NUMERICAL INVESTIGATIONS ON THE SEISMIC RESPONSE OF MASONRY INFILLED STEEL FRAMES

S. Personeni[1], M. Di Pilato[2], A. Palermo[3] and S. Pampanin[4]

[1]*Research Assistant,* [2]*Post-doctoral Fellow,* [3]*Assistant Professor, Dept. of Structural Engineering, Politecnico di Milano, Campus Leonardo, Milano, Italy*

[4]*Senior Lecturer, Dept. of Civil Engineering, University of Canterbury, Christchurch, New Zealand*
Email: per.sara@libero.it, dipilato@stru.polimi.it, palermo@stru.polimi.it, stefano.pampanin@canterbury.ac.nz

ABSTRACT :

Infill panels can strongly affect the global static and dynamic behavior of seismic resistant structural frames. In this paper, numerical investigations have been carried out on masonry infilled steel frames with rigid or semi-rigid beam-to-column connections. A simple and reliable model based on a lumped plasticity approach is adopted for the structural steel members as well as for the masonry infills, with the in-plane non-linear behavior being simulated by a single (equivalent) diagonal strut element. After a preliminary validation of the model on beam-column joint subassemblies and on a one-storey masonry infilled frame, the response of multi-storey 2D masonry-infilled steel frames are investigated through push-over and non-linear time-history analyses. The results of a series of parametric analyses are presented, by varying not only the mechanical properties of the beam-column connections but also the geometric configuration of the frames and the spatial distribution of the infills. The numerical validation of the model onto experimental results available in literature confirms the reliability of the model presented. The parametric analyses performed on the multi-storey frames provide preliminary useful indications for an accurate seismic design.

KEYWORDS: Masonry infills, multi-storey steel frames, strut model, rigid / semi-rigid connections

1. INTRODUCTION

Increasing interest in the effects of infill panels to the inelastic response of both reinforced concrete (RC) and steel frame structures has been observed during the last five decades. Extensive studies based on either experimental or analytical research programs have been conducted to provide a better understanding of the topic, especially after observations of post-earthquake buildings damage around the world. (Figure 1).

a) b) c)

Figure 1 – Observed damages of infilled steel frames: a) global collapse for soft storey mechanism and b) out of plan failure of the infill (Bam Earthquake, 2003); c) global collapse of the structure

At present, the knowledge of this topic is far from having a univocal and proven solution to the problem. Despite this, codes in different countries recognize the relevance of the infill panels on the seismic response of buildings, not only in terms of regularity and uniformity of stiffness/strength distribution, but also for the

possible influence on the global and local collapse mechanism [Kaushik *et al.*, 2006]. The complex interaction between infills and surrounding frames has been investigated since the mid-1950s. Researchers focused on the study of various systems with different combinations of frame and infill materials [Shing *et al.*, 2002]. While comprehensive experimental and analytical studies are now available in literature in relation to RC structures (very common worldwide, both in Mediterranean Countries and in the US), there is a lack of information on the seismic response of infilled multi-storey steel frames. Although a good knowledge of the behavior of bare steel frames has been obtained, due, in part, to extensive experimental tests performed on beam-column connections [SAC, 1997], investigations on the effects of their interactions with masonry infills have mainly focused on simple one bay-one storey frames, both from a theoretical [El-Dakhakhni *et al.,* 2003; Biondi *et al.*, 2006] and experimental point of view [Moghaddam, 2004; Mander *et.al.*, 1993].
In this contribution, the results of numerical investigations on the seismic behavior of multi-storey steel frames, with and without masonry infill panels, are presented. A lumped plasticity model is adopted for the steel members with equivalent diagonal struts representing the masonry infills. Experimental-numerical validations of quasi-static tests on beam-column steel connections, as well as on bare and infilled one-storey frames available in literature are first presented. Parametric push-over and time-history analyses on three different frames considering the influence of the mechanical properties of beam-column connections and the geometrical distribution of masonry infills have been carried out. The preliminary results confirm that these infills can significantly affect the damage and failure mechanism of code-designed seismic resistant steel frames and thus cannot be neglected in seismic design. Appropriate simplified design methodologies are required to account for the effects of frame-infill interaction on the overall seismic performance of steel frames.

2. MODELLING ISSUES

The intent of the numerical investigations focuses on a general understanding of the influence of infill panels on the global seismic response of infilled frame systems. In consideration of the inherent uncertainties associated to the mechanical properties and cyclic behavior of the masonry as well as the mechanical properties of the panel-frame interfaces, the use of a simple macro-modeling approach was considered the most viable solution, in opposition to more sophisticated modeling approaches (e.g. based on fiber elements or 3D finite elements). Due to these considerations, the numerical analyses were carried out using a nonlinear structural analysis program RUAUMOKO [Carr, 2006], based on a lumped plasticity approach for the structural members with the use of an equivalent diagonal strut for the infills (a generally acknowledged method).

2.1. Modeling of Structural Members: Lamped Plasticity Approach

The structural steel beams and columns have been modeled using elastic beam finite elements, while all the inelastic behavior of the system is lumped in rotational springs placed at the beam-column interfaces and at the column bases (plastic hinges). The moment-rotation relationship of the springs is defined by appropriate hysteresis rules, able to describe the cyclic behavior of the steel members and the different peculiarities of the beam-to-column connections (i.e. bolted or welded). For welded connections the typical Baushinger effect is well captured by hysteresis rules devised specifically for steel elements (Ramberg-Osgood [Kaldjian, 1967] and Dodd-Restrepo [Dodd and Restrepo-Posada, 1995]), while for bolted connections the characteristic pinching and stiffness degradation, due to bolts sliding, suggest the use of bi-linear or tri-linear hysteresis rules (Takeda [Otani, 1974] and Wayne-Stewart [Stewart, 1987]).

2.2. Modeling of Masonry Infill Panels

Over the years, different analytical models have been developed to evaluate the influence of masonry infill panels on structural behavior. The differences between them are related to their complexity and the ability to reproduce typical failure mechanisms of infilled frames. The most common of these failures are: horizontal sliding cracking at the mid-height, diagonal cracking, sliding of multiple bed-joints and a diagonal strut mechanism with two distinct parallel cracks, often accompanied by corner crushing. Recent and comprehensive overviews of the topic, including a comprehensive state-of-the-art survey of modeling approaches and alternative analytical models, can be found in literature [Crisafulli, 1997; Shing and Mehrabi, 2002]. The simplest and most widely-adopted approach to model the infill panels is the use of equivalent diagonal struts linking the centers of the beam-column joints (Figure 2). Several researchers have contributed to improving the model, proposing different semi-empirical expressions to properly evaluate the strut width (i.e. Holmes [1961], Stafford Smith [1967], Mainstone and Weeks [1970]). Modeling issues related to the sliding shear behavior of the masonry panel and a variable number of the equivalent struts have also been investigated [Crisafulli, 1997; Biondi *et al.*, 2006; El Dakhakhni *et al.*, 2003]. In this contribution, in consistence with the lumped plasticity approach adopted for the steel structural elements, a single diagonal strut model has been adopted, associated with the hysteresis rule proposed by Crisafulli [1997] to describe the axial cyclic behavior of infill panels.

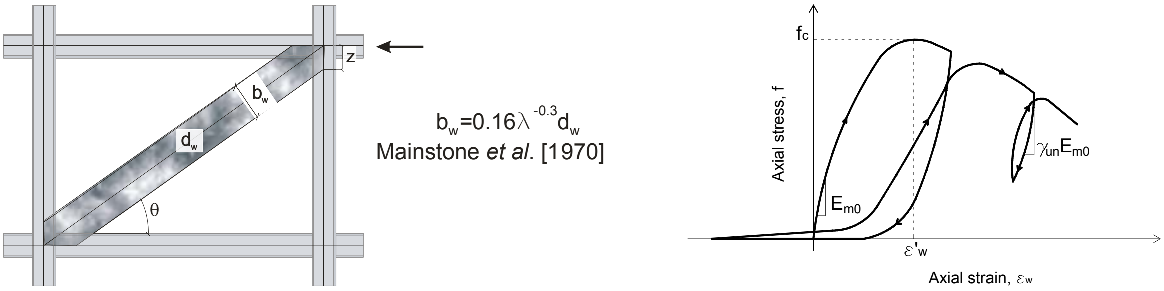


Figure 2 – Equivalent strut model for masonry infills and the associated cyclic stress-strain (Crisafulli, [1997])

3. VALIDATION OF NUMERICAL MODEL

3.1. Beam-Column Joints

A first validation of the numerical model adopted for the steel structural members has been performed on experimental results of quasi-static cyclic tests on beam-to-column connections (American SAC Project [SAC, 1997]). The reliability of the adopted model has been confirmed referring to beam-to-column subassemblies with different connection types (welded or bolted), structural dimension and mechanical properties (stiffness, strength and energy dissipation capacity).
An example of the numerical-experimental comparison of the cyclic response of semi-rigid welded connections (SAC Code: FF-UCSD [SAC, 1997]), subjected to a quasi-static cyclic testing protocol, is presented in Figure 3, where the dashed red line represents the numerical results obtained using a Ramberg-Osgood hysteresis rule. As shown by the agreements of the force-displacement curves reported in Figure 3, the adopted model satisfactorily simulates the global cyclic behavior of the subassembly and the typical Baushinger effect.

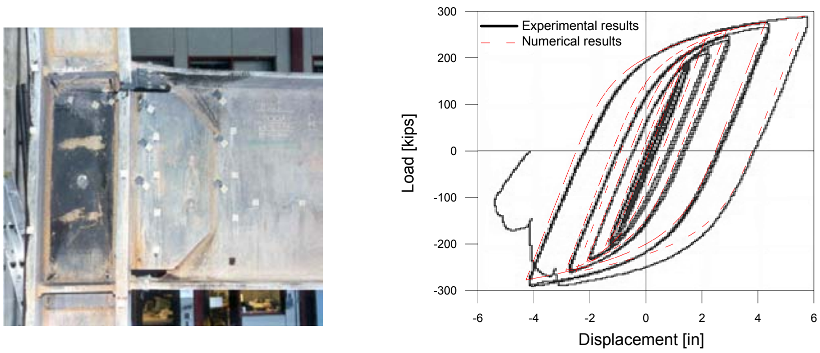
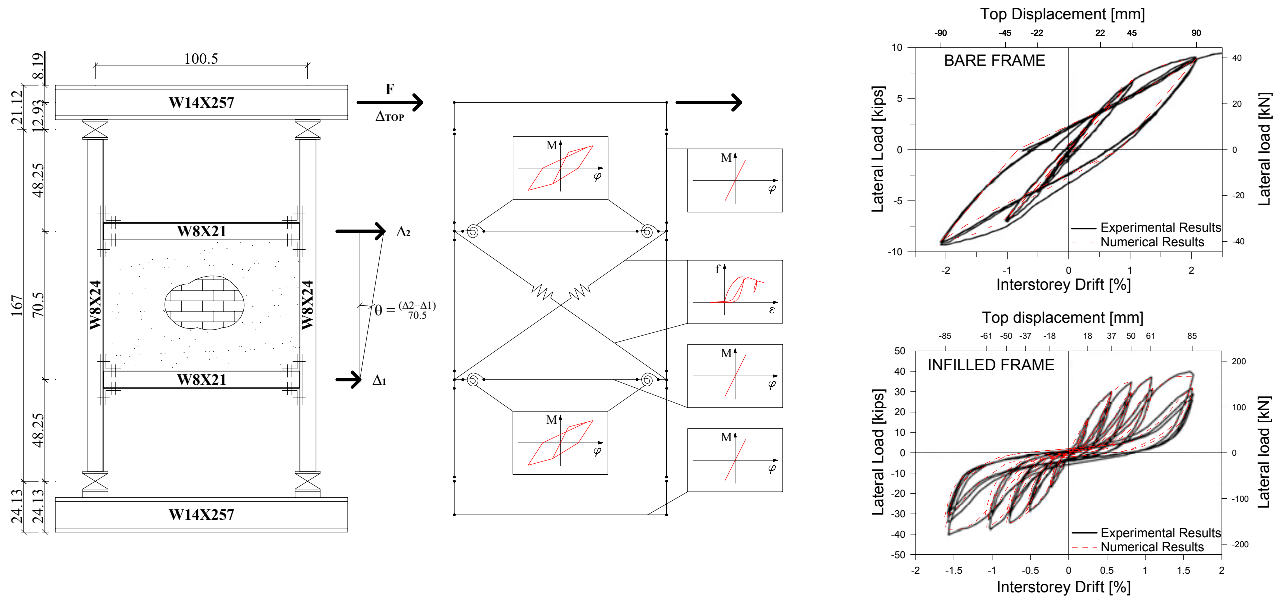


Figure 3 – Semi-rigid welded connection: tested beam-column subassembly, analytical model (Ramberg-Osgood hysteresis rule) and numerical-experimental force-displacement curves.

3.2. One-Storey Frame with Infills

The equivalent strut model adopted for the infill panels, using the Crisafulli hysteresis rule, has been validated on the basis of quasi-static cyclic tests performed at the NCEER at State University of New York, Buffalo [Mander *et al.*, 1993], as shown in Figure 4a. The experimental investigation aimed to evaluate the performance

of steel frames with semi-rigid bolted connections, infilled with clay brick masonry with and without retrofit. In order to validate the numerical model, comparisons were drawn with the experimental test response considering both bare and infilled frames. For the latter, a masonry panel (3.5 inches thick) reinforced on one side with a ferrocement overlay (1/2 inches thick) has been positioned between the steel members.
The steel frame modelling complies with the characteristics previously discussed. The Wayne-Stewart hysteresis rule has been implemented to describe the cyclic behaviour of the rotational springs, and the mechanical parameters have been calibrated using the experimental results of the bare frame specimen. This hysteresis rule, created for plywood nailed timber walls, if properly calibrated can successfully describe the cyclic behavior of the steel bolted connections, typically characterized by pinching phenomena. The stress-strain cyclic relationship of the diagonal struts [Crisafulli, 1997], adopted to describe the infill panels, has been calibrated on the basis of the mechanical properties of the masonry and successively modified to fit the experimental results. The strut width is defined by the empirical expression proposed by Mainstone *et al.* [1970] and displayed in Figure 2. Further details can be found in Personeni [2007]. The model representation of the infilled frame is illustrated in Figure 4b. The comparison presented in Figure 4c indicates a good agreement between numerical and experimental results in terms of global response (base shear-top displacement curves); the model predicts in a satisfactory way the stiffness, strength and energy dissipation of both bare and infilled frames.



a) b) c)

Figure 4 –Quasi-static cyclic tests on one-storey frame [Mander *et al.*, 1993]: a) test set-up; b) numerical model; c) numerical-experimental comparison for bare and infilled frames

4. NUMERICAL INVESTIGATIONS ON MULTI-STOREY FRAME SYSTEMS

The implementation and calibration of the lumped plasticity model previously illustrated provided a simple and reliable tool for the prediction of the behavior of masonry infilled steel frames.
The feasibility of the numerical model allows the investigation into the influence of different parameters on the seismic response of the system. Therefore, parametrical analyses have been performed varying the height of the frames (5, 8 and 10 storeys, Figure 5), the type of beam-column joint (rigid and semi-rigid welded connections) and the distribution of infills along the frame elevation. Based on common architectural layouts, three relevant structural configurations of planar infill panels have been considered: bare frames - BF, uniformly infilled frames - UIF, and non-uniformly infilled frames - NUIF (with no infills at the first floor). All of the bare frames are designed following the provisions of Eurocode 8 (2004) and according to a Direct Displacement Based Design (DDBD) approach [Priestley *et al.*, 2007] with design drifts of 1.7%, 1.1% and 0.9% respectively. These drifts correspond to the artificial plateau of a pseudo-displacement design spectrum, using a corner period of 3.0 seconds. The EC-8 design spectrum corresponding to PGA of 0.35 g and a soil type B has been used.

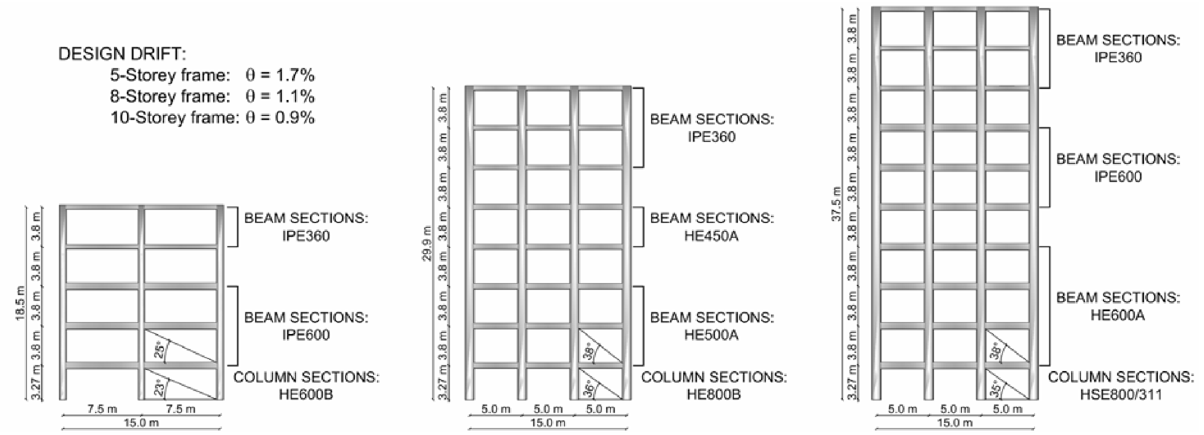


Figure 5 – Multi-storey frame systems: mechanical and geometrical data of analyzed frames

4.1. Push-over analyses

Force controlled push-over analyses have been performed with a triangular (first mode) distribution of the applied forces. In Figure 6a, a typical base shear vs. top drift curve represents the monotonic response of the 8-storey frame with rigid connections, for the three different configurations (BF, UIF, NUIF). As expected, the presence of the masonry walls in the structural grid leads to a favorable increase in both the stiffness and strength of the system. On the other hand, the infilled frames, when compared with the corresponding bare frames, are characterized by a different distribution of the internal forces along the elevation of the frame, with a concentration of shear stresses in the first floors of the frame and the consequent major contribution of the steel columns to the base shear (dashed line in Figure 6a).

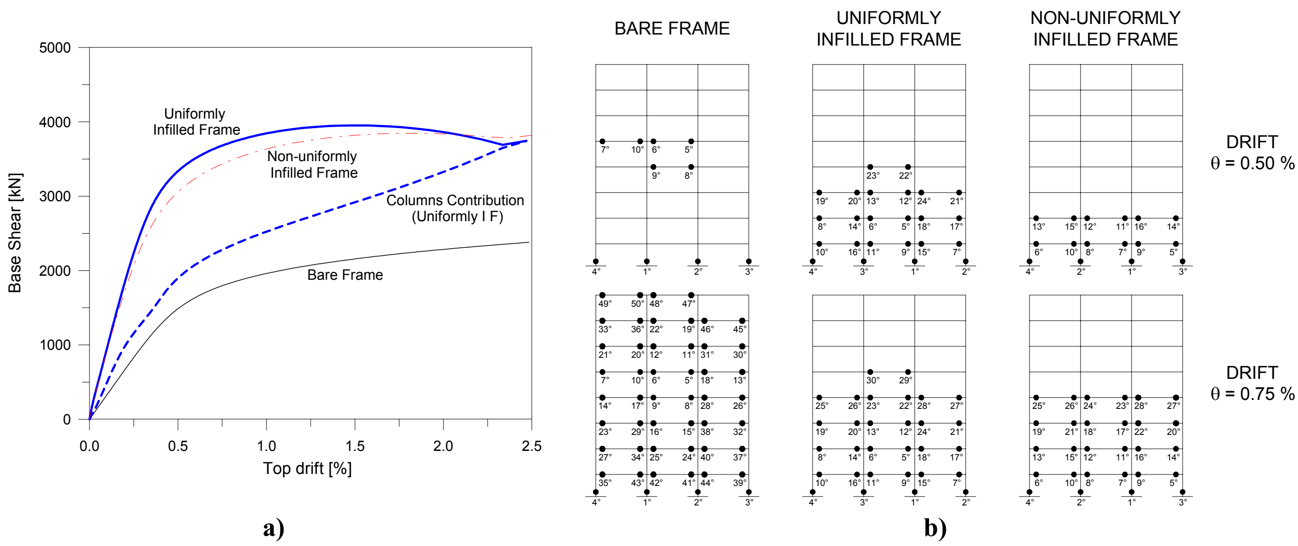


Figure 6 – Pushover analyses on 8-storey frames: a) monotonic response; b) comparison of the damage mechanism, with activated plastic hinges at 0.50% and 0.75% top drift

In Figure 6b the global inelastic mechanisms are shown for the same case study (8-storey frames). A concentration of the inelastic demand occurs for both infilled frames (UIF ad NUIF). In fact, the UIF experiences extensive cracking and premature failure of the infills at the second floor level, while the absence of infills at the ground floor in the NUIF can lead, as expected, to a typical soft-storey mechanism at that level.
For all the multi-storey frames described in Figure 5, with both rigid or semi-rigid welded connections, the effects of infills are investigated by means of two parameters: the initial stiffness increase with respect to the correspondent bare frame and the top drift corresponding to the peak of the base shear vs. top drift curve. As clearly shown in Table 1 for the case of the UIF, the stiffness increase due to masonry panels is higher for

frames with semi-rigid connections. This can be attributed to the increased activation of strut elements at the early stages of the load history, due to the lower stiffness of the structural system. Furthermore, the positive contribution of the infills decreases as the frame height increases. Together with the increasing contribution of the infills, the inter-storey drift corresponding to the maximum strength of the infilled frame is reduced, due to the earlier collapse of the panels.

Beam-to-column connection type		5-storey frame rigid	5-storey frame semi-rigid	8-storey frame rigid	8-storey frame semi-rigid	10-storey frame rigid	10-storey frame semi-rigid
Increase in initial stiffness in respect to the bare frame	$(K_{0\_uif} - K_{0\text{-}bf}) / K_{0\_bf}$	440 %	553 %	277 %	350 %	181 %	232 %
Top drift at the pick of the base shear-top drift curve	θ_{max}	1.08 %	1.09 %	1.52 %	1.47 %	1.95 %	1.87 %

Table 1 – Summary of results of pushover analyses on uniformly infilled frames

4.2. Time-History Analyses

The dynamic behavior of the frames referred in Figure 5 has been studied investigating the influence of building height, stiffness of beam-column connections and masonry infills configuration. Twenty recorded and properly scaled natural accelerograms [Pampanin *et. al.*, 2002] have been used, with satisfactory compatibility between the mean elastic response spectrum and the EC-8 response spectrum adopted for the DDBD procedure.
According to FEMA-302 [NEHRP, 1997], two different earthquake intensity levels have been considered in the numerical analyses, subjecting the structure to two corresponding response spectra: the Maximum Considered Earthquake (MCE) ground shaking, (probability of exceedance of 2% in 50 years), and the Design Earthquake (DE) ground shaking (probability of exceedance of 10% in 50 years). Referring to a performance objectives matrix [SEAOC, 1995], the Basic Safety Objective is attained when a structure achieves both the Life Safety Performance level under the DE level and the Collapse Prevention Performance level under the MCE level.

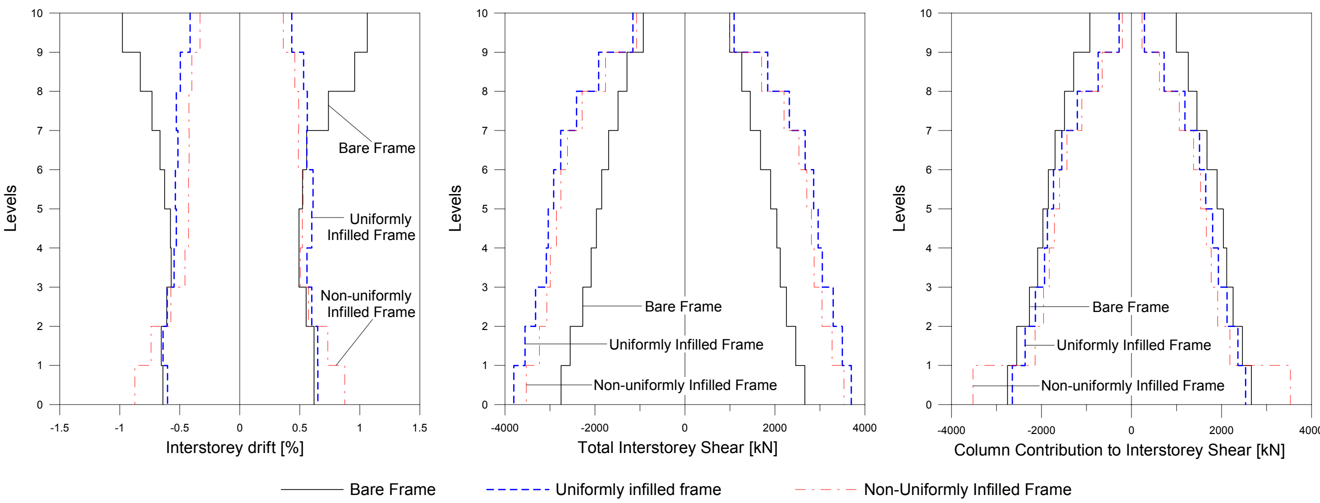


Figure 7 –Time-history analyses on the 10-storey BF, UIF and NUIF with rigid connections: maximum inter-storey drift, total inter-storey shear and column contribution to inter-storey shear for Design Earthquake (DE) intensity

In Figure 7, the results related only to the 10-storey frames with rigid connections are presented. It is worth noting that in the bare frame the drift profile diverges from the constant distribution assumed in the design, due to the marked influence of higher vibration modes. Confirming the results of push-over analyses, the dynamic analyses show that the presence of infill panels leads to a different structural behavior of the frame, moving the maximum drift demand down from the top floors (BF) to the first floors (UIF and NUIF). A constant increase of total inter-storey shear demand due to the presence of the infills is also evident (Figure 7, center). Furthermore, for the case of NUIF, the column contribution to the inter-storey shear at the ground level appears to be significantly higher than for the BF and UIF conditions (Figure 7, right).
This effect is emphasized in the 5-storey frame, here not reported. In fact, due to the lower inclination of the equivalent masonry struts and consequently higher shear demand in the columns, the irregular distribution of

infills along the elevation leads to the collapse of the frame under the action of the most severe earthquake.
The influence of the infills can drastically affect not only the maximum profile, but also the residual drift/displacements of the system, which, according to recent advances in performance based seismic design [Pampanin *et al.*, 2002], have been suggested as an important complementary damage index for a post-earthquake performance assessment of the structure. In Figure 8 the performance matrices, corresponding to the DE and MCE intensity levels, are presented for the 10-storey frames with rigid connections, plotting maximum vs. residual inter-storey drift. Table 2 presents the summary of results of the non-linear dynamic analyses on BF, UIF and NUIF subjected to the selected earthquake ground motions. It is evident how the increased stiffness provided by masonry infills (UIF and NUIF) leads to a noteworthy reduction of both maximum and residual drifts, especially at the DE level. As expected, this effect is more significant for the residual drift/displacements, due not only to the increased stiffness of the system, but also to the natural "self-centering" behaviour of the Crisafulli hysteresis rule adopted to simulate the masonry struts.

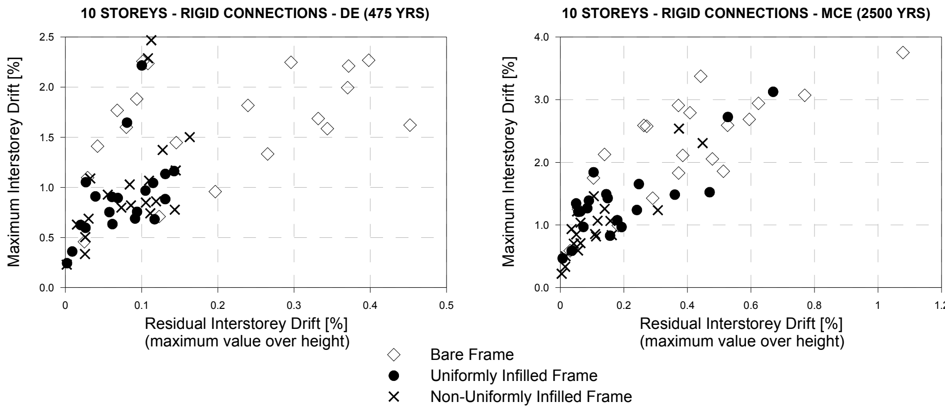


Figure 8 – Residual displacement based performance matrices: DE (Design Earthquake: 500 years) and MCE (Maximum Considered Earthquake: 2500 years) ground motion intensity

		No. of Storeys	Maximum drift [%] bf	u-if	nu-if	Residual drift [%] bf	u-if	nu-if	Residual/Max Drift bf	u-if	nu-if
RIGID CONNECTIONS	DE	5	2.05	1.37	2.19	0.30	0.14	0.32	0.13	0.09	0.12
		8	1.53	1.01	1.26	0.17	0.09	0.12	0.10	0.08	0.08
		10	1.48	0.91	1.02	0.21	0.08	0.08	0.14	0.09	0.08
	MCE	5	3.10	2.53	3.32	0.73	0.37	0.41	0.20	0.13	0.12
		8	2.21	1.53	1.88	0.33	0.24	0.24	0.13	0.12	0.10
		10	2.01	1.37	1.63	0.40	0.21	0.32	0.19	0.14	0.17
SEMI-RIGID CONNECTIONS	DE	5	2.25	1.48	2.15	0.27	0.13	0.26	0.11	0.08	0.09
		8	1.70	1.04	1.28	0.24	0.10	0.11	0.13	0.08	0.07
		10	1.62	0.96	1.03	0.23	0.15	0.12	0.13	0.13	0.10
	MCE	5	3.32	2.59	3.32	0.71	0.54	0.45	0.16	0.15	0.12
		8	2.47	1.62	1.90	0.39	0.25	0.27	0.14	0.13	0.12
		10	2.23	1.43	1.67	0.31	0.24	0.28	0.13	0.15	0.15

Table 2 – Mean values of maximum drift, residual drift and residual/maximum drift ratio for the three different masonry infills configurations: bare frame (bf), uniformly infilled frame (u-if), non-uniformly infilled frame (nu-if)

5. CONCLUSIONS (FIRST)

In this contribution, the influence of masonry infills on the behavior of steel frames has been investigated, in line with increasing international interest in the topic shown over the last decades for both existing and designed buildings. Push-over and time-history analyses have been performed through simple computational models based on a lumped plasticity approach for both the structural and non-structural components, in order to investigate the global seismic response of steel frames, with and without infills.
The reliability of the model has been confirmed by the numerical validation of tests on beam-to-column subassemblies and one bay-one storey frame. The results of numerical parametric analyses have confirmed that the masonry infills strongly affect the static and dynamic response of the structural system in terms of global

stiffness and strength, as well as maximum and residual inter-storey drift. Furthermore, the presence of masonry infills can change the sequence and distribution of the inelastic demand within the structure, with the possibility of a concentration of inter-storey deformation/drift demand at the lower floors. In particular, for non-uniform masonry infill configurations, this can result in a premature collapse due to the activation of a soft or weak storey mechanism. On the other hand, provided the development of such a mechanism is protected, a comprehensive performance based design approach should rely on the positive contribution of the masonry panels. In fact, especially for low ground motion intensity level, infills can provide a reduction of both maximum and residual drift demand, due to the increased stiffness and strength.

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