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INFLUENCE OF THE INFILL TYPOLOGY IN THE EVALUATION OF THE ANNUAL LOSSES OF RC STRUCTURES THROUGH THE APPLICATION OF A NEW METHOD

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Abstract

The comparison of the seismic performance of different typologies of masonry infills is usually related to the technical response according to post-seismic inspection, experimental outcomes, and numerical studies. Although the economic losses due to the achievement of some limit states could represent one of the main aspects that should be taken into account in the selection of the masonry typology to be adopted for the masonry infills, few studies have been addressed to this topic ([1]). Moreover, the economic losses related to the seismic response of masonry infills are usually dealt without a specific procedure that differentiates the masonry of the infill, the infill typology (e.g., traditional vs innovative infill systems), and other aspects which may modify the seismic performance of non-structural element. In this paper, a comparison in terms of the economic losses between different "traditional" masonry infills and an innovative solution with sliding joints [2] is presented through the adoption of a new approach ([8]) that computes the expected annual losses based on the PERR's PBEE procedure.

Keywords: Expected annual losses, infill seismic performances, innovative infill, sliding joints, performance based earthquake engineering.

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1 INTRODUCTION

The seismic performance of buildings is usually evaluated through their structural response that is typically referred to their capacity to withstand earthquake action without threaten human-life and/or exceed specific levels of damage. Although it is widely recognized that the safety of the inhabitants must always be guaranteed, the need to limit the damage, also in case of minor earthquakes, and to reduce the costs of repairing or of the downtime of the activities conducted within the structure, is becoming more and more important.

Focusing the attention on reinforced concrete (RC) frame buildings with masonry infill walls, field observations after major earthquake events have highlighted the vulnerability of these structure, if specific seismic-details are not adopted (*i.e.*, [3],[4],[5]). Moreover, also in case of recently designed buildings or with low seismic actions, where the collapse of the structural members is avoided, the damage could occur in the infill walls due to their high vulnerability and the associated monetary losses could be quite relevant.

In order to improve the seismic response of the infills and to reduce the detrimental effects of the interaction between RC frame and masonry panels, typical for traditional masonry walls built in-place with no specific seismic details, the researchers of the University of Pavia have conceived an innovative infill typology, characterized by the presence of deformable mortar at the frame/infill interface and sliding joints within the masonry panel. Further information regarding the system is provided by Morandi *et al.* ([2]).

This technology proved to be promising, ensuring good seismic performance of the infill-frame system based on an extended experimental campaign, as fully described by Morandi *et al.* ([2]) about the in-plane seismic behaviour, by Milanesi *et al.* ([6]) regarding the out-of-plane seismic response and by Manzini *et al.* ([7]) for the overall building performance.

The recent studies conducted ([1],[8]), mainly developed in numerical researches on this topic (*i.e.* [9],[10]), have also demonstrated that the adoption of such an innovative infill typology could lead to a significant reduction of the costs of reparation of the building and of the elements after significant seismic events, despite an initial higher cost of construction, compared to the same RC structure with traditional infill walls.

This paper aims at presenting the results by applying a novel methodology proposed by Rossi *et al.* ([8]) and evaluate the influence of the choice of a specific infill typology in the computation of the expected annual losses for a typical RC frame building in Italy, built in compliance with current seismic codes. Moreover, the structural and monetary advantages given by the adoption of the innovative infill solution are discussed.

2 NEW METHODOLGY FOR THE EVALUATION OF THE LOSSES IN INFILLED RC FRAMES

A novel procedure to evaluate the expected annual losses (EAL), tailored specifically for RC infilled frame, was conceived by Rossi *et al.* ([8]) within the well-known PEER's PBEE framework ([11],[12],[13],[14],[15]).

The EAL is the expected loss that should be paid every year in order to repair or even replace a specific reference structure. The novel procedure considered to evaluate the EAL of infilled frame structures has been partially adopted in early studies carried by Rossi *et al.* ([16] and [17]). Recently, Di Trapani *et al.* ([1]) presented a similar study, also based on PBEE framework, including an innovative infill solution with sliding joints studied by Preti *et al.* ([18]).

In the present work, the definition of damage in all the considered infills was inferred directly by laboratory tests and by numerical simulations on the same infill panels adopted in the analyses and the computation of losses is referred to updated official costs of repair and replacement actually adopted in Italy. Finally, the evaluation of the seismic performance in terms of EAL of a building with specific indications on infill typologies allows a consistent comparison on the effective contribution of different infill solutions and an estimation of the economic benefits that the use of innovative infill systems could provide as respect to traditional infills.

According to Baker and Cornell ([13]), EAL is computed as a discrete summation:

$$EAL = \Sigma E[DV|IM=im]\cdot\Delta\lambda_{IM}(im) \tag{1}$$

where, *DV* is decision variable, *IM* is the intensity measure, *im* is a specific value of the *IM*, *E[DV|im]* is the expected total repair/replacement cost conditioned on $IM = im$, $\lambda_{IM}(im)$ is the conventional hazard curve, being equal to the mean annual frequency (*MAF*) of *IM* exceeding a certain value of *IM*.

The decision variable, *DV*, is the metric necessary to evaluate the performance of the building and used by the stakeholders to justify their decisions.

Therefore, according to Equation (1) the EAL is the area under the curve, depicted in Figure 1.

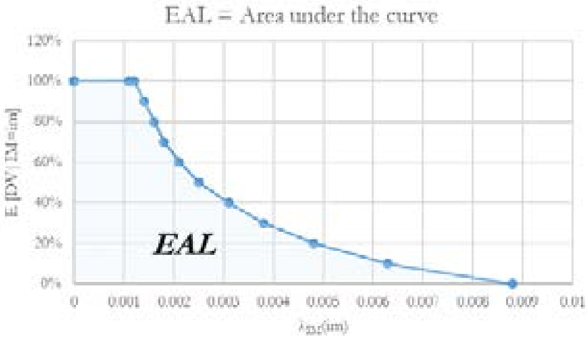


Figure 1. Graphical representation of a generic EAL.

The methodology, in accordance with Cornell and Krawinkler ([11]), could be subdivided in four steps: facility definition, structural analysis, damage analysis and loss analysis.

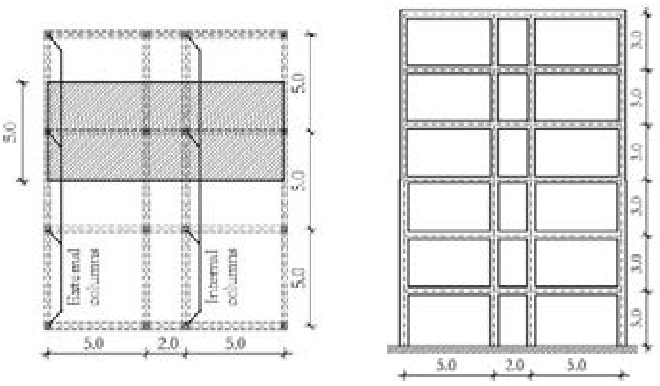
The aspects of novelty introduced in the methodology proposed stay in the fact that the attention is mainly focused on the influence of infill walls in the evaluation of the structural and economic performance of the overall building. In particular, the definition of damage in all the considered infills was inferred directly by laboratory tests and by numerical simulations on the same infill panels adopted in the analyses; moreover, the computation of the losses was referred to updated official costs of repair and replacement actually adopted in Italy. The evaluation of the seismic performance in terms of EAL of a building with specific indications on infill typologies has allowed a consistent comparison on the effective contribution of different infill solutions and an estimation of the economic benefits that the use of an innovative infill system could provide as respect to traditional infills.

3 APPLICATION TO CASE STUDY

3.1 Facility definition

In order to evaluate the influence on infill walls in the seismic performance of a common newly designed 6-storeys high RC frame building in Italy, a reference model was assumed. The structure aimed to represent a typical residential building designed in high ductility class in compliance with the Italian standards ([19]) and Eurocodes ([20],[21]), therefore in accordance with the “capacity design” principles. The building is regular both in plan and elevation and located on a flat ground, with soil type C, in L’Aquila (Italy). Further details on the reference structure are fully described in Hak *et al.* ([22]).

The numerical model is a 2D frame of the entire building, being representative of all the structure, as shown in Figure 2. The analyses have been conducted using Ruaumoko finite element program ([23]) and a macro-model approach was deemed to be the best compromise between reliable results and reasonable computational burden.



(a) (b)

Figure 2. Geometry of the reference building: (a) plan view; (b) elevation view.

In the numerical model, only the structural elements (RC beams and columns) and the infills walls have been modelled, whereas the other non-structural components, assumed to be irrelevant to the structural response of the system, although they could be damaged, have been accounted only *a-posteriori*.

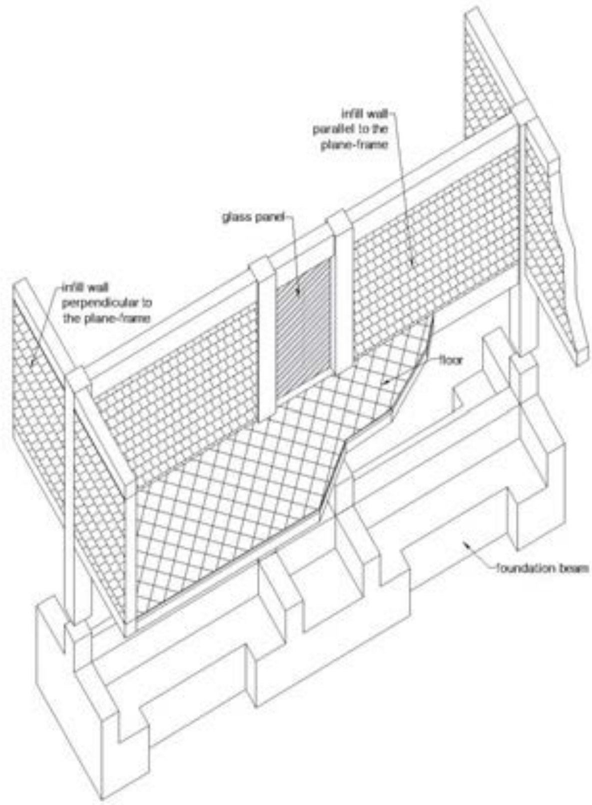


Figure 3. Portion of the structure analyzed.

The additional shear demand on the columns of the frame due to the in-plane thrust of the infill and the out-of-plane verification on the panels have been considered as reported according to Rossi *et al.* ([8]); the local interaction method is based on the specific studies ([24],[25]), meanwhile the out-of-plane verifications are referred to researches based on experimental results (*i.e.* [26],[27],[28],[29],[6]).
The foundation beam is considered to behave perfectly elastically and not damageable, except in case of global collapse of the building. In that case, also the costs of demolition and rebuilding of the foundation beam is accounted. In the central bay of the in-plane frame, a glass panel representing a door is assumed to be placed.
The arrangement of infills and of all the elements is shown in Figure 3, and a list of all damageable elements accounted in the case-study is reported in Table 1.

Typology	Element	Dimension	Quantity		
Structural elements	Foundation beam	Height: 1.7 m; Width $_{top}$: 0.45 m; Width $_{bottom}$: 2.0 m; Length: 23 m	53.25	m^3	in total
	Columns	Section of elements at the 1st, 2nd and 3rd floor: 45x45cm; otherwise: 35x35cm	11.70	m^3	in total
	Beams	Section: 30x45cm	23.65	m^3	in total
Non structural elements	Infill panels – in-plane, 1st floor	2.775x4.55m each	25.25	m^2	per floor
	Infill panels – in-plane, 2nd & 3rd floor	2.55x4.55m each	23.21	m^2	per floor
	Infill panels – in-plane, 4th, 5th, & 6th floor	2.55x4.65m each	23.72	m^2	per floor
	Infill panels – out-of-plane, 1st floor	2.775x4.55m each	25.25	m^2	per floor
	Infill panels – out-of-plane, 2nd & 3rd floor	2.55x4.55m each	23.21	m^2	per floor
	Infill panels – out-of-plane, 4th, 5th, & 6th floor	2.55x4.65m each	23.72	m^2	per floor
	Ground floor slab	60 m^2 (per floor)	60.0	m^2	
	Floor slab, from 1st to 5th floor	60 m^2 (per floor)	60.0	m^2	per floor
	Roof	60 m^2 (per floor)	60.0		
	Ceilings	60 m^2 (per floor)	60.0	m^2	per floor
	Piping (hot and cold water) and electrical, hydraulic and heating systems	Small diameter threaded steel pipes (≤60mm)	60.0	m	per floor
	Internal glass door, 1st floor	2.775x1.55m each	5.33	m^2	per floor
	Internal glass door, 2nd & 3rd floor	2.55x1.55m each	5.10	m^2	per floor
	Internal glass door, 4th, 5th, & 6th floor	2.55x1.65m each	5.20	m^2	per floor
	Sanitary, boilers and machines	Engineering judgement: 140 € / m^2			
	Partition wall	Gypsum walls, 18mm thick	25	m^2	per floor
	Frames	4 windows and 4 doors	10.24	m^2	per floor

Table 1. List of the damageable elements.

With the aim to represent common traditional masonry infill typologies, three different "non-ductile" solutions have been studied. The masonry panels were made of clay units and considered as built in full contact with the surrounding RC frame. Their characteristics are resumed herein and sketched in Figure 4:

1) Typology T2 ("medium strength/stiffness infill solution"). It is represented by double-leaf 12 cm thick walls with horizontally perforated "weak" units, with 60% void percentage. The panel has 1 cm of plaster on each external side of the leaf.
2) Typology T3 ("strong infill solution"). It is made of vertically perforated (50% of void) robust 30 cm thick blocks. The panel has 1 cm of plaster on each side of the wall.
3) Typology TA ("strong infill solution"). Similar to T3 solution, but the vertically perforated blocks are thicker (35 cm) and the tested masonry infills has provided higher deformation capacity than T3. The panel has 1 cm of plaster on each side of the wall.

The strength and stiffness of the infills adopted are different in order to account for a quite wide range of typologies.

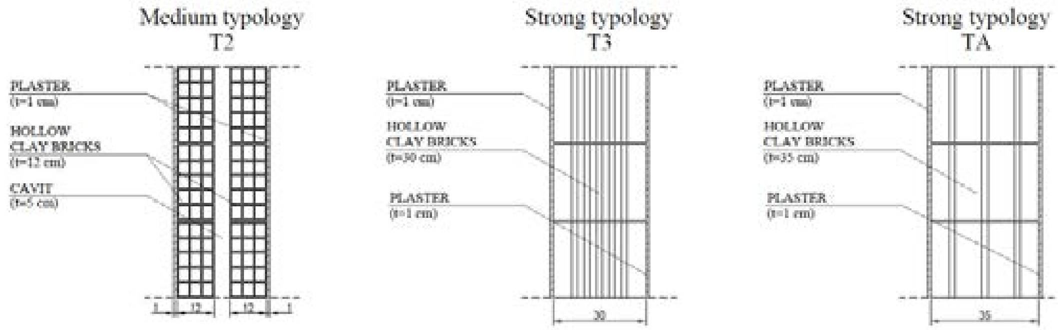
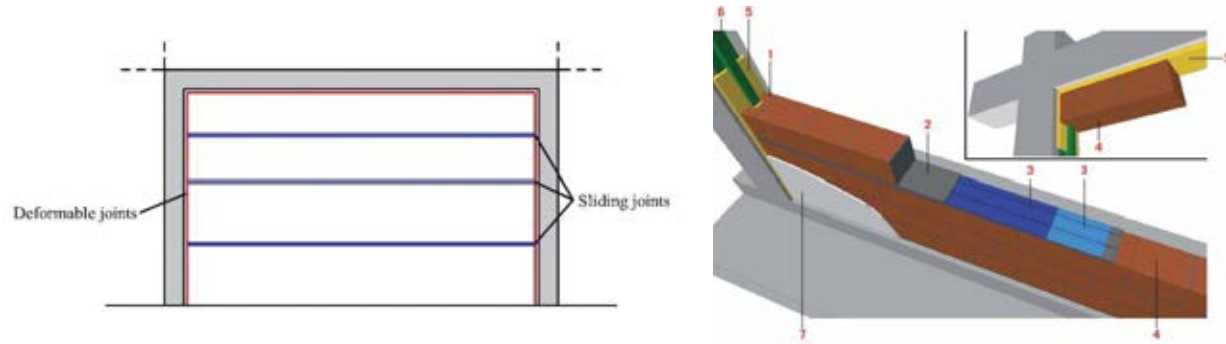


Figure 4. Masonry infill typologies [22].

Besides the non-ductile “traditional” infills, the innovative ductile infill solution with sliding joints (“TSJ1”, [2]) was considered, with the goal to estimate if this solution improves the performance as respect to rigidly attached typologies, also terms of economic benefits. The innovative system was conceived to improve the seismic response of the infills and to control the damage and the stability of the panels, being characterized by the presence of sliding joints that divide the infill into horizontal stripes and that are designed specifically to allow the relative in-plane movement of these stripes ([2]). Moreover, at the wall-frame interface, flexible joints limit the detrimental interaction of the infill with the surrounding RC frame (Figure 5). The unreinforced masonry of the infill is constructed with 25 cm thick vertically perforated lightweight clay units and 1 cm thick general-purpose mortar bed- and head-joints. The overall thickness was equal to 29 cm due to the plaster layer of 2.0 cm which is placed on both sides of the panel. The out-of-plane stability of the infill is ensured by properly designed steel “shear keys” attached to the column and by “C-shaped” clay units placed at the edges of the masonry panel. Further details on this innovative system are described in Morandi *et al.* ([2]) and Milanesi *et al*. ([6]).



(a) (b)

Figure 5. (a) Innovative masonry infill with sliding joints and deformable joints at the panel/frame interface; (b) Details of the innovative masonry infill with sliding joints: 1.C-shape units; 2. mortar bed-joints; 3. Sliding joints; 4. clay units; 5. interface joints; 6. shear keys; 7. plaster [2].

3.2 Seismic Hazard

In order to apply the same (scaled) ground motions to all the building models with different infill walls, there was the need to adopt an intensity measure metric capable to ensure consistency. For the purpose of the present work, the average spectral acceleration, *AvgSa*, introduced in Kohrangi *et al.* ([30],[31]), was assumed to be a good metric, defined as the mean of the log spectral accelerations (*Sa*) at a set of *n* periods of interest (Equation 2 from [31]):

$$AvgSa = [\Pi^{n}_{i=1} Sa(T_i)]^{1/n} \quad (2)$$

This metric is considered a good predictor for both inter-story drift ratio and peak floor acceleration; moreover, its hazard can be evaluated using existing ground motion prediction equations.

Twenty ground acceleration time histories, for each of the ten levels of intensity-measure, were supposed to be sufficient for the purposes of the present study, giving sufficiently good estimation of the most probable response of the structure. Each ground motion, *GM*, had two perpendicular components (North-South and East-West); the one adopted in the analyses was the one with the highest value of spectral acceleration at *T*=1 second. Further information is reported in Rossi *et al.* ([8]).

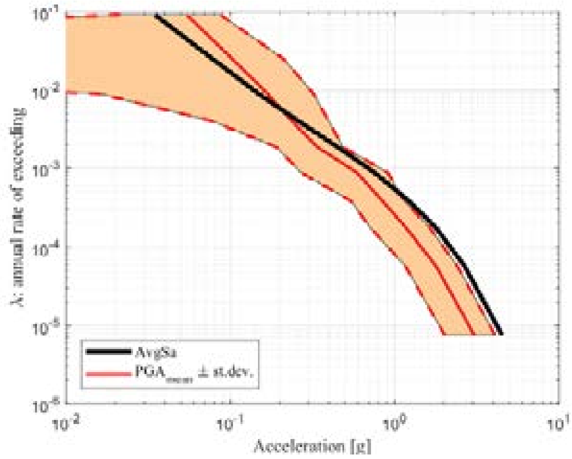


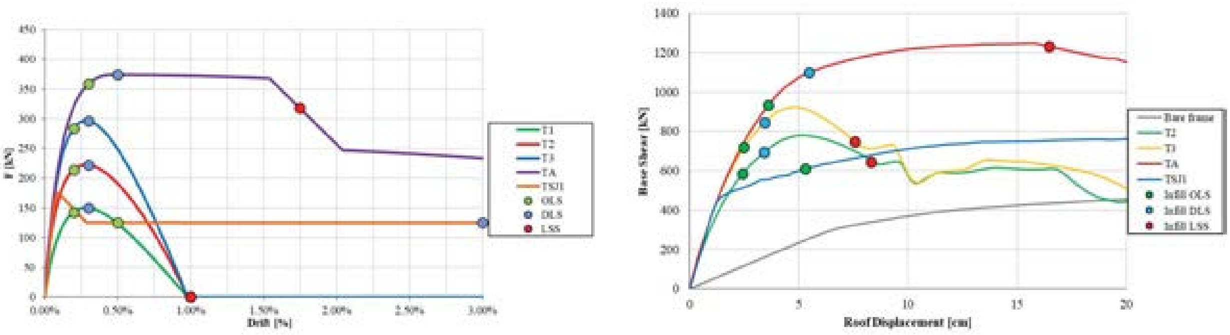
Figure 6. Seismic hazard curve.

4 STRUCTURAL PERFORMANCE

In order to evaluate the infill influence in the overall performance of the building, both nonlinear static and nonlinear dynamics analyses have been performed. The discrete selection of *IM* level, introduced previously, was adopted to perform a MSA (Multiple Stripe Analysis) ([32]). As aforementioned, the numerical models were considering only columns, beams and infill walls. The other non-structural elements were assumed to do not influence the structural behavior of the frame also in the case of the nonlinear time-history analyses.

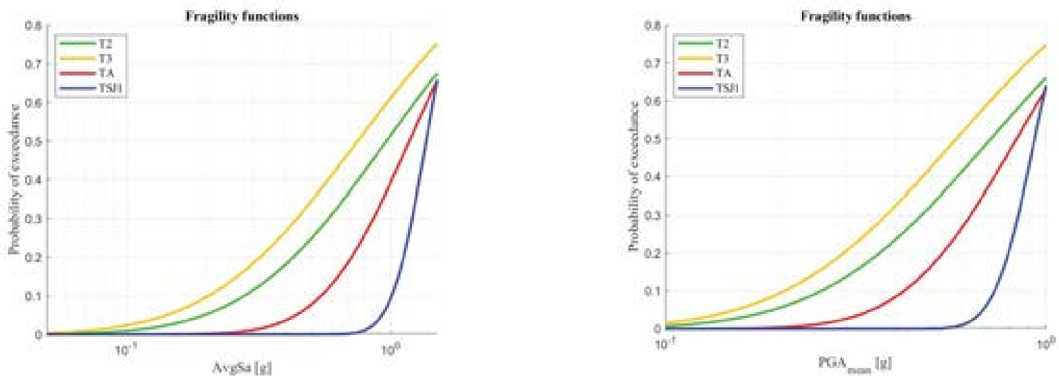
The overall results reported in Figure 7 have demonstrated that the structural performance of the RC frames is strictly related to the structural performance of the typology of infill walls adopted. In particular, Figure 7a, shows the force-drift envelopes of the infill contribution only to the in-plane horizontal response. The dots indicate the thresholds of the three limit states of infill walls in terms of inter-storey drift, as defined by Morandi *et al.* ([33],[2]): operational limit state (OLS), damage limitation limit state (DLS) and life safety/ultimate limit state (LSS/ULS). In Figure 7b, the pushover curves of the buildings characterized by the presence of the four same infill typologies are shown. Also in this case, the dots show when the performance-level thresholds are exceeded by the infill walls.

The main results of the MSA are represented by the fragility curves of the infilled RC frames, which are reported in Figure 8.



(a) (b)

Figure 7. (a) Envelopes of the infill contributions; (b) Pushover curves.



(a) (b)

Figure 8. Global fragility function related to the collapse of the infilled frames, (a) nonlinear dynamic analyses; (b) nonlinear static analyses.

The fragility function related to the plane-frame with the innovative ductile infill system shows very small values of probability of exceedance till a value of *AvgSa* of about *1g*, where the slope of the curve rapidly increases. In the range of ground motions with *PGA* similar to the design *PGA*, the adoption of the innovative infill solution leads to have a probability of collapse almost equal to zero. This result is quite encouraging, because it demonstrates that the adoption of this innovative infill, conceived with the purpose of limiting the damage in the infill, has also a beneficial influence on the overall in-plane seismic response of the structure.

A peculiarity of this work is the fact that the correlation between the level of damage in the infill walls and the inter-story drift (*i.e.*, the *EDP*: engineering demand parameter) was deterministic. Actually, this fact does not represent a limitation, because the *EDP*-damage relationship was inferred from full-scale in-plane cyclic tests on the same infill panels adopted in the analysis, with the same mechanical characteristics and aspect ratio.

5 MONETARY LOSSES

The results of the structural analyses have allowed the identification of specific values of damage, which can be therefore correlated with costs of repair/reconstruction. The consequence function specifically created for each vulnerable element ([8]), both structural and non-structural, have permitted to define the costs in accordance with the Public Works Price-list of 2020 ([34]) along with an engineering judgement. Table 2 resumes all the cost of construction and demolition for each of the aforementioned elements.

		T2	T3	TA	TSJ1
Structural elements	Foundation beam	21'008 €	21'008 €	21'008 €	21'008 €
	Columns & beams	31'111 €	31'111 €	31'111 €	31'111 €
Non structural elements	Infill panels IN PLANE	16'765 €	19'620 €	21'316 €	23'309 €
	Infill panels OOP (with thermal insulation)	20'366 €	23'222 €	24'917 €	26'910 €
	Floor slabs (roof included)	89'143 €	89'143 €	89'143 €	89'143 €
	Ceilings	17'833 €	17'833 €	17'833 €	17'833 €
	Pipings (hot and cold water)	73'109 €	73'109 €	73'109 €	73'109 €
	Internal glass door	6'915 €	6'915 €	6'915 €	6'915 €
	Sanitary, boilers and machines	50'000 €	50'000 €	50'000 €	50'000 €
	Partition walls	11'982 €	11'982 €	11'982 €	11'982 €
	Windows and doors	35'741 €	35'741 €	35'741 €	35'741 €
	Complete demolition	17'463 €	17'463 €	17'463 €	17'463 €
	Demolition + Building cost	391'437 €	397'148 €	400'539 €	404'525 €
	Building cost only	373'974 €	379'685 €	383'075 €	387'062 €
	Building cost / m^2	1'039 €	1'055 €	1'064 €	1'075 €

Table 2. Costs of demolition and complete reconstruction of all damageable elements.

Furthermore, the global vulnerability functions, related to each building with the four infill typologies, are shown in Figure 9. The costs have been evaluated through consequence functions specifically defined for each element. Further information is reported in Rossi *et al.* ([8]). The expected annual loss (EAL) of one building is the area under its vulnerability curve. In agreement with the previous results, the smallest value of EAL is the one of the RC frame with the innovative infill system (EAL=0.16%). This value is more than about 2 and 4 times smaller than the one of buildings with the TA (EAL=0.28%) and the T2-T3 (EAL=0.57%-0.63%) traditional infills, respectively. The return period of the investment has been found to be always less than 10 years (9 years for TA, 8 years for T2, and 4 years for T3). This period of time has been estimated without considering the risk of injuring and losing the life of people and without considering all the indirect costs and the downtime of the building, which would have furtherly decrease such time.

Finally, according to the Italian law DM 58 ([35]) which defines a method for a simplified classification of the seismic risk for constructions, the building with infill typologies T2 and T3 would lie in risk class A_{EAL}, whilst the building with infill typologies TA and TSJ1 in class A^{+}_{EAL} (*i.e.*, the class with best performance).

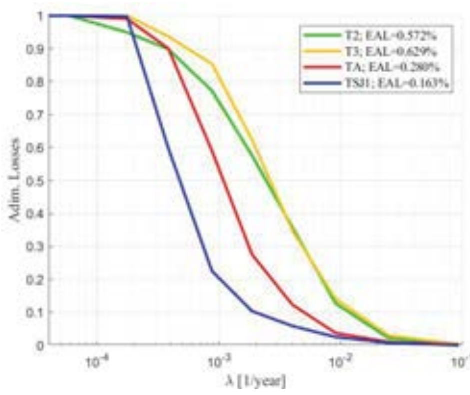


Figure 9. Global vulnerability function relative to the frame structure (Rossi *et al.* ([8]).

6 CONCLUSIONS

Within the present work, a novel method to compute the annual losses has been adopted to compare the seismic performance of three different non-ductile rigidly attached masonry infills and an innovative ductile system with sliding joints [2]. The economic comparison, which has been conducted through nonlinear analyses on code-compliant RC frame structure, has demonstrated the influence of the infill typology in the computation of the expected annual loss (EAL). The choice of a particular infill typology, in a common RC frame structure, could strongly influence both the structural and economic performance of the building, despite the fact that they are assumed to be non-structural elements. The EAL has been calculated through a novel methodology presented by Rossi *et al.* ([8]) in accordance with PERR's PBEE procedure ([11],[12],[13],[14],[15]). In the computation, the influence of the non-structural elements that do not interfere with the seismic structural response has been considered *a-posteriori*.

The monetary losses relative to each building were compared. The loss analyses confirm that the adoption of an infill system conceived with the explicit intent to reduce the damage in the masonry panel, as the innovative solution proposed by researchers of Pavia ([2],[6]), should be preferred as respect to traditional infill solutions without any seismic detail. The initial investment, due to the higher cost of construction of a structure with the innovative infill solution, is paid back by the lower costs of repair in the case of seismic events.

Although the present work shows an updated picture of the situation, the ongoing research on ductile masonry infills with sliding joints (*e.g.,* about the local structure-panel interaction effects [36]), and the continuous development of new typologies of masonry infills (*e.g.*, [37],[38]), represent some of the future upgrades of the presented study.

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