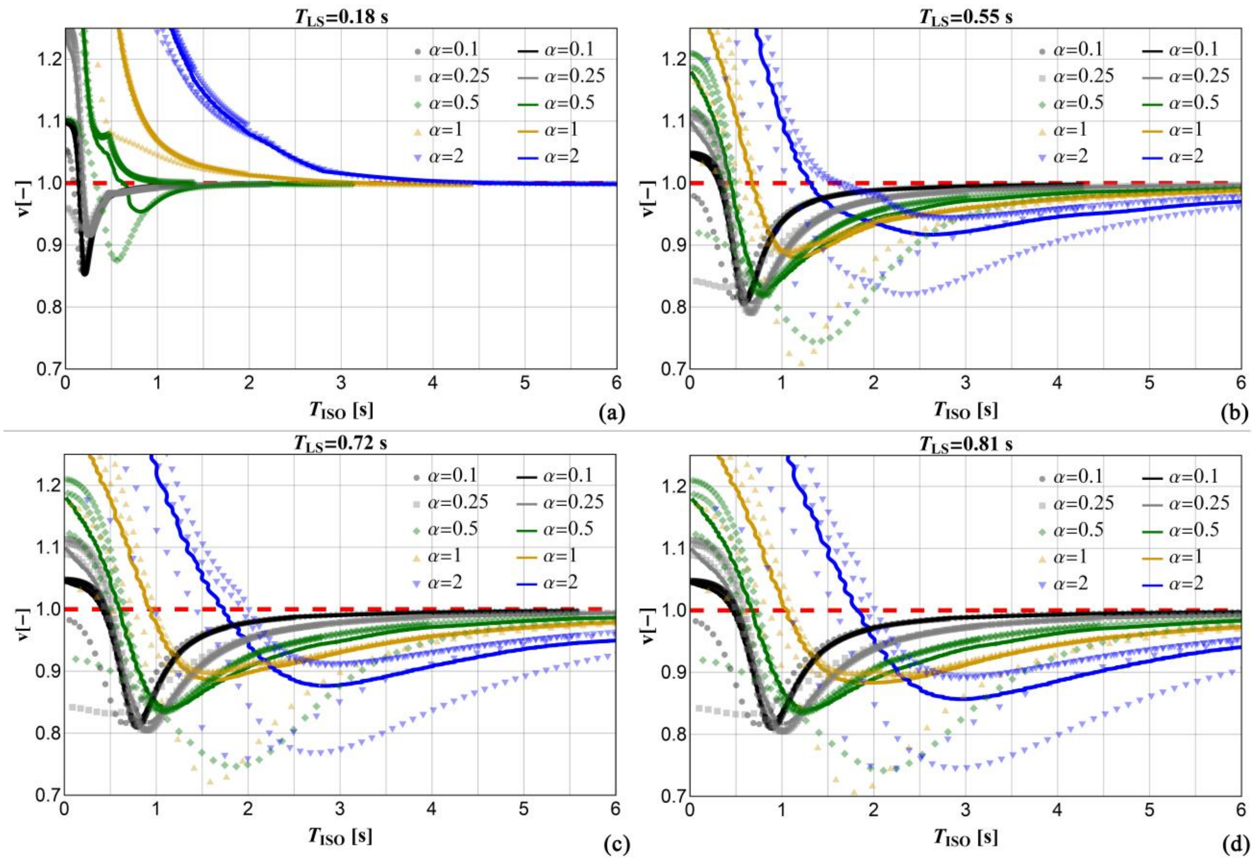

Fig. 11: Base shear ratio v as a function of T_{ISO} for: (a) T_{LS} = 0.18 s, (b) T_{LS} = 0.55 s, (c) T_{LS} = 0.72 s, (d) T_{LS} = 0.81 s.

With mass ratio equal to 1 and 2, the dynamic behaviour of the retrofitted configuration directly goes from fixed base to perfect isolation, without showing the range of non-perfect isolation or mass damping. In fact, the lower structure is so rigid that the mass damping effect does not arise. For the mass ratio equal to 0.5 for T_{ISO} between 0.5 – 0.7 s, the fitted curve and the punctual results show values both larger and smaller than, one. As reported in section 3.2 and Figure 5, for the Pozzuoli's sample of buildings the mass ratio is always smaller than 0.5. Therefore, it always exists a range of values of T_{ISO} for which the mass damping effect, though limited, is activated. By considering quite flexible lower structures (T_{LS} larger than 0.36 s) the effect of the mass ratio clearly emerges from Figure 11. In fact, by increasing the value of α, an ever-wider range of values of T_{ISO} driving to base shear ratio smaller than one can be observed.

In the charts of Figure 12 the effect of T_{LS} (from 0.18 s to 0.81 s) on the fitted values of the base shear ratio ν is provided for the mass ratio α equal to 0.25. Here it can be observed that, by increasing the period of the lower structure, the minimum value of ν shifts on the right, and the

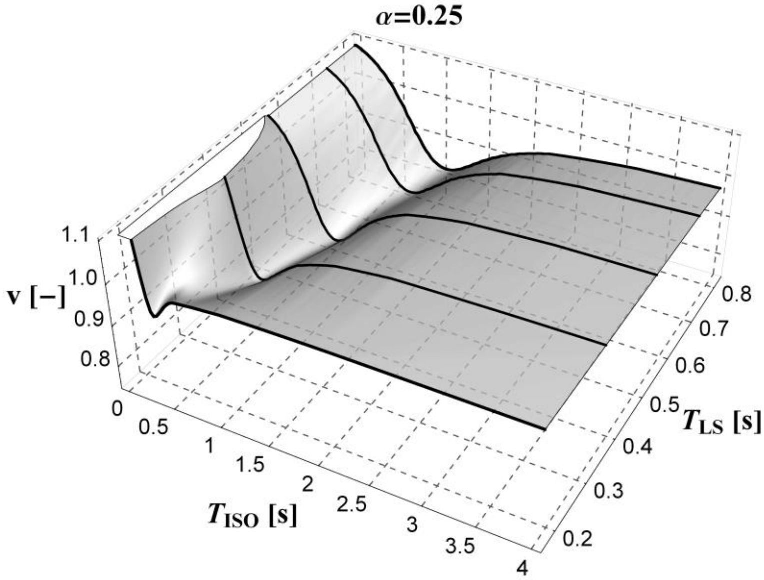

Fig. 12: Base shear ratio ν as a function of the T_{ISO} for α=0.25.

zone characterized by non-perfect isolation is enlarged. Hence, by increasing the flexibility of the lower structure, it exists an ever-wider range of values of T_{ISO} such that the base shear ratio is smaller than one. Consequently, by adopting a quite flexible lower structure and a heavy vertical addition, the system becomes more robust, thus even more design configurations can be adopted for effectively retrofitting the existing masonry building.

It should be underlined that the parametric analysis carried out on simplified 3DOF lumped mass models, represents a preliminary exploration of the feasibility of this retrofit technique. As it can be noted from Figure 4, the masonry buildings of the Pozzuoli district here investigated are characterized by different dimensions in plan and elevation, thus the structural characteristics and dynamic properties of the lower structures vary widely. The parametric analysis has accounted for this variability, by varying the fundamental period of the lower structure. However, the evaluation of the expected failure mechanism, without and with the upward extension, is a further and fundamental aspect that should be evaluated, when a specific case-study building is selected. In fact, this issue is strictly related to the specific masonry building assumed, the dimensions of the masonry piers, the floor structural system, the number of existing and new floors, the connections among orthogonal masonry piers, particularly at the corner, etc. Starting from the general overview of Pozzuoli, the authors have further investigated the above aspects with reference to a specific masonry building by means of more refined 3D models [21].

4.4. Time-History Analyses

For the time-history analyses, seven natural acceleration inputs are used, by selecting the ground motion records whose average response spectrum matches the target response spectrum (already

provided in Figure 9), with a tolerance of 30%. More in detail, the selection process is carried out by the software REXEL v 3.5 [50]. The spectrum-compatible records are chosen by imposing that the peak ground acceleration (PGA) is between 0.1 g and 0.3 g in the periods of interest, from 0.15 s to 2 s, and by considering any site class in the SIMBAD database (Selected Input Motions for displacement-Based Assessment and Design) [51].
Therefore, Table 1 reports the waveform, the earthquake ID and station identification codes of the defined ground motion inputs with their PGA values. The profiles of the acceleration response spectra and the average spectrum, are depicted in Figure 13.

Table 1. Seven spectrum-compatible input signals

Waveform ID	Earthquake ID	Station ID	Earthquake name	Date	PGA [g]
317	133	MOG0	Emilia_Pianura_Padana	29/05/2012	0.17
335	137	TPLC	Darfield	03/09/2010	0.19
341	142	RHSC	Christchurch	21/02/2011	0.25
389	149	RHSC	Christchurch	12/06/2011	0.19
411	34	TKS	Hyogo - Ken Nanbu	16/01/1995	0.18
449	93	WSM	Superstition Hills	24/11/1987	0.21
459	99	ST_24389	Northridge	17/01/1994	0.22

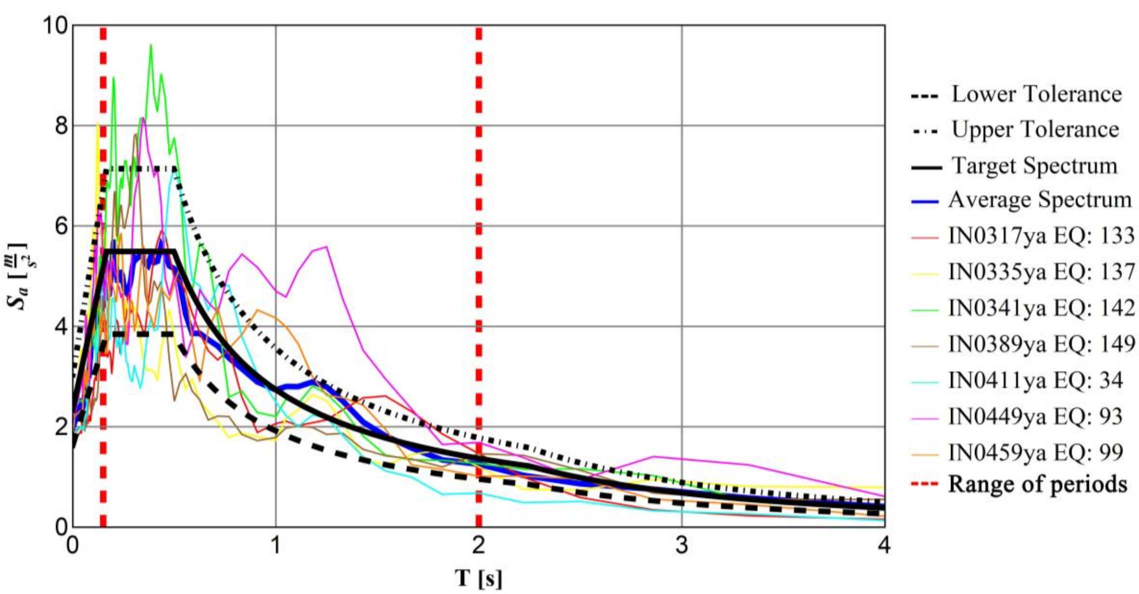

Fig. 13: Set of seven spectrum-compatible acceleration response spectra.

According to the results of the section 4.3, two optimal configurations are here chosen, i.e. the cases (2) and (3) (Figure 10a, c and d) here appointed as IIS 2 and IIS 3, respectively accounting for the *non-perfect* and *perfect isolation*. In addition, the SDOF LS model identified by the period T_{LS} equal to 0.36 s is considered for comparison purpose.
As already mentioned in section 4.2, since the 3DOF IIS model is characterized by non-proportional damping, here two hypotheses for the damping ratios are adopted, according to which

the damping ratio ξ is interpolated as a function of the mode of vibration. It is worth recalling that, for the IIS 2 and 3 models the three modes are the modes of the isolation system (pure or not), the lower structure (pure or not) and the upper structure, respectively. The first hypothesis (hp 1) is the same one adopted for the RSA, i.e. the modal damping ratios are assumed equal to the structural damping ratios of the three parts: $\xi_1 = \xi_{ISO} = 0.15$, $\xi_2 = \xi_{LS} = 0.05$, $\xi_3 = \xi_{US} = 0.02$. The second hypothesis (hp 2), instead, considers the complex modal damping ratios η of the three vibration modes; i.e.: for the case IIS 2 $\eta_1 = 0.11$, $\eta_2 = 0.08$, $\eta_3 = 0.08$; for the case IIS 3 $\eta_1 = 0.15$, $\eta_2 = 0.06$, $\eta_3 = 0.05$.

In the following, some analysis results are discussed, in terms of: peak relative story displacement and peak absolute story acceleration (Figure 14), and time history of the base shear (Figure 15). In particular, the charts in Figure 14a and b depict the envelopes of the average values obtained considering the maximum peak story drifts and peak absolute accelerations, obtained for the 3DOF IIS and 3D LS models under the seven seismic inputs, respectively by adopting the first and second hypothesis on the damping. The relevant average shear ratios v are also provided in the figure. The results plotted in the figures can be explained in the light of the mass damping and isolation effects. The mass damping effect consists in a reduction of the LS peak displacement with respect to the LS counterpart, clearly shown by the IIS 2 model. The isolation effect consists in reduction of the peak acceleration of the isolated US, large isolation displacement, and negligible US displacement; both the IIS models exhibit these behavioural aspects, but they are much more evident in the IIS 3 model than in the IIS 2 one.

In addition, it can be stated that the adoption of the values of the structural damping ratios for the modal damping ratios is an approximation on the safe side, particularly for the LS response and in

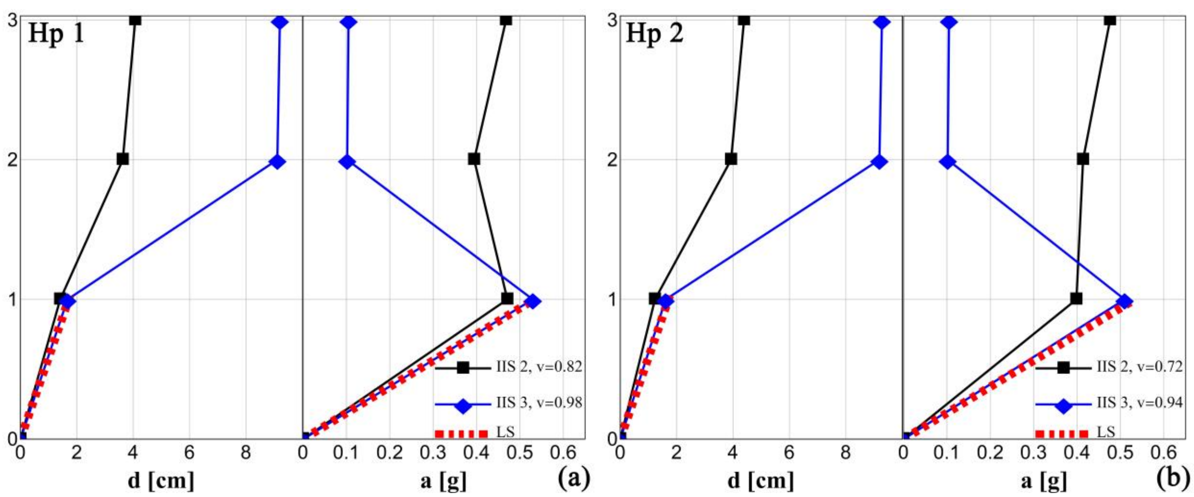

Fig. 14: Peak story displacement and absolute accelerations: (a) hp 1 of structural damping, (b) hp 2 of complex damping.

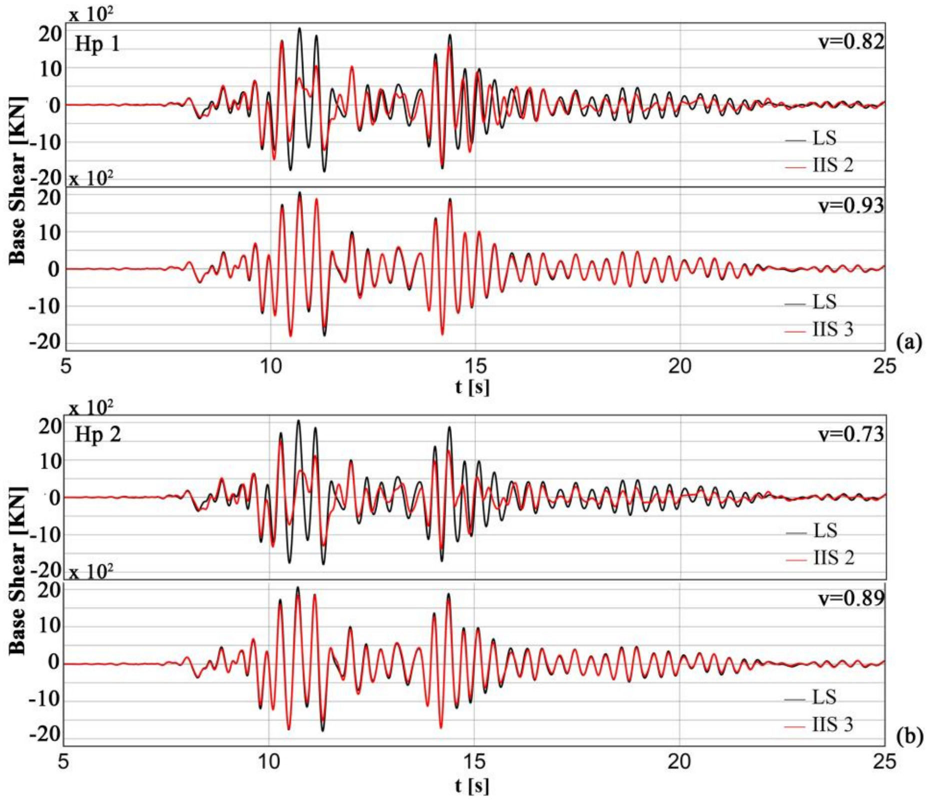

Fig. 15: Time histories of base shear for the record Emilia_Pianura_Padana: (a) hp 1: structural damping, (b) hp 2: complex damping.

the case of non-perfect isolation. In fact, as discussed in the section 4.3, the base shear of the retrofitted configuration is mainly due to the contribution of the lower structure. For the IIS 2 model, the complex damping ratio of the LS mode is larger than the structural damping ratio of the LS (i.e. $\eta_2 = 0.08$ and $\xi_2 = 0.05$). Therefore, the LS response is further reduced, as compared to predictions made on the basis of the RSA utilizing the structural damping ratios.

The average value of the peak base shear ratio, ν, evaluated considering the seven input waves, is always less than 1 in the two IIS configurations; in particular, it is equal to: 0.82 and 0.72 for the IIS 2, respectively for the hp 1 and hp 2; 0.98 and 0.94 for the IIS 3, respectively for the hp 1 and hp 2. It is worth observing that the results for the hypothesis 1 agree with the ones obtained from the RSA (Figure 10a), that provides values of the base shear ratio respectively equal to 0.85 and 0.98, for the cases IIS 2 and 3.

As an example, in Figure 15a and b are shown the time histories of the base shear obtained for the retrofitted and reference configurations under the record of Emilia_Pianura_Padana (317), respectively for the first and second hypothesis on damping. In the graphs the minimum values of the base shear ratio ν is also provided. It can be observed that the base shear of the IIS is always less than the LS counterpart, with minimum values equal to: 0.82 and 0.73 for the IIS 2, respectively for the hp 1 and hp 2; 0.93 and 0.89 for the IIS 3, respectively for the hp 1 and hp 2. These results further confirm that the hypothesis 1 is on the safe side, leading to largest values of the base shear ratio ν.

5. CONCLUSIONS

This paper deals with an innovative scheme for vertical extensions of buildings in seismic zones, able to address contemporarily the problems of urban densification and seismic retrofit of existing buildings. In particular, the suggested strategy can be applied to all European cities where two needs meet at the same time: the necessity to extend vertically the existing built heritage and the requirement of seismic retrofitting the existing structures.

In this perspective, the proposed applications can be ascribed to the category of “antifragile” interventions: a system that not only is unharmed by adverse events but is actually strengthened by them.

This idea is proposed for Pozzuoli (town of the province of Naples, south Italy) characterized by a long history of cyclic disasters and resurgences, for which a wide plan to reconstruct rooftop volumes, previously demolished, has been recently issued. This plan aims to confer to the historic centre of Pozzuoli its ancient form, with a strategy that, however, makes impossible reading the new interventions. In the present paper, according to the main guidelines on the cultural heritage, an alternative strategy is proposed, with vertical extensions that are intentionally designed to be legible from both architectural and structural point of view, namely with contemporary architectural forms clearly denouncing the intervention time, and structural systems and techniques that radically, but advantageously, modify the behaviour of the extended structural complexes with respect the as-is configurations.

For these reasons, the potential improvement of the building extensions is analysed from different points of view, namely architectural language, also accounting for the urban context, and seismic response.

Indeed, considering the architectural aspects, the new volumes are conceived and designed by taking into account some zones of the city that are strongly representative of the territory’s identity, namely: around the square, along the roads, waterfront. In addition, for emphasizing the peculiarities of the urban context, different compositive strategies are defined for different situations, namely: the urban corner, the completion as urban texture repair, the reconfiguration of the façades.

From a structural point of view, the concept of intermediate isolation system is adopted, with the extension equipped with seismic isolation system at its base. In order to evaluate its feasibility and effectiveness for retrofitting existing masonry structures, a wide parametric analysis is carried out on simplified lumped mass models. The properties of the isolation system and the upper structure are assumed as the design unknowns.

From the results of the analyses, the following observations can be derived.

- the intermediate isolation system has the potentiality of working by combining the isolation and mass damping strategies; in particular, the isolation layer behaves as an isolation system for the vertical addition, while the isolated vertical addition behaves as a mass damper for the existing structure;
- two distinctive kinds of behaviour can be identified, respectively emphasizing the mass damping effect (non-perfect isolation) and the isolation effect (perfect isolation);
- among the non-perfect isolation behaviour, configurations characterized by higher modes coupling effects are not considered as design solutions, due to the resonance response of the upper structure;

- remarkable reductions of the base shear force are obtained by designing isolation systems with low - medium isolation ratios, thus with isolation periods not as large as in the case of base isolated structures;
- by increasing the number of floors of the lower structure and selecting heavy vertical addition, a significant increase can be observed for the range of feasible design configurations of the isolated extension that ensures a reduction of the building seismic response.

While a pilot study for Pozzuoli is here considered, this multidisciplinary strategy, based on the vertical extension of existing masonry buildings, can be applied to the building stock of any cities located in seismic zones. However, some fundamental prerequisites should be satisfied for applying this technique: a rigid floor diaphragm should be present at each floor, orthogonal walls should be effectively tied to each other, particularly at the building corners, thus guaranteeing an effective global box behaviour of the structure.

From the methodologic point of view, it is worth pointing out that the suggested approach is different from traditional ones: the intermediate isolation reduces the seismic demand on the lower structure and allows for a vertical extension, while conventional standard retrofit interventions only act on the seismic capacity of the existing structure. Therefore, a fair comparison in terms of cost-effectiveness between this and traditional retrofit strategies should be carried out by means of a value engineering approach. Indeed, the increase of the real estate value resulting from the new floors cannot be neglected in the comparison, especially in urban areas that are already heavily densified and characterized by high land rent. Moreover, long-term benefits resulting from the reduction of the seismic demand should be considered in a life cycle perspective, properly accounting for the reduction of massive damages in the aftermath of a seismic event, with consequent lower repair costs, functionality losses, and downtime. Therefore, the proposed strategy has the potential to lead to better performance and long-term cost effectiveness, ultimately connoting more sustainable design solutions. For this purpose, some specific case studies are currently under investigation.

The authors are confident that the present study can trigger a wide and fruitful discussion on sustainable and resilient, or even antifragile, approaches for urban densification in earthquake-prone cities.

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