ingegneria sismica
International Journal of Earthquake Engineering
Under revision

INFLUENCE OF INFILLED FRAMES ON SEISMIC VULNERABILITY ASSESSMENT OF RECURRENT BUILDING TYPOLOGIES

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SUMMARY: *This work is aimed at the assessment of the influence of the infill presence on the seismic vulnerability evaluation. In particular, some results obtained from recurrent building typologies models, defined from the observation of the existing building stock, will be presented, and differences regarding structural behavior, seismic demand and analyses output on the basis of various hypothesis on the infills presence.*

KEYWORDS: *Seismic vulnerability, RC buildings, infills, existent buildings*

1 Introduction

In the seismic vulnerability assessment of the existent buildings, the influence of the infill panels over the global structural behavior has a high relevance.
In the technical literature, it's widely accepted that the presence of infilled frames modify relevantly the structural behavior, eventually causing the collapse of the buildings for their distribution in plane and in elevation. In the structural design is usual analyzing the structure as bare, while the presence of infills cause a global stiffening that usually brings to higher seismic demands, as well as the cited change in the structural behavior, resulting in unforeseen consequences.
This paper highlights that in the field of seismic vulnerability analysis at a territorial level, the presence of infills can lead to different results. This result will be shown by the variation of a Seismic Vulnerability Index defined on the base of a multi-parameter analysis calibrated for the purpose.

1.1 Structural model selection and description of their characteristics

For the present purposes 4 recurrent building typologies were selected and identified inside the urban territory of the municipality of Bovino (FG), assumed as reference territory for assessment of the Seismic Vulnerability Index. Specifically, the building typologies were identified through the informations collected via quick survey forms ([Uva *et al.*, 2016]), that could be also useful to evaluate simplified indices related to both structural and non-structural seismic performance [Perrone *et al.*, 2015). The performance of non-structural elements is of paramount importance in the loss estimation framework, even if few information about their fragility and consequence functions are available.

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Selected typologies can be described as follows:

- Typology “A”: representing all of the structural shapes elongated plan, with a predominant dimension, sub-divided into two additional sub-typologies on the basis of the number of columns alignments along the shorter side (3 for the “A1” type, 4 for the “A2”)
- Typology “B”: including all of the compact-shaped buildings, with substantial equal dimensions along the two main directions, same number of columns alignments along those directions, whose number defines the sub-division into two additional sub-types (3 alignments for the “B1” typology, 4 for the “B2”).

Figure 1 – *The 4 selected building typologies. From left: A1, A2, B1, B2*

Those typologies represent a wide percentage of the whole existent building stock of the reference territory, and this situation is presented in particular in Table 1, where the number of recorded buildings of every typology with respect of a total of 169 residential RC buildings recorded on the territory of Bovino.

Table 1 – *Presence of every typology in the municipality of Bovino (FG)*

Building typology	Number of recorded buildings
A1	73
A2	32
B1	14
B2	9

The percentual importance of those typologies with respect of the total is clearly seen through Figure 2, where a color-based map presents the position of the buildings categorized in a given typology.

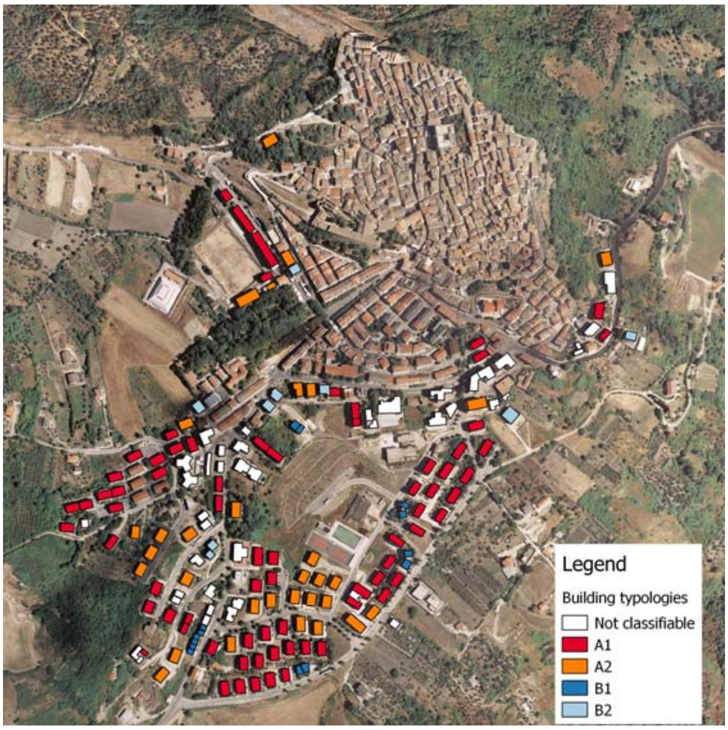

Figure 2 – *Incidence of the 4 considered building typologies in the selected territory*

For the analyses at which this work is aimed, for the identified typologies two kinds of parameters were defined:

- Deterministic parameters, including geometric characteristics of the whole building, *i.e.* number of floors, structural elements lengths and infill presence;
- Probabilistic parameters, not surely definable with a simple survey, including infill thickness, structural materials characteristics.

2 State of art

The importance of the seismic vulnerability assessment, in particular for seismic prone regions, is demonstrated by recent methodologies aimed to evaluate the expected annual losses [FEMA P-58, D.M. 58/2017]. Recently, in Italy has been introduced a seismic risk classification guideline, it is aimed to incorporate some of the more recent advancements in the field of the seismic risk assessment into a simple procedure integrated with existing codes in Italy. The guideline provides a classification system based on the structural safety and the expected annual losses. The structural safety index is defined as the ratio between the peak ground acceleration required to achieve the life safety limit state and the peak ground acceleration defined by the code for the site in which the building is located. A simplified procedure is proposed to evaluate the expected annual losses. Two seismic risk classes are separately evaluated for the structural safety index and the expected annual losses; the lowest class is assumed to be representative of the seismic risk for the analyzed building.
In the technical literature, the problem of defining an appropriate procedure for the seismic vulnerability assessment is highly treated. However, in this context it's considered important to cite only the mechanical approaches, by which the non-linear behavior of the considered

structure is studied in terms of maximum displacement, strength or stiffness through capacity curves conveniently defined.

The principle of Performance-Based Earthquake Engineering is to define the response of a MDoF structure via the capacity curve of an equivalent single degree of freedom structure, and then (after the definition of appropriate limit states) to compare it with the seismic demand defined in terms of acceleration-displacement response spectra, by appropriate procedures, as for example the N2-method [Fajfar, 2000].

Starting from the model proposed in [Calvi, 1999], one of the most relevant methodologies is "D-BELA" [Crowley *et al.*, 2004], in which buildings are divided on the basis of the type of resisting system and collapse mechanism. For each building class, thanks to Monte Carlo analyses, a displacement that represents the capacity of the structure for the specific limit state is defined, by an analogy with a single degree of freedom system, which is subsequently compared with the seismic demand, defined by a displacement response spectrum. In this methodology, great importance is given to the estimation of the natural period of the equivalent SDoF system, which is given empirically as a function of the building height.

In SP-BELA methodology [Borzi *et al.*, 2008], instead, the structural capacity is defined through simplified pushover analyses, in which an elastic-perfectly plastic behavior is assumed, while the seismic demand is computed by varying the spectral shape with fixed PGA. Starting from the results of pushover analyses computed for a sample of homogeneous buildings, together with the probability that a specified spectral shape occurs in the considered period, it is possible to develop a probabilistic approach to the problem, as well as to define fragility curves, by changing the PGA.

Whatever is the selected approach for the seismic vulnerability assessment, operationally there is the problem of the implementation of the infills influence in the structural model. In the technical literature, this problem is treated by three different approaches:

- Micro-modelling approach, by which the infill constitutive elements are modeled in detail, bringing to a modelling that, also if can appear as more accurate, has the problem of the definition of the detailed mechanical characteristics of those elements, as well as the high computational time
- Meso-modelling approach, where the infill panel is treated as a continuum with appropriate characteristics, in a way that it's possible to describe adequately the global behavior
- Macro-modelling approach, where the infill influence is considered by one or more equivalent struts.

This last approach, thanks to its simplicity and physic significance, is the most used and cited in the technical literature, starting from the work of [Polyakov, 1956], that was the first to propose the possibility of describing the infills influence via an equivalent strut, on the basis of observations on the phenomenon physics, consisting in the detachment of the infill from the surrounding frame in the corners not subjected to compression, in this way reacting only along the compressed diagonal.

Starting from this work, in the following years the cited problem was deepened by other authors, that proposed various ways aimed at the infill modeling.

The approaches proposed by the authors that gave a contribution to the problem can be categorized on the basis of the number of the struts used for the modeling of the panel and its position. In fact, while the use of a concentric single strut is the most cited in the technical literature, a lot of authors proposed the use of more than one strut. To this category of proposals belong the ones by [Syrmakezis and Vratsanou, 1986], [Zarnic and Tomazevic, 1988], [Chrysostomou, 1991], [Crisafulli, 1997], [Al-Chaar, 2002].

One of the most important parameters that widely affect the infill panel modeling is surely its width, able to influence both elastic and post-elastic behavior. For the purpose of its definition, a lot of authors proposed that many different formulations, amongst which it's worth mentioning the work of [Stafford-Smith, 1966], on which a lot of successive formulations are based, the proposal by [Mainstone, 1971], later used also in [FEMA 273], the work of [Decanini and Fantin, 1987], that proposed different formulations for cracked and uncracked infill, and the suggestion of [Paulay and Priestley, 1992], which is an attempt of giving a practical and simple approach to the problem.

Cited works assume a solid infill panel, without any opening. The evaluation of the opening influence is usually defined through a multiplicative reduction factor of the infill width, whose expression is function of the opening to infill areas ratio, using expression that go from the linear shape as in [Papia and Cavaleri, 2001], to the polynomial of [Al-Chaar, 2002], the power of [Asteris *et al.*, 2011], to the exponential shape of [Decanini *et al.*,2012].

In the practical implementation of the equivalent strut implementation another problem is represented by the assessment of its strength. In this context it's worth mentioning the work by [Bertoldi *et al.*, 1993], that consider a wide set of possible collapse mechanisms of the infill panel, and what is suggested in [FEMA 306], being a surely authoritative source on the subject, as well as the works by [Stafford-Smith and Carter, 1969], [Zarnic and Gostic, 1997], [Dolsek and Fajfar, 2008] and [Cavaleri *et al.*, 2013].

Once defined those quantities, the problem of defining a post-linear behavior model of the infill arises, considering that it widely influences final results of the analyses. In this context the proposals of [Bertoldi *et al.*, 1993], [Panagiotakos and Fardis, 1994], [Dolšek and Fajfar, 2008], [Cavaleri and Di Trapani, 2014] and [FEMA 273] must be cited.

Strictly speaking, it must be cited that some of important aspects were neglected in the following analyses:

- The local influence of the infill panel, in terms of variation of stresses in the adjacent structural elements, factor neglected in this work because it's explicitly aimed at the evaluation of the only global structural behavior
- The behavior of the equivalent strut under cyclic loads, neglected because the analyses that will follow will be limited to the static type (pushover analyses).

In this context, a greater attention would be needed in modeling the pilotis configuration, specifically concerning joints between infilled and bare frames, where there is the higher probability of the triggering of a soft-story mechanism.

3 Estimation of the elastic period for building classes

The evaluation of the natural frequencies is of paramount importance in earthquake design and seismic assessment. The fundamental elastic period of a structure allows an improved understanding of demand under a given seismic input [Crowley and Pinho 2004]. The mass and stiffness are the main parameters involved in the evaluation of the elastic period; these parameters are affected by many factors such as structural regularities, design procedures, infill panels, material mechanical properties. The difficulties in the correct evaluation of the natural frequencies is often demonstrated by the discrepancies between the periods predicted by the numerical modelling and the periods recorded using ambient vibration measurements [O'Reilly *et al.*, 2017]; the results of some investigations demonstrated that one of the most

important parameter in the evaluation of the period is the assumption of cracked or un-cracked section both for structural elements and masonry infills [Ricci *et al.*, 2011]. Advanced numerical models were developed by researchers in order to match the frequencies evaluated by experimental investigations [O'Reilly *et al.*, 2017]. Such sophisticated estimation of the natural periods as well as of the seismic fragility curves definition can lead to accurate loss estimation of reinforced concrete structures. However, this approach is not feasible, and could be ineffective, when carrying out loss estimation studies referred to urban areas of seismic prone regions. In this case simplified methodologies are required for the estimation of the periods and of the fragility curves able to furnish the loss estimation of a whole class of buildings. Focusing on the simplified evaluation of the elastic periods some methodologies are available in the literature [Crowley and Pinho 2004, Perrone *et al.* 2016, Ricci *et al.* 2012]. These methodologies are mainly focused on the estimation of fundamental period including the influence of the masonry infills. In this section the influence of the masonry infills on the fundamental period is investigated for the building classes identified in this work. In this study, the height of the buildings and the infill configurations (bare and full infilled building configuration) were varied, while the mean values for the other investigated parameters were assumed. According to [Amanat and Hoque, 2006] for buildings characterized by more than four bays, the influence of the number of bays as well as of their length is not the most affecting parameter in the evaluation of the period. Depending on the typology of bricks and the quality of the mortar, the mechanical properties of the masonry infills vary in a wide range [Hendry 1998, Paulay and Priestley 1994]; for this reason, two values of the Young modulus of the masonry infills (1500 and 3500 MPa) were considered in the analyses. The dimensions of the columns as well as the Young modulus of the reinforced concrete were varied in the range proposed in Section 1.2 for the four identified building typologies; only the results related to the upper and lower thresholds of these mechanical and geometrical properties are discussed in this Section.
In Figure 3 the elastic periods are plotted for the bare frame configurations. The elastic periods vary in a wide range, the variability is mainly attributed to the inertia of the columns. The highest variability was observed for the 3-storey frames, in which the fundamental period varies in the range between 0.37 and 0.67 sec. The results are compared with the simplified formulation proposed by [Perrone *et al.*, 2016] for RC bare frames:

$$\mathrm{T} = 0.056\mathrm{H} \tag{1}$$

where H is the height of the building expressed in meters. This relationship was developed for buildings not designed according to seismic previsions, for this reason its capability to predict the elastic period is higher for more deformable buildings.

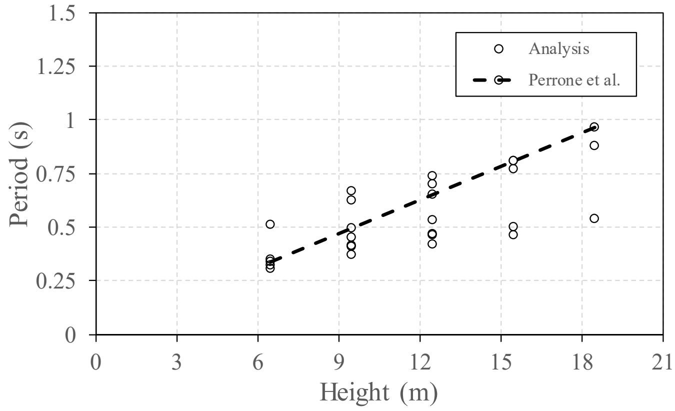

Figure 3 - *Elastic period bare frame configurations*

The modal participating mass was calculated in order to establish if the fundamental period of the buildings is representative of their dynamic behavior. Due to the regularity of the buildings, both in plan and elevation, for all analyzed cases the participating mass associated to the first mode is higher than 80% of the overall seismic mass.
The evaluation of the fundamental periods for the bare frame buildings is an interesting starting point to understand the variability of the dynamic properties of the analyzed buildings classes. Despite this, also at urban scale, it is of paramount importance evaluating the influence of the masonry infills. Figures 4 and 5 report the elastic period of the infilled buildings considering a Young modulus of the masonry infills (E_w) equal to 1500 and 3500 MPa, respectively. Also in this case the results are compared with the elastic periods predicted by the formulation proposed by [Perrone *et al.*, 2016] for infilled frames:

$$T = \lambda \cdot E_w{}^{-0.267} \cdot \left(1 + \frac{\mu}{0.2}\right)^{0.286} \cdot H \tag{2}$$

where:

- E_w is the Young Modulus in GPa,
- λ takes into account the beam deformability, it is equal to 0.0366 for rigid beams and 0.0411 for deformable beams
- μ is a coefficient related to the presence of openings, it is the ratio between the area of opening and the area of the full infilled panels. If the frames are full infilled without opening it is equal to 0, this coefficient has been validated up to percentage of opening equal to 60%.
- H is the height of the building, in meters.

This formulation was developed in order to accounting for the mechanical and geometrical properties of the masonry infills (Young Modulus and percentage of openings in the panel).
The high influence of the masonry tends to reduce the variability of the periods for the considered building classes, even if a significant variation is still observed. As for the bare frame configurations, the variability is mainly related to the design approach and, consequently, to the dimensions of the columns. The adopted design approach, affecting the panel-to-frame stiffness [Smith, 1969], modifies the contribute of the masonry infills to the global stiffness of the buildings. The influence of the infills is lower in stiffer buildings designed according to seismic codes. The obtained results pointed out the effectiveness of the

formulation proposed by [Perrone *et al.*, 2016]; this formula is more accurate to predict the fundamental periods of more flexible buildings, as also observed for the bare frame buildings.

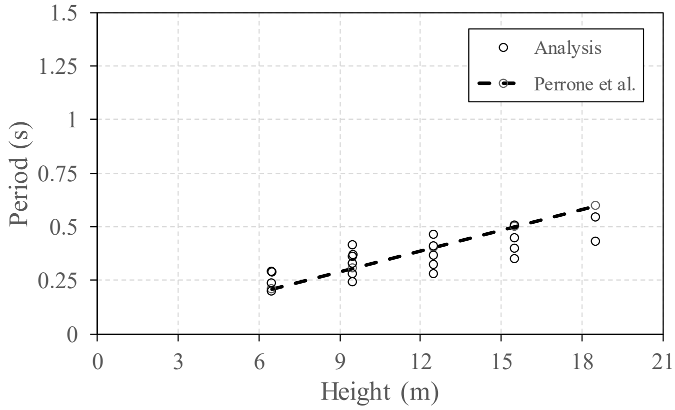

Figure 4 – *Elastic period infilled buildings (Ew=1500 Mpa)*

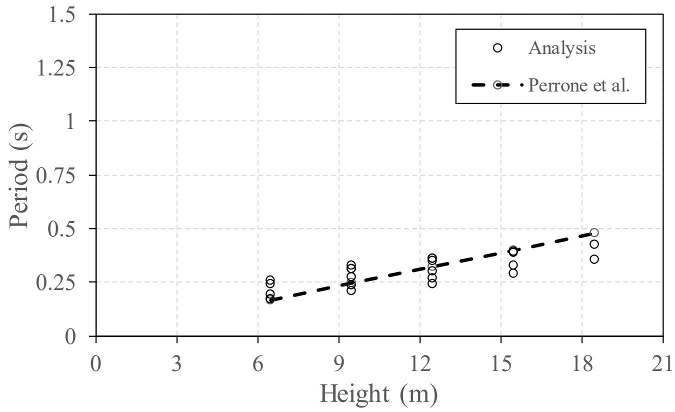

Figure 5- *Elastic period infilled buildings (Ew=3500 Mpa)*

In Table 2 are compared the mean fundamental periods of the bare and infilled buildings analyzed in this study. The fundamental periods are significantly reduced by the presence of the masonry infills and increasing E_w the periods further decrease. The buildings with masonry infills characterized by a Young modulus equal to 1500 MPa show a reduction of the period between 47% and 31% for 2-storey and 6-storey buildings, respectively; while, if the Young modulus of the masonry infills is equal to 3500 MPa, the percentage decreases of the first period is equal to 55% and 45% for 2-storey and 6-storey buildings, respectively.

Table 2 – *Mean values of the fundamental periods*

Building Configuration	H (m)				
	6.5	9.5	12.5	15.5	18.5
Bare	0.48	0.60	0.68	0.64	0.71
Infilled (E_w=1500 Mpa)	0.25	0.33	0.38	0.42	0.49
Infilled (E_w=3500 Mpa)	0.21	0.28	0.31	0.35	0.39

The dynamic properties of the structures affect the seismic demand experienced by the buildings. The accurate evaluation of the fundamental period could significantly improve the seismic vulnerability assessment both for studies performed at individual and urban scale. The usual range of variation of the fundamental periods observed for bare buildings leads to lower Spectral acceleration (S_a) with respect to the S_a calculated if the masonry infills are considered during the eigenvalue analyses. Figure 6 reports the elastic design spectrum for the urban center of Bovino considering a return period of the seismic action equal to 475 years. In Figure 4 are also marked the mean fundamental periods of the 4-storey building for the bare and infilled building configuration. Due to the introduction of the masonry infills, the spectral acceleration that should be considered for the design/assessment of the 4-storey building increases of 14%. The influence of the masonry infills in the evaluation of the S_a is higher for more deformable buildings (buildings designed only for gravity loads); in the case of the analyzed 4-storey buildings the highest percentage difference in terms of S_a is equal to 87%.

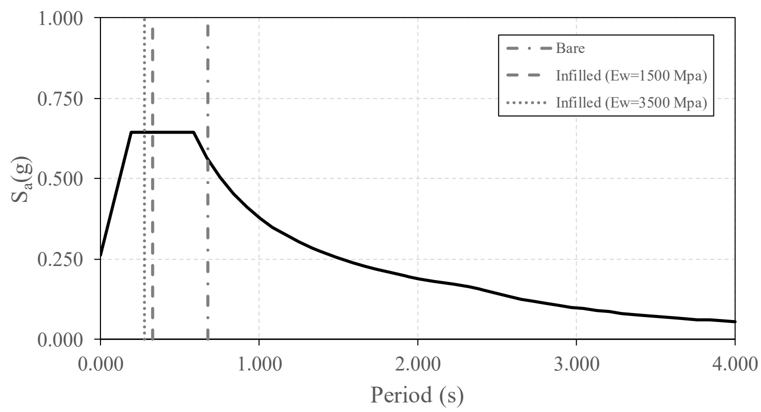

Figure 6- *Comparison of the S_a for the 4-storey building*

4 Methodology for the seismic vulnerability analysis at the regional scale

For the analyses aimed at the application of the proposed methodology, a set of models for each building typology were prepared. Starting from a structural model representative of the single typology, all deterministic parameters were varied within a pre-defined range. The variability of the parameters was evaluated through a direct approach, i.e. by creating a structural model for every possible combination of the parameters values, but excluding situations having no practical relevance (e.g. structure that are instable also with only vertical loads). The implementation and selection of those models having features coherent with the assigned variables was made by means of a simple tool designed in VBasic language. Specifically, the selected parameters set is presented in table 3, where it has been named as *x* the direction parallel to the maximum dimension of the building plan.

Table 3 – *Variation range of selected parameters for each building typology*

A1		A2		B1		B2	
6	8	6	6	3	3	4	4
2	5	3	6	2	4	2	4
4m	6m	4m	6m	5m	7m	5m	7m
5m	6m	5m	6m	5m	7m	5m	7m
Yes	No	Yes	No	Yes	No	Yes	No
Bare, pilotis, infilled		Bare, pilotis, infilled		Bare, pilotis, infilled		Bare, pilotis, infilled	
0.3m	0.4m	0.3m	0.4m	0.3m	0.4m	0.3m	0.4m
10%	20%	10%	20%	10%	20%	10%	20%
0.2m	0.3m	0.2m	0.3m	0.2m	0.3m	0.2m	0.3m
0 m^2/m	2 m^2/m	0 m^2/m	2 m^2/m	0 m^2/m	2 m^2/m	0 m^2/m	2 m^2/m
15MPa	25MPa	15MPa	25MPa	15MPa	25MPa	15MPa	25MPa
10MPa	25MPa	10MPa	25MPa	10MPa	25MPa	10MPa	25MPa
FeB22k	FeB44k	FeB22k	FeB44k	FeB22k	FeB44k	FeB22k	FeB44k

Parameter	# of x-direction frames	# of floors	L_x	L_y	One-direction frames	Infills presence	Infill thickness	Openings percentage in infills	Slab thickness	Balconies incidence	R_{ck} (design)	f_c (real)	Steel type

Once developed all models by varying cited parameters, a simulated design of the structures has been made, following the technical rule prescriptions of the 1980s, and specifically [DM 26th March 1980], with the purpose of defining sections and reinforcing percentage of structural elements.

The selection of the prescriptions to be followed has been made on the basis of the observation that most of the buildings existent in Italy was constructed in periods with lacking prescriptions in terms of adequate details aimed at a good seismic response. In figure 7 this situation is clearly depicted, and the selected code is thought to be well representing the average requirements of the whole buildings of territory.

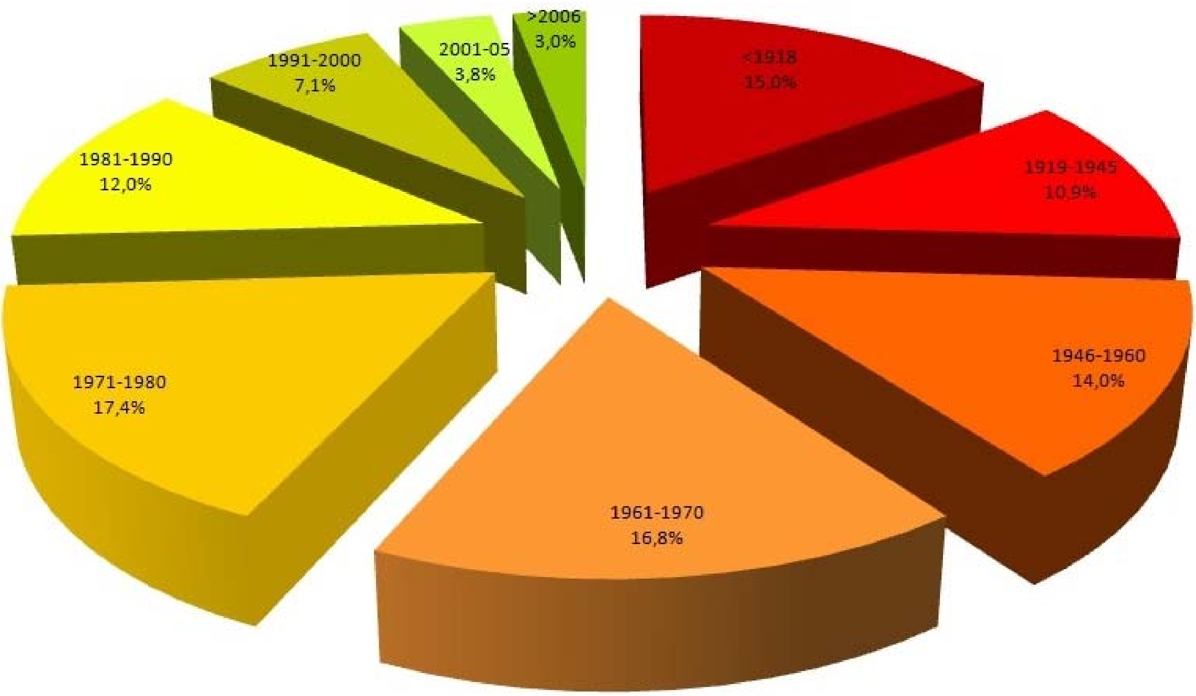

Figure 7 – *Residential buildings percentage on the basis of the construction period in Italy (own elaboration on the basis of ISTAT 2011 census data).*

After their complete definition, each structural model has been subjected to pushover analyses, following the prescriptions of the Italian Building Code (§7.3.4.1 of the D.M. 14th

January 2008), *i.e.* using two different load patterns, both applied in the two main directions of the building: the first proportional to the first modal shape, the second uniform along the building height. Starting from the results of these 4 different analyses, the characteristics of an equivalent elastic-perfectly plastic SDoF oscillator have been deduced, and on this basis, the seismic demands in terms of displacement for three different limit states (IO, LS and NC limit states) have been defined. For this purpose, on the basis of the hypothesis that all of the analyzed structures have a main residential use, a reference period V_R=50 *years* has been selected, and an "A" category ground has been considered.
On the basis of the pushover analyses results, the structural capacity has been computed, which has been operationally defined as the displacement that corresponds to the last analysis step where no one of the structural elements has overcome the specified limit state. This evaluation have been made by assuming a non-linear behavior of the structural elements by means of a lumped-plasticity model, where plastic hinges has been defined according to [FEMA-356].

4.1 Seismic Vulnerability Index definition

For each considered seismic load pattern and for each selected Limit State, referencing the generic structural model the following Seismic Vulnerability Index has been defined:

$$I_V = \frac{d_C}{d_D} \tag{3}$$

where d_C and d_D respectively represent the previosly defined Capacity and Demand in terms of displacements.
In this way, for every model a total of 12 indices has been defined (one for each of the 4 load patterns and each of the 3 limit states), reduced to 3 (one for each limit state) considering only the minimum between those associated to the seismic load patterns.
Consequently, given the n deterministic parameters and the m probabilistic parameters considered within the procedure, named lp_j ($j=1,\ldots,4$) the 4 seismic load patterns, the vulnerability index related to the i-th limit state can be expressed in the following way:

$$I_V(SLi) = \\ = \min_{j=1\ldots4} I_V(lp_j, param_{determ,1}, \ldots, param_{determ,m}, param_{probab,1}, \ldots, param_{probab,n}) \tag{4}$$

Having in mind the subsequent evaluations, the Vulnerability Index defined through equation 3 has been defined for each possible set of the deterministic parameters, assuming that its value is equal to the average of the indexes evaluated by changing the values of the others parameters.
This condition can be expressed in the following way:

$$I_V(SLi, set_{determ}) = \frac{\sum_{j=1}^{n} I_V\left(SLi, set_{determ}, set_{probab,j}\right)}{n} \tag{5}$$

being:

$$set_{determ} = \{param_{determ,1}, \dots, param_{determ,m}\} \tag{6}$$

$$set_{probab} = \{param_{probab,1}, \dots, param_{probab,n}\} \tag{7}$$

4.2 Infills modeling

For the purpose of the infills modeling, an equivalent single-strut macro-model has been selected. Specifically, for the definition of the equivalent struts the procedure suggested in [Cavaleri *et al.*, 2014] was followed.

It's worth highlighting that, in this work, many different variables that are involved when modeling infilled frames, and that can severely affect the actual response of the structure([Fiore *et al.*,2012], [Porco *et al.*, 2015*a*, 2015*b*]), haven't been included.

Figure 8 – *Example of the structure modelling for the 3 considered infills configurations. Clock-wise starting from higher left: image of the building typology, bare frame, completely infilled and pilotis configuration models.*

In figure 8 the 3 considered configurations are depicted.

The non-linear behavior of equivalent struts have been modeled but their state haven't been considered in the definition of the overcoming of the generic Limit State, which is defined only with reference to the structural elements.

5 Some remarks on the variability of results

On the basis of observations on the typical variation range of the structural materials strength in the existent building stock ([Porco *et al.*, 2014], [Uva *et al.*, 2013, 2014*b*]), concrete strength has been assumed ranging from 15 to 25 MPa, and the steel type from FeB32K to FeB38K.

The parameters set presented in table 3, for the purposes of the present work, has been reduced assuming what follows:

- Two-directions frames
- Infill thickness: 0.3m
- Openings in infills: 15%
- Slab thickness: 0.25m
- No balconies
- Concrete strength coherent with design prescriptions

On the basis of the assumed parameters variability, for every set of deterministic parameters associated to a specific building typology, a total of 36 values for the Vulnerability Index has been defined, and specifically 12 values for each of the 3 considered Limit States (IO, LS and NC), 4 for each of the 3 assumed infills configurations (bare frames, pilotis and fully infilled), derived from the possible combinations between the considered values of concrete strength and steel type.

Once defined the Vulnerability Index associated to the deterministic parameters set, defined as the average of the 4 available values, a class-based categorization have been made, defining the classes on the basis of Vulnerability Index ranges as follows:

- Class A: $I_V \geq 1$
- Class B: $0.8 \leq I_V < 1$
- Class C: $0.6 \leq I_V < 0.8$
- Class D: $0.4 \leq I_V < 0.6$
- Class E: $0.2 \leq I_V < 0.4$
- Class F: $0 \leq I_V < 0.2$

On the basis of the presented procedure, the results described in the following tables have been obtained.

Table 4 – *Vulnerability classes (in parentheses Vulnerability Indexes values) defined for the "A1" typology buildings with 6 x-direction frames*

# floors	*Lx*	*Ly*	Infills configuration	*I_V (IO)*	*I_V (LS)*	*I_V (NC)*
3	4	6	infilled	E (0.23)	F (0.07)	F (0.05)
			pilotis	E (0.26)	F (0.07)	F (0.05)
			bare	B (0.94)	E (0.25)	F (0.18)
3	5	6	infilled	F (0.15)	F (0.04)	F (0.29)
			pilotis	E (0.21)	F (0.06)	F (0.04)
			bare	B (0.90)	E (0.24)	F (0.17)
3	6	6	infilled	C (0.61)	E (0.27)	F (0.19)
			pilotis	E (0.21)	F (0.06)	F (0.04)
			bare	B (0.91)	E (0.24)	F (0.18)
4	4	6	infilled	A (1.11)	D (0.44)	E (0.32)
			pilotis	F (0.18)	F (0.05)	F (0.04)
			bare	B (0.98)	E (0.26)	F (0.19)
4	5	6	infilled	A (1.08)	D (0.45)	E (0.32)
			pilotis	F (0.18)	E (0.24)	F (0.17)
			bare	B (0.89)	E (0.24)	F (0.18)

Table 5 – *Vulnerability classes (in parentheses Vulnerability Indexes values) defined for the "A1" typology buildings with 8 x-direction frames*

# floors	*Lx*	*Ly*	Infills configuration	I_V *(IO)*	I_V *(LS)*	I_V *(NC)*
3	3	6	infilled	A (1.13)	C (0.68)	D (0.48)
			pilotis	D (0.40)	F (0.11)	F (0.08)
			bare	B (0.99)	E (0.26)	F (0.19)
4	4	6	infilled	A (1.11)	D (0.41)	E (0.29)
			pilotis	E (0.29)	F (0.08)	F (0.06)
			bare	B (0.99)	E (0.26)	F (0.19)

Table 6 – *Vulnerability classes (in parentheses Vulnerability Indexes values) defined for the "A2" typology buildings with 6 x-direction frames*

# floors	*Lx*	*Ly*	Infills configuration	I_V *(IO)*	I_V *(LS)*	I_V *(NC)*
3	5	5	infilled	F (0.08)	F (0.02)	F (0.02)
			pilotis	E (0.27)	F (0.07)	F (0.05)
			bare	A (1.30)	E (0.34)	E (0.25)
4	4	4	infilled	E (0.25)	F (0.07)	F (0.05)
			pilotis	E (0.31)	F (0.08)	F (0.06)
			bare	A (1.48)	D (0.47)	E (0.34)
4	4	5	infilled	F (0.14)	F (0.04)	F (0.03)
			pilotis	E (0.23)	F (0.06)	F (0.04)
			bare	A (1.88)	D (0.50)	E (0.36)
4	5	5	infilled	C (0.69)	F (0.18)	F (0.13)
			pilotis	E (0.22)	F (0.06)	F (0.04)
			bare	A (1.43)	E (0.38)	E (0.27)

Presented results have been applied to the territory of Bovino, which was selected as reference territory for a territory-based representation of the seismic vulnerability on the basis of different hypothesis regarding the infilled frames. In the following figures, the selected classification has been described through a color-based thematical map, obtained via GIS model, with the purpose of giving an intuitive and immediate information.

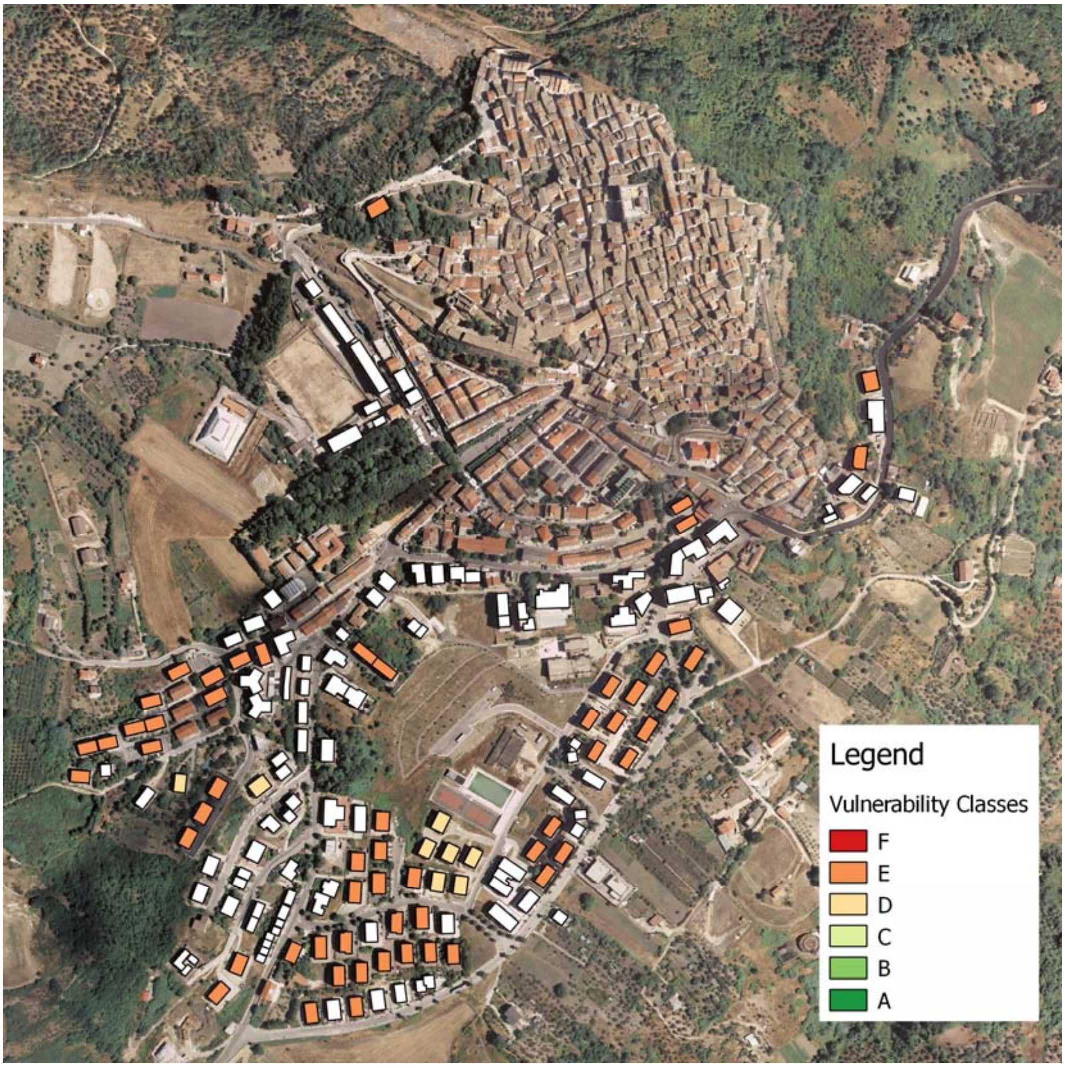

Figure 9 – *Seismic Vulnerability map at LS Limit State considering every building as bare frame*

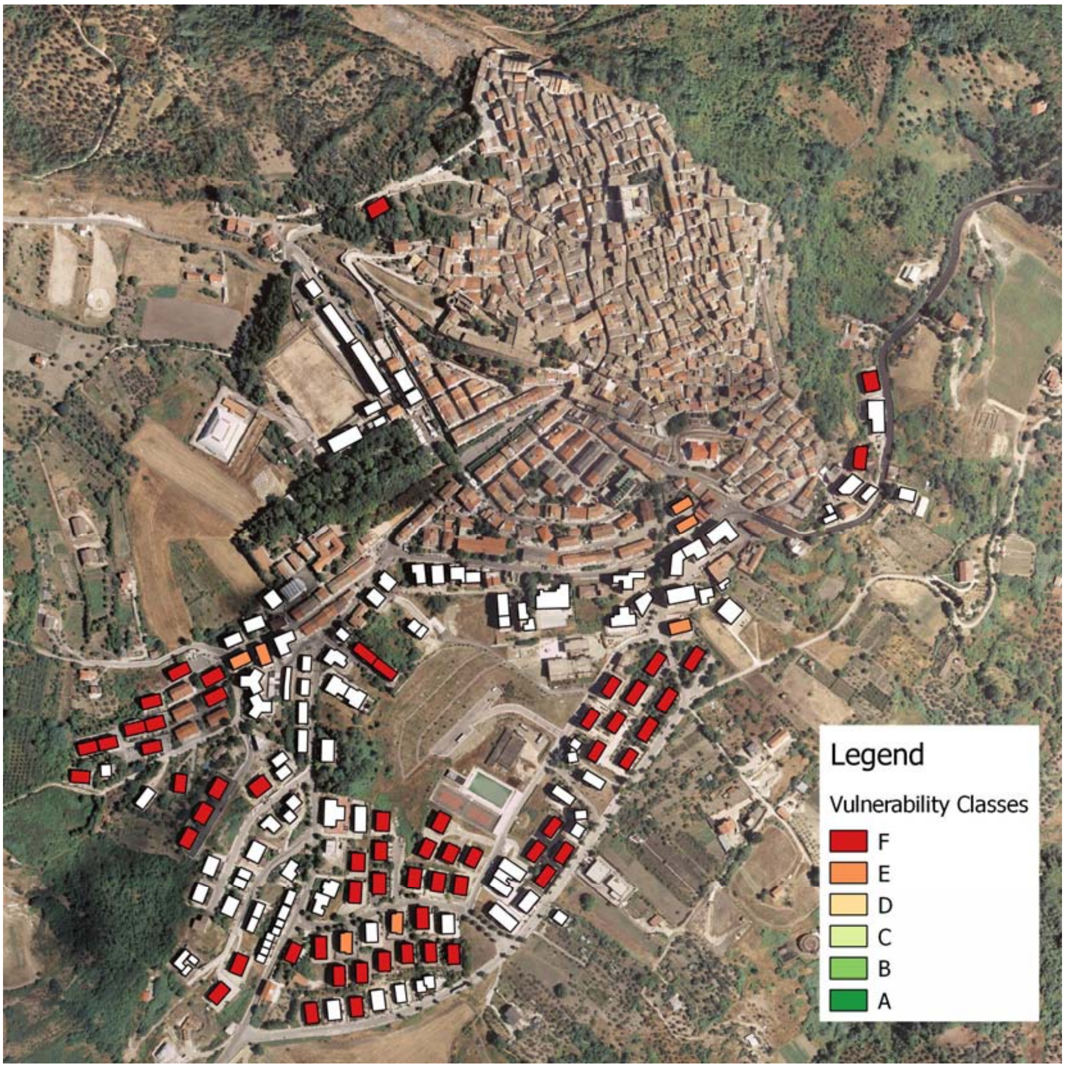

Figure 10 – *Seismic Vulnerability map at LS Limit State considering every building as having pilotis configuration*

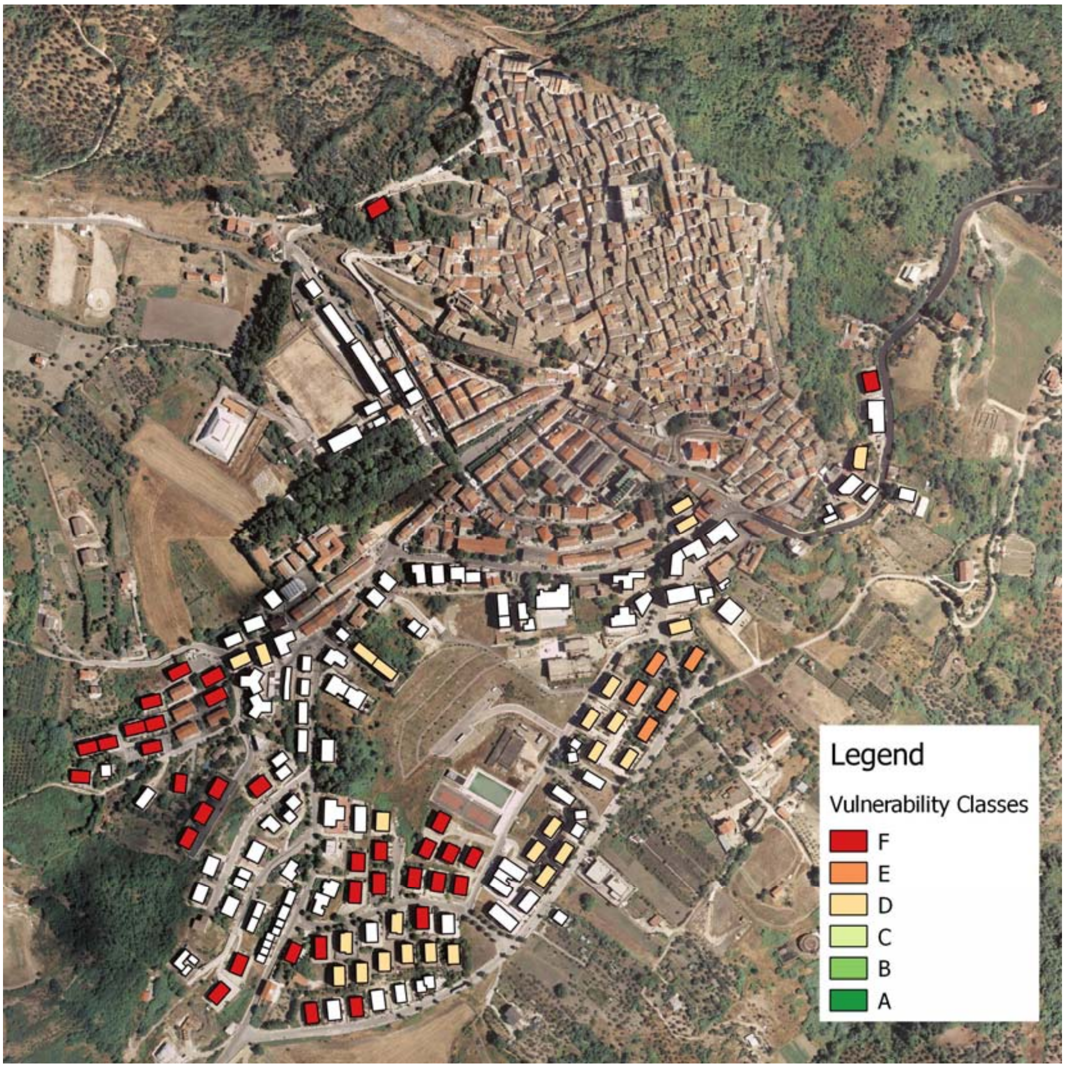

Figure 11 – *Seismic Vulnerability map at LS Limit State considering every building as fully infilled*

6 Conclusions

On the basis of presented results, the influence of infills configuration on the seismic vulnerability assessment can be clearly appraised. By considering 3 different configurations, in fact, a great worsening of the results can be seen passing from the bare frame configuration, through the fully infilled one, to the pilotis configuration.

From figure 10 it can be seen that, if considered as having a “pilotis configuration”, most of the structures result in belonging to the lowest vulnerability classes, while on the contrary assuming any of the other configurations the results appear to be better, proving the great influence of the infill configuration on the analyses results. This effect is specifically imputable both to the increase of the seismic demand, caused by the stiffening effect of the infills, and to the structural capacity decrease, caused by the change in the global structural behavior.

It can be also observed a worsening of the results considering Limit States having higher reference period V_R (i.e. associated to a lower probability of overcoming of the seismic event in the reference period V_R).

In conclusion, it can be observed the absolute necessity of including the infills configuration in the framework of the seismic vulnerability assessment. It's worth highlighting that the equally important influence of the infills mechanical characteristics, not included in the present work, should have to be also included in the assessment procedure, while its evaluation is delayed to further deepenings.

Aknowledgements

The research study was partially funded by the national joint project DPC-ReLUIS 2014-2018 - general theme: "*PR 2 - Strutture in cemento armato*", and Regional Theme "*TT1: "Inventario delle tipologie strutturali esistenti"*").

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ingegneria sismica
International Journal of Earthquake Engineering
Under revision

INFLUENZA DELLE TAMPONATURE NELLE VALUTAZIONI DI VULNERABILITÀ SISMICA SU TIPOLOGIE COSTRUTTIVE RICORRENTI

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SUMMARY: *Scopo del presente lavoro è la valutazione dell'influenza delle tamponature nell'ambito delle valutazioni di vulnerabilità sismica. In particolare, verranno presentati alcuni risultati ottenuti su modelli di edifici dalle caratteristiche ricorrenti nel patrimonio edilizio esistente, e ne verranno valutate le differenze comportamentali e in termini di output dell'analisi sulla base delle diverse ipotesi sulla presenza della tamponatura.*

KEYWORDS: *Vulnerabilità sismica, edifici in c.a., tamponature, strutture esistenti*

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