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NONLINEAR STATIC ANALYSIS OF INFILLED RC FRAMES: A NOVEL SIMPLIFIED PROCEDURE

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Abstract. *Masonry walls are generally utilized in Reinforced Concrete (RC) structures, either as external infills or as internal partitions. Although the interaction between them and structural members significantly affects the seismic response of structures, infills are usually not included in the structural models and, hence, their mechanical contribution is neglected in seismic analysis and assessment of existing structures. However, additional strength provided by infills may either play a beneficial role or affect the resulting structural capacity. Although several proposals are currently available for simulating the nonlinear response of masonry infills in structural analysis, no well-established methods emerged so far for evaluating the global response of infilled RC frames.*
This paper proposes a simplified procedure based on Nonlinear Static (NLS) analysis and an extension of the well-known N2 Method. A statistical assessment and validation is also presented with the aim to confirm accuracy and reliability of the proposed analysis procedure.

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1 INTRODUCTION

In the last fifty years, Reinforced Concrete (RC) frames have been widely realized also in seismic areas, such as the Mediterranean region. In these structures, masonry walls are generally used as both internal partitions and external infills.

Although the behaviour of masonry infills may significantly affect the overall structural response as well observed in recent seismic events [1], only RC members are considered in practice-oriented structural analysis, while masonry infills are considered to contribute to the structural response by mean of their weight and mass only [2]. However, such a simplification, generally accepted for new structures designed according to the Capacity Design rules in which the stiffness of columns is higher than the one of masonry, may affect the response of existing RC structures characterized by strong infill walls and weak columns [3].

Plenty of researches have been developed on this subject by the scientific community. Specifically, simulating the actual dynamic behaviour of infills and evaluating their influence on the global seismic response of RC structures is the main investigated issues [4][5]: nevertheless, no procedures capable to take into account the contribution of masonry infills in practice-oriented seismic analyses of RC frames have been established so far. As a matter of fact, Nonlinear Static (NLS) analyses are commonly employed nowadays for evaluating the seismic response of both new and existing structures. They are generally performed on bare frame models, but recent developments aiming at introducing the effect of masonry infills have been recently proposed in the scientific literature. One of those procedures is based on formulating a specific R–μ–T relationship [6] for partially dissipative systems. It aims at to consider the effects of a the significant softening response induced by the progressive damage affecting masonry walls during the seismic shaking. Moreover, further improvements have been achieved for assessing the seismic capacity of infilled frames within the framework of the Capacity Spectrum Method (CSM) and the Coefficient Method [7].

Although, such procedures generally result in accurate predictions of the actual seismic response of infilled RC frames, their analytical definitions are formally complicated and based on several parameters whose determination is not generally straightforward. Moreover, if "weak" infills are considered, they do not clearly reduce (as, in principle, one should expect) to the well-known N2 Method widely accepted for bare structures [8].

Therefore, the challenge of formulating a simple, accurate and reliable procedure for determining the performance point of masonry infilled RC structures by means of NLS analysis is still a relevant issue within the scientific community.

A possible extension of the N2 Method to the masonry infilled structures has been recently proposed by Martinelli et al. in 2015 [9]. Specifically, they analysed the capacity curves of both bare and infilled configurations of a series of parametric structures and proposed a scalar "shape" parameter aimed at simply correlating the displacement of the infilled structure to the one of the bare configuration evaluated via the N2 Method.

In the present paper, the aforementioned procedure is firstly presented, then a further validation of such a proposal is outlined based on a parametric set of structure wider than the one originally used by the Authors [9]. Specifically, NonLinear Time History (NLTH) and NLS analyses performed on two- four- and six-storey structures are herein integrated by considering one-, three- and five-storey buildings in which the monotonic and cyclic response of infill walls is simulated through the computationally convenient approach based on the "equivalent strut" concept [10]. Finally, a sample application on an existing frame considered as a relevant case study is reported.

2 OVERVIEW OF THE SIMPLIFIED PROCEDURE

Martinelli et al. [9], based on the results of several numerical experiments and comparisons between the capacity curves of bare and infilled structures, suggested a direct estimation of the displacement demand of infilled frames via NLS analyses. Specifically, the top displacement of a framed structure with masonry infills can be obtained from the one evaluated via N2 Method [8] for the corresponding bare structure:

$$\Delta_{top,infill}^{NLS} = \alpha \cdot \Delta_{top,bare}^{NLS}, \tag{1}$$

where α is a parameter defining the "shape" of the two capacity curves corresponding to the infilled frame and the corresponding bare one. Specifically, the shape parameter α is defined as the ratio between the areas beneath the two aforementioned capacity curves, up to a displacement equal to the demand determined through the N2 Method on the bare structure:

$$\alpha = \frac{A_{bare}^{NLS}}{A_{infill}^{NLS}}. \tag{2}$$

As a matter of fact, the nonlinear behaviour of a structure with and without infills is captured by the capacity curves reported in Figure 1. The proposed procedure is based on evaluating the displacement demand of the bare configuration through the well-know and widely accepted N2 Method (Figure 1,a). Thus, the parameter α is determined by means of eq. (2) (Figure 1,b) and the displacement demand of the infilled structure is obtained through eq. (1).

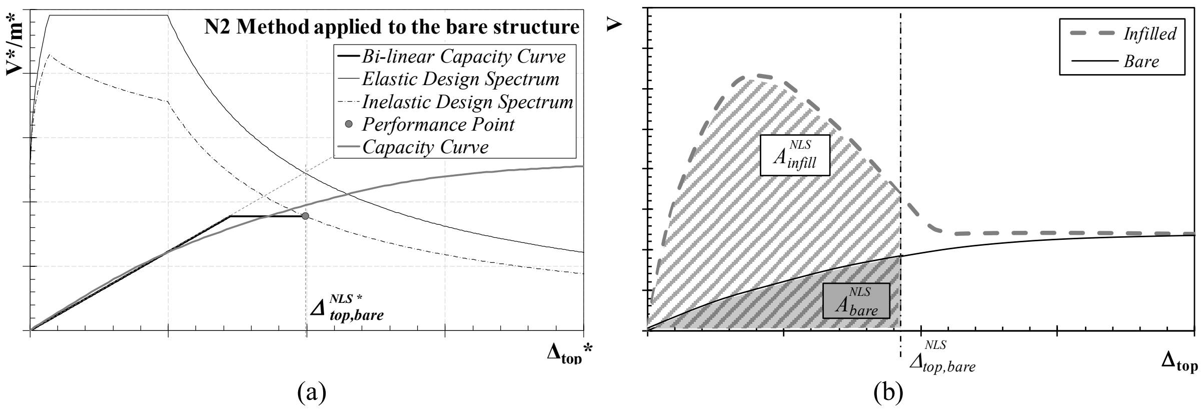


Figure 1: Seismic demand evaluation on the bare structure (a) and comparison between capacity curves (b).

Actually, as it clearly emerges from its definition, the parameter α ranges between 0 and 1. Specifically, values close to *0* are obtained for "strong" masonry walls and/or "low" seismic actions, while values close to the unit denotes "weak" masonry infills and/or "strong" seismic actions. Moreover, the proposed procedure reduces to the widely accepted and validated N2 Method if the gap between the two curves tends to close and, hence, α tends to *1*.

3 THE STRUCTURES FOR THE PARAMETRIC ANALYSIS

Two-, four- and six-storey 3D frames, generated through a simulated design inspired to codes and practices in force in 1960s in Italy [11], are considered in a parametric analysis intended at investigating the mechanical contribution of masonry infills on the resulting response of RC frames. The 3D frames are characterised by three-bays along the y-direction and either three- or five-bays along the x-direction. Further geometric and mechanical properties are omitted herein and can be found in a previous work [9].

As a matter of fact, the simulated design only considers gravitational; it was originally developed for the six-storey frame, whereas two- and four-storey structures were firstly obtained by removing the lower levels [9]. In the present study, three further structures are generated by adopting the same criteria: specifically, one, three and five floors are removed from the six-storey frame in order to obtain one-, three- and five-storey structures. Moreover, only analysis along the x-direction are performed and presented in this paper with the aim to further validate the procedure proposed in [9].

Infilled frames are analysed considering three different distributions of masonry walls. Specifically, type A denotes fully infilled structures in which masonry walls are located in all the , type B refers to structures in which infills are inserted alternatively by bay, while type C indicates frames in which the first storey is not infilled [9].

Moreover, since openings can significantly affect the behaviour of infills, and then the global structural response, different opening ratios are considered. As matter of fact, according to the model by Papia et al. [12], the role of openings is taken into account by reducing with a parameter λ_0 both strength and stiffness of the equivalent strut which simulate infills:

$$\lambda_0 = 1 - 1.5 \cdot \mathrm{a} \geq 0 \quad where \quad \mathrm{a} = l_v / l_w \,. \tag{3}$$

in which l_v and l_w are the horizontal lengths of the opening and the wall, respectively. As previously defined, the parameter a can range between 0 (for which $\lambda_0=1$) corresponding to infill walls without openings and a=0.667 which means to not consider the infill wall in the structural model ($\lambda_0=0 \rightarrow$ bare structure) due to its weak expected response. Consequently, opening ratios a ranging between 0 and 0.60 were considered.

Finally, two levels of the seismic intensity are considered in the parametric analysis. Values of Peak Ground Acceleration (PGA) equal to 0.10 g and 0.35 g are taken into account in order to simulate both low and medium-to-strong seismic shakings. As matter of principle, these two levels of intensity are intended at reproducing the seismic input required for serviceability and ultimate limit states in medium-to-strong seismic areas, according to EC8 [13] and the Italian code [14]. The corresponding two Linear Elastic Design Spectra (LEDS) needed for performing NLS analyses are obtained according to EC8 provisions [13] considering a soil category A and a spectrum type A for buildings. Furthermore, NLTH analyses are developed taking into account two sets of seven unscaled natural accelerograms selected from the European Strong Motion Database [15] which are compatible with the two aforementioned LEDS.

Further details about the seismic input, as well as the characteristics of structures and infills are herein omitted for sake of brevity and can be found in Martinelli et al. [9].

Seismic intensity	0.10 g			0.35 g		
Storeys	1	2	3	4	5	6
Bays along x-direction	3			5		
Infill distribution	A		B	C		bare
Infill openings (a)	0.00	0.20	0.40	0.60		(a=0.667)

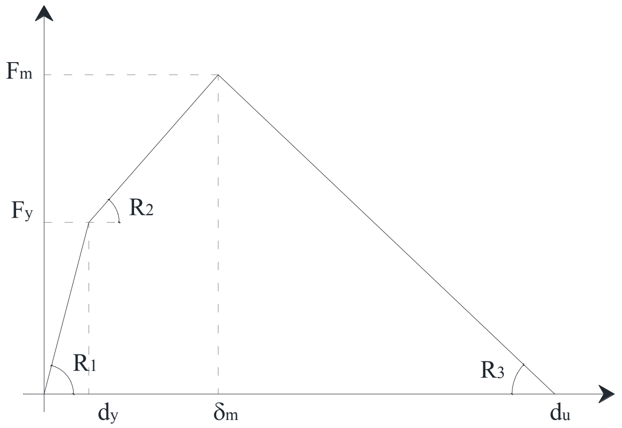
Table 1: Matrix of the cases considered in the parametric analyses.

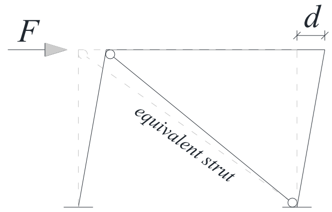
Table 1 reports the matrix of the cases analyses in the parametric study. A total number of 148 structures is analysed against two different levels of the seismic intensity. Specifically, considering that in NLTH analyses two sets of seven accelerograms are taken into account, 148x14=2072 nonlinear dynamic analyses are performed on 3D models. Moreover, 148x2=296 pushover analyses are conducted according to EC8 [13] by applying two different horizontal force patterns (referred to as “modal” and “uniform”) to the structure under investigation.

3.1 Modelling RC frames and masonry infill walls

The RC frames described are modelled in OpenSEES [16] by employing force-based distributed plasticity elements (namely "*nonlinearbeamcolumn"*) available in the software for simulating RC elements. Both cover and core concrete are modelled by adopting the *Concrete01* material [16], which does not consider tension strength of the material, while *Steel01* stress-strain law with hardening ratio equal to 1% is considered for rebars. The models take into accounts both mechanic and geometrical nonlinearity adopting the *"P-Delta geometric transformation"* command included in the software.

Truss elements are used for modelling the "equivalent struts" simulating the masonry infills made of artificial blocks of expanded clay with thickness equal to 30 cm. Specifically, two diagonal equivalent struts which can carry loads only in compression are defined according to the model by Dolsek and Fajfar [17]. Hence, the infill walls are macro-modelled by means of a tri-linear relationship representing their horizontal force-displacement behaviour (Figure 2).





$$R_1 = \frac{G_w t_w l_w}{h_w}$$

$$F_y = 0.6 F_m$$

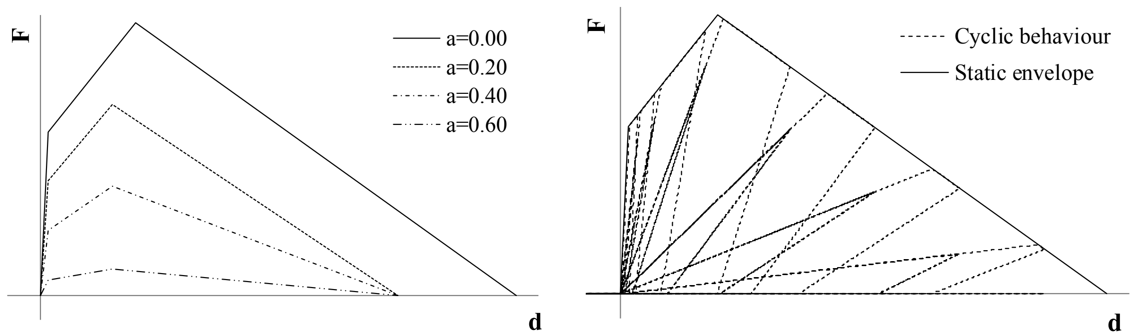
$$F_m = 0.818 \frac{f_{ws} t_w l_w}{1.925 \, l_w / h_w} \left(1 + \sqrt{\left(1.925 \, l_w / h_w\right)^2 + 1} \right)$$

Figure 2: Force-displacement law of the diagonal strut in along the horizontal direction.

Figure 2 shows the force-displacement relationship adopted for modelling the two diagonal trusses simulating the masonry wall with the following meaning of symbols:

- G_w = shear modulus of the masonry;
- t_w = thickness of the wall;
- f_{ws} = shear strength of masonry.

Figure 3 reports some examples of force-displacement curves adopted in this study for simulating the behaviour of masonry infills.



(a) (b)

Figure 3: Force-displacement law of the diagonal strut in along the horizontal direction.

The horizontal displacement d_m corresponding to the maximum force F_m depends on different limitations of the storey drift, assumed equal to 0.2% for short walls, 0.15% for long walls with a window and 0.10% for walls with a door opening. The post-peak negative stiffness of the softening branch is obtained considering an ultimate displacement $d_u=5d_m$ which corresponds a residual strength equal to zero. The presence of openings is taken into account by duly reducing R_1 and F_m through the coefficient λ_0 defined in eq. (3) (Figure 3,a).

The nonlinear response of the simulated equivalent struts was carefully calibrated against experimental results available in the scientific literature [9] and is defined by adopting the "Pinching04" model available in OpenSEES which cyclic behaviour is depicted in Figure 3,b. The cyclic behaviour of the truss clearly demonstrates that such an element only responds in compression.

4 RESULTS OF THE PARAMETRIC ANALYSIS

NonLinear Time History and NLS analyses are carried out on the aforementioned structures considering the two sets of natural accelerograms and the LEDS outlined and the section 3.

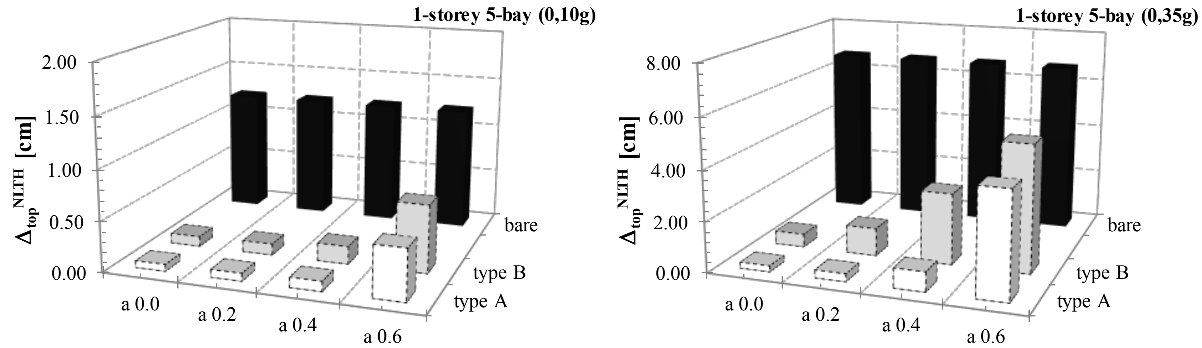


Figure 4: Top displacement demand obtained through NLTH analyses for the one-storey five-bay structures.

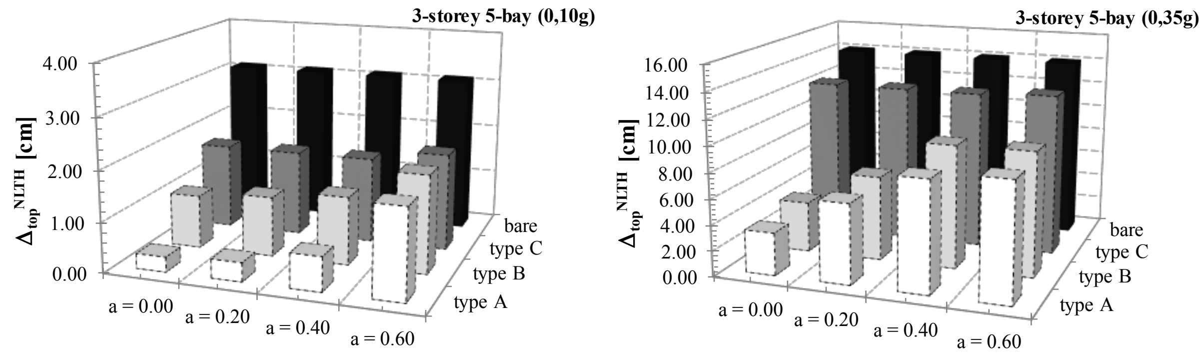


Figure 5: Top displacement demand obtained through NLTH analyses for the three-storey five-bay structures.

The results of NLTH analyses are presented in terms of top absolute displacement demands Δ_{top}^{NLTH}, which represent the average value of the maximum response obtained for the seven accelerograms that simulate the seismic intensity (Figure 4 to Figure 6). Specifically, due to space constraints, only results of the structures with five bays are shown herein, being the ones obtained for three-bay frames very similar to them. Furthermore, it is worth highlighting that results obtained for two-, four- and six-storey frames are not shown herein as they have been already outlined by Martinelli et al. [9], while the new contribution of the present work is represented by the analyses of the one-, three- and five-storey structures.

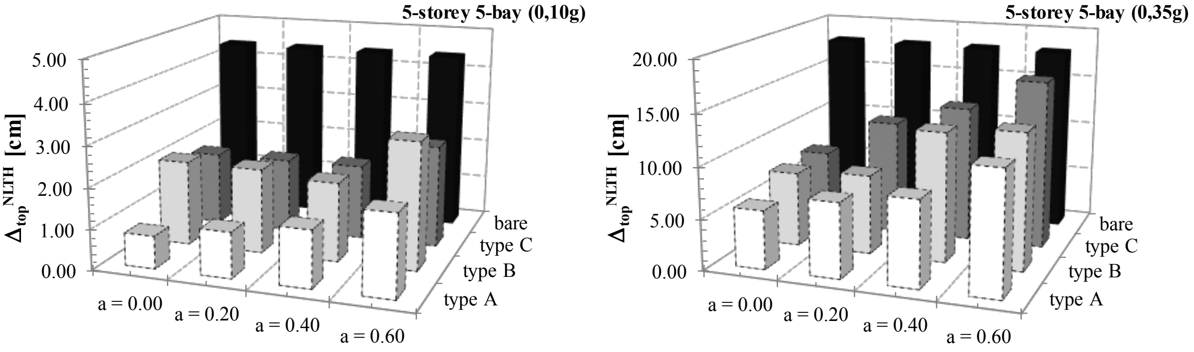


Figure 6: Top displacement demand obtained through NLTH analyses for the five-storey five-bay structures.

As expected, NLTH analyses demonstrate that top displacements of infilled frames are generally smaller than ones obtained in bare structures. Moreover, the Δ_{top}^{NLTH} values reduce with the opening ratio a, as a clear effect of the stiffness of infill which contribute to the lateral capacity of the structure. Although masonry infill walls reduce the displacement demand on RC frames, it is worth to highlight that the results herein presented cannot be merely summarised by claiming that masonry infill walls play a "beneficial" effect on the seismic response of RC frames. In fact, the internal distribution of forces as well as the total force transferred to foundations can be significantly different (and often higher) than the corresponding values determined for a bare RC frame resulting in premature brittle failure of columns and joints which reduces the displacement capacity [18].

4.1 Application of the proposed procedure

NonLinear Static (NLS) analyses are carried out on the same set of frames analysed via NLTH analyses, in both bare and infilled configurations. The representation of the capacity curves derived for both "modal" and "uniform" distribution is herein omitted for sake of brevity. As expected, a significant variation in terms of both lateral stiffness and maximum strength is induced by openings of increasing dimensions [18].

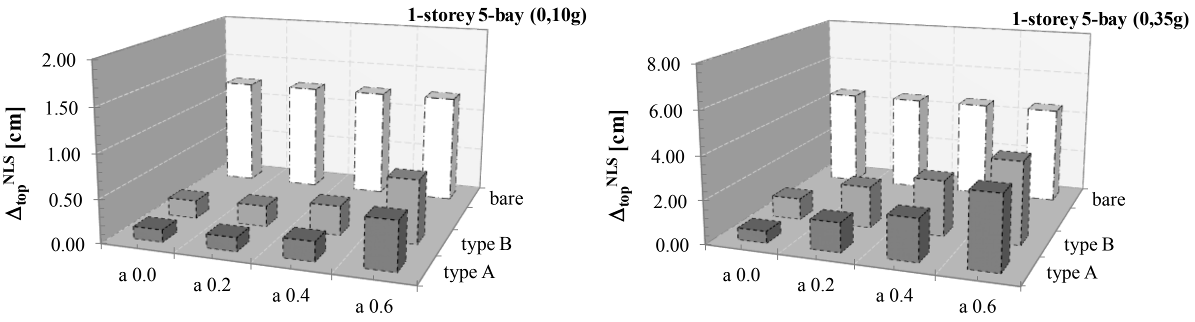


Figure 7: Top displacement demand obtained through the novel procedure for the one-storey five-bay structures.

Then, the values of the top displacement demand of the RC bare frames are obtained by applying the well-known N2 Method [8] and the parameter α is defined according to eq. (2) by considering the "shape" of the two capacity curves corresponding to the infilled frame and the corresponding bare one (Figure 1). Finally, the displacement demand $\Delta_{top,infill}$ of infilled structures is evaluated by means of equation (1) and, according to EC8 [13] provisions, the displacement demand of the structures evaluated through NLS analysis is taken equal to the maximum value between the ones obtained by applying the two different horizontal force distributions ("modal" and "uniform"). Figure 7 to Figure 9 report the top absolute displacement demands

Δ_{top}^{NLS} evaluated by applying the procedure proposed by Martinelli et al. [9] for infilled structures and the N2 Method for the corresponding bare frame. The results about one- three- and five-storey structures are outlined in the same shape already used for presenting the top displacement demand evaluated via NLTH analyses (Figure 4 to Figure 6), while the representation of Δ_{top}^{NLS} for frames with two, four and six storeys is omitted herein for the sake of brevity.

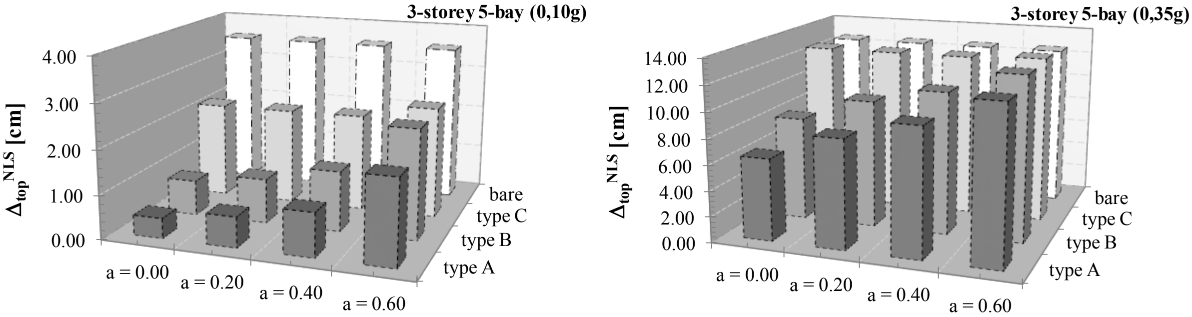


Figure 8: Top displacement demand obtained through the novel procedure for the three-storey five-bay structures.

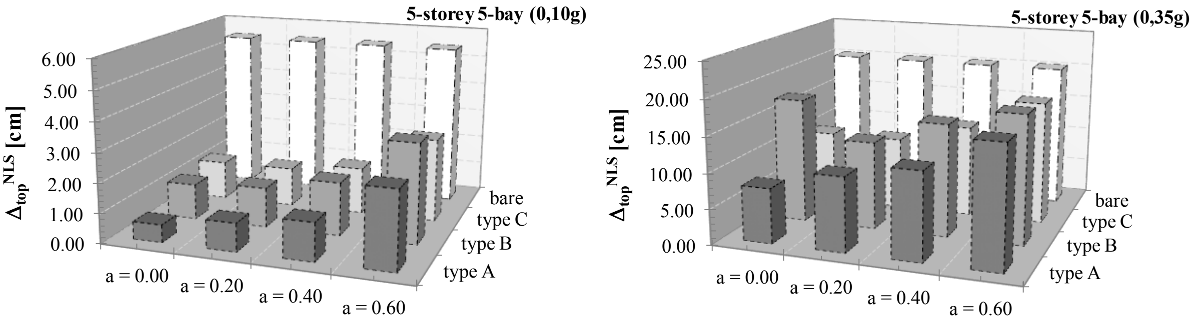


Figure 9: Top displacement demand obtained through the novel procedure for the five-storey five-bay structures.

Figure 7 to Figure 9 demonstrate that top displacements of infilled frames evaluated through the proposed procedure are always smaller than the ones obtained in bare structures. The Δ_{top}^{NLS} values are in agreement with the general trend already observed for NLTH analyses. Specifically, they reduce with the opening ratio a, as a clear effect of the higher stiffness provided by masonry infill.

4.2 Validation against NLTH results and accuracy of the proposed procedure

Martinelli et al. [9] performed a validation of the proposed procedure considering the same two-, four- and six-storey frames used for calibrating and formulating the novel method. They obtained a very high correlation and accuracy of the results compared with ones derived with NLTH analyses. Herein, the results originally obtained from the Authors are enriched by considering one-, three- and five-storey structures with three and five bays, characterised by different distribution of masonry walls and openings. Specifically, the results obtained by applying the novel procedure are compared with the ones evaluated through NLTH analyses.

Figure 10 shows the comparison between the top displacement demand evaluated via NLTH analyses (average value of seven accelerograms) and the corresponding value determined through the proposed procedure. The two charts are divided for outlining the influence of the two level of seismic intensity under consideration. It is worth to highlight that the following comparisons include results obtained for three- and five-bay structures, but they are not explicitly depicted as similar results are obtained for both types of frames demonstrating that the number of bays is a parameter that does not affect the accuracy of the proposed procedure.

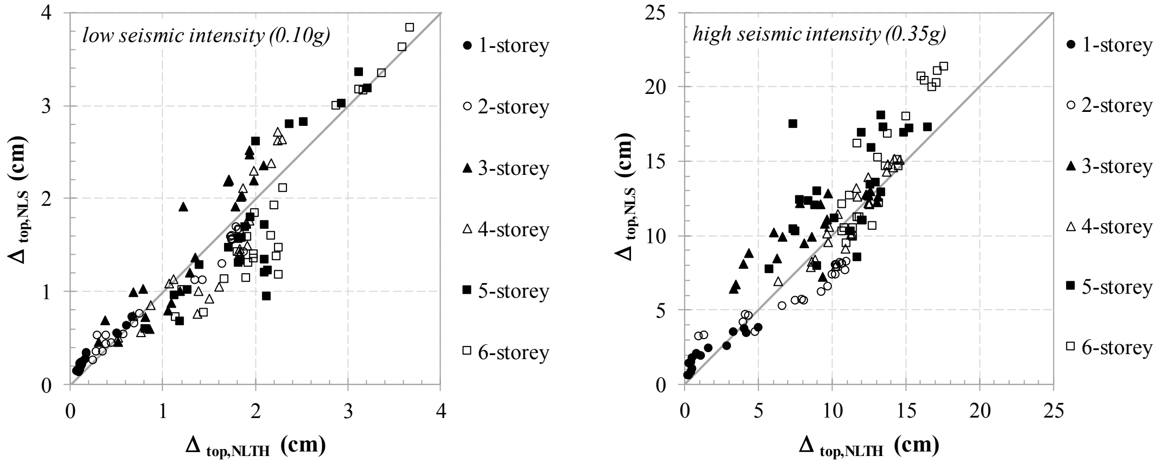


Figure 10: Comparison of the proposed procedure against NLTH analyses.

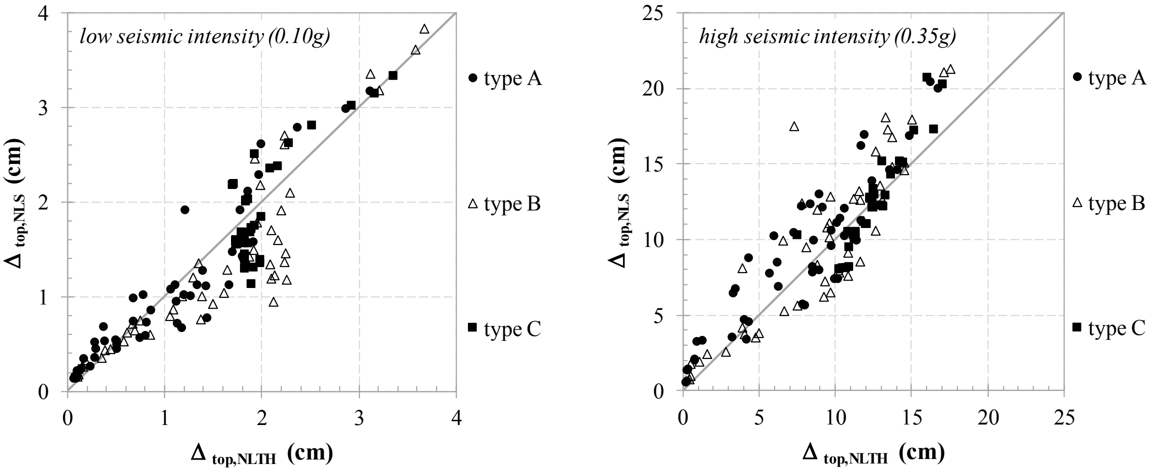


Figure 11: NLS proposed procedure vs. NLTH analyses: structures grouped against the infill distribution.

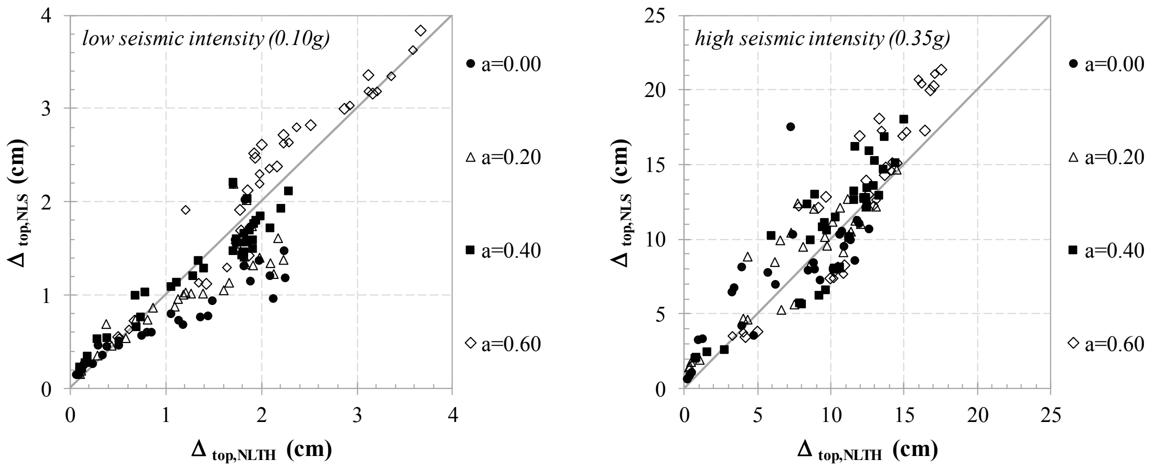


Figure 12: NLS proposed procedure vs. NLTH analyses: structures grouped against the infill opening ratios.

Further comparisons are depicted in Figure 11 and Figure 12 in which results are grouped considering the infill distribution (Type A, B and C) and the opening ratios (a from 0 to 0.60).

The significant level of accuracy of the novel procedure emerges in Figure 13, which reports the comparison of results for all the analysed structures as a whole and the two level of seismic intensity under investigation. Figure 13 also reports the value of the correlation factor $R^2=0.92$ and the distribution of the ratio between the displacement demand evaluated trough the novel procedure and the corresponding value obtained via NLTH analysis.

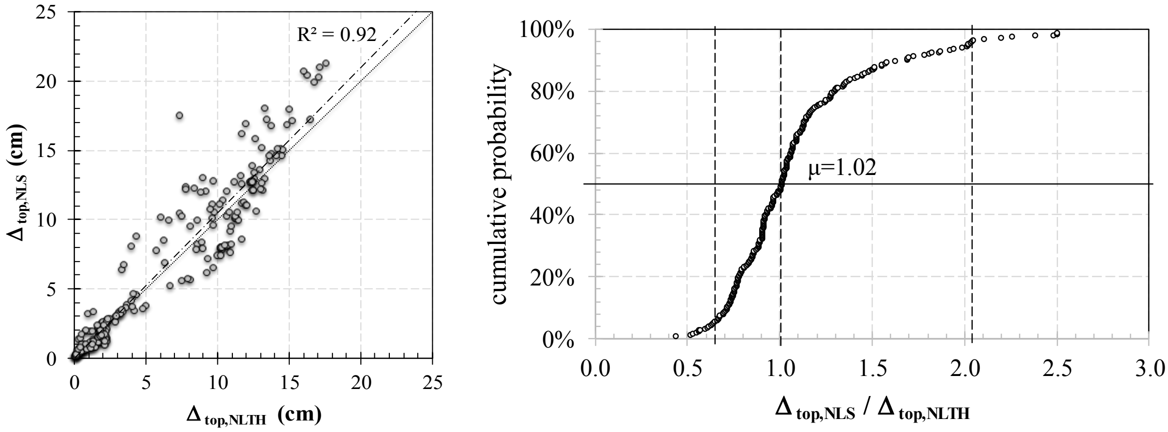


Figure 13: Accuracy of the proposed NLS procedure.

The comparisons demonstrate the very good agreement between the results estimated through the proposed procedure and the ones of NLTH analyses. The coefficient of determination R^2, originally evaluated equal to 0.934 by Martinelli et al. [9] for a database of structures smaller than the one considered in this study, is very close to the unit yet confirming the accuracy of the proposed procedure. Moreover, the median value of the displacement ratio of the entire database of structures as a whole is equal to 1,02 resulting in slightly conservative estimation of the displacement demand (median value of $\overline{\Delta_{top}^{NLS}}$ is greater than $\overline{\Delta_{top}^{NLTH}}$). This result confirms an accuracy greater than the one originally evaluated by Martinelli et al. [9] on a smaller parametric field.

Finally, it is worth to highlight that the standard deviation of the ratio $\Delta_{top}^{NLS} / \Delta_{top}^{NLTH}$ is equal to 0.49 resulting in medium-to-low values of uncertainties introduced by the novel procedure. As expected, such a value is slightly higher than the one originally evaluated by Martinelli et al. [9] as it is derived considering a parametric field of structures wider than the one originally used for calibrating the procedure.

5 CASE STUDY

The present section reports an application of the proposed procedure to an seven-story building considered as a relevant case study. Specifically, an existing masonry infilled RC structure located in Southern Italy and designed for only gravitational loads according to codes of practice in force in the 1970's.

Masonry infills are made of artificial blocks of clay forming a wall with thickness equal to 35 cm and the following mechanical properties:

- specific weight: $\gamma_w = 8.00$ kN/m^3;
- compressive strength: $f_m = 4.00$ N/mm^2;
- shear strength: $\tau_0 = 0.30$ N/mm2;
- elastic modulus: E = 3600 N/mm^2;
- shear modulus: G = 1080 N/mm^2.

The concrete cylindrical strength is equal to 20.75 MPa, while f_{sm}=220 MPa is considered for the yielding strength of steel bars. , Figure 14 represents further details, taken from the original design documents.

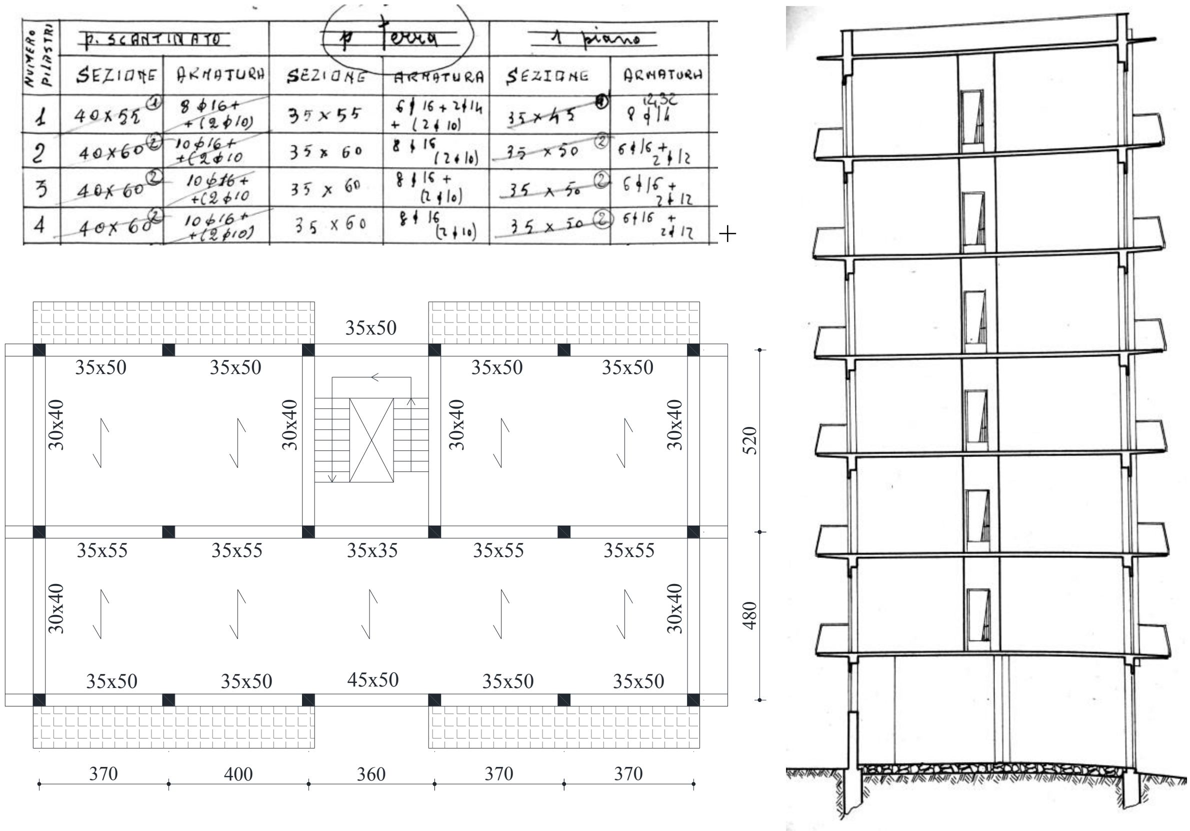


Figure 14: Details of the analyzed structure.

The structure is modelled in OpenSEES [16] with Concrete01 and Steel01 laws simulating the nonlinear behaviour of materials. Beam and column are modelled through distributed plasticity force-based elements (namely *"nonlinearbeamcolumn"*), while masonry infill walls are simulated adopting the equivalent strut model by Dolsek and Fajfar [17] described in section 3 and already used within the parametric analysis presented above. Specifically, the real distribution of openings within infills is taken into account considering the opening ratio defined in section 3. Figure 15 shows the horizontal force-displacement relationships of the masonry infill under investigation. Opening ratios equal to a=0.00 (solid wall), a=0.40 and a=0.60 are considered in order to modelling infills with increasing dimensions of windows and doors, while infills at the first storey are not included within the structural model as very large doors are present at that level. Moreover, the strain-stress relationship of the equivalent strut simulating infills is depicted in Figure 15. This relationship is modelled in OpenSEES by adopting the *Pinching04* material assigned to truss element with cross sectional area equal to $A_{truss}=1\ mm^2$.

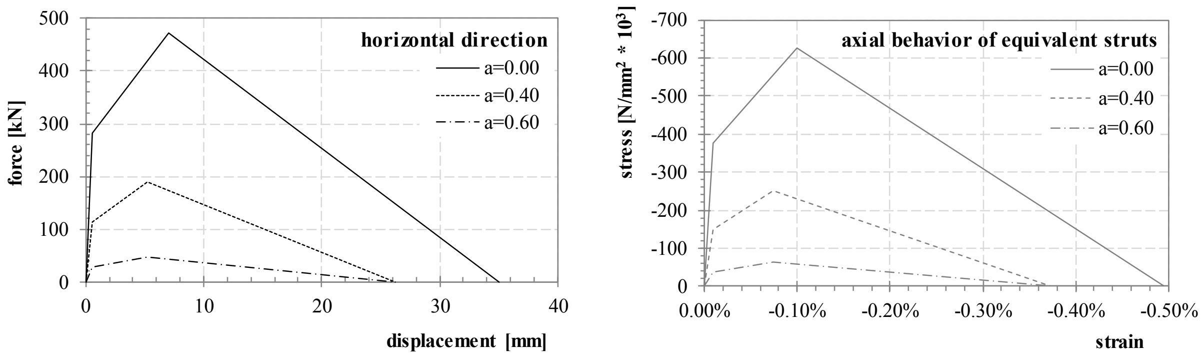


Figure 15: Nonlinear behaviour of the equivalent struts simulating masonry infill walls.

Both Nonlinear Static (NLS) and NLTH analyses are performed. According to the current Italian code [14], two levels of seismic intensity related to the Limit State of Damage Preservation (SLD) and of Life Safety (SLV) are considered. Figure 16 shows the two Linear Elastic Demand Spectra (LEDS) considered in NLS analyses along with the spectra of the two sets of seven natural accelerograms used in NLTH analyses.

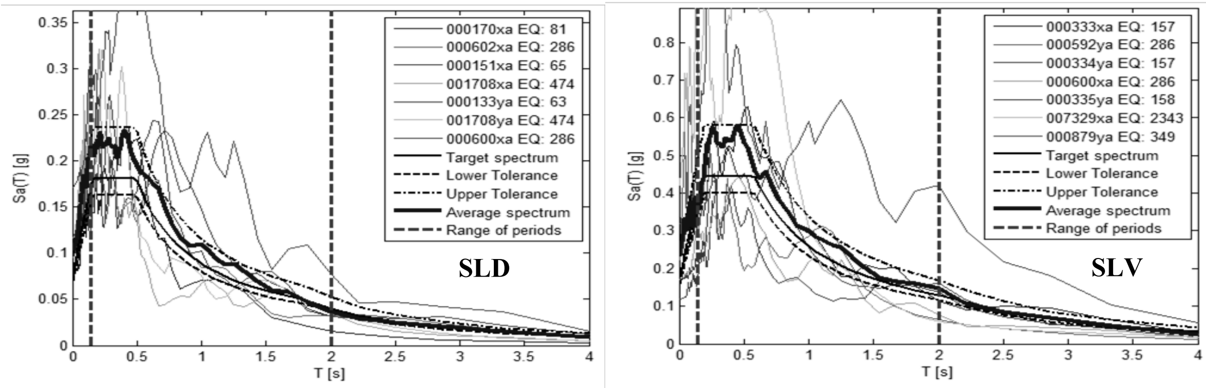


Figure 16: Spectra representing the seismic input levels of NLS and NLTH analyses.

NLS analyses are performed on the infilled structure and the corresponding bare one in the two-main directions; both "uniform" and "modal" patterns are considered. Figure 17 shows the eight capacity curves obtained for the infilled model along with the curves of the bare frame.

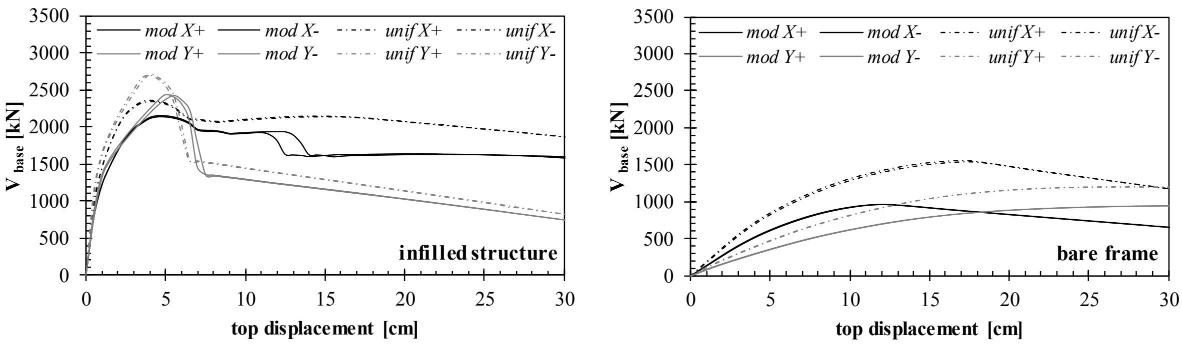


Figure 17: Capacity curves of the infilled structure and the corresponding bare one.

The response of the structure in the negative verse is quite similar to the positive one in both x- and y-direction. Moreover, the presence of infills significantly increases both stiffness and strength of the analysed model. The N2 Method is applied based on the capacity curves of the bare structure and the displacement demand $\Delta^{NLS}_{top,bare}$ is evaluated. Then, the proposed novel procedure is adopted as described in Section 2 with the aim to evaluate the top displacement demand of the structural model including masonry infills.

Moreover, NLTH analyses are also performed in order to compare the results obtained through the proposed procedure with the ones provided by the dynamic analysis. These results are reported in Table 2 and Table 3 for the Limit State SLD and SLV, respectively, along with the values of the shape parameter α and of the areas beneath the capacity curves, up to a displacement demand of the bare structure determined through the N2 Method. The displacement demands obtained via NLTH analyses are intended as average values of seven numerical simulations related to the seven natural accelerograms. Specifically, the column denoted as $\Delta^{NLS}_{top,bare}$ reports the displacement demand evaluated via N2 Method on the bare frames, while the other two columns (A^{NLS}_{bare} and A^{NLS}_{infill}) report the areas beneath the capacity curves, up to a displacement demand $\Delta_{top,bare}^{NLS}$. The shape parameter α is evaluated through eq. (2) and the column $\Delta_{top,\mathrm{infill}}^{NLS}$ reports the valued of the top displacement demand of infilled structures determined adopting the proposed procedure. Moreover, the maximum displacement demand for each direction is highlighted and compared to the one obtained via NLTH analysis.

Direction		NLS analysis: bare (N2 Method) $\Delta_{top,bare}^{NLS}$ [cm]	A_{bare}^{NLS}	NLS analysis: infilled $A_{\mathrm{infill}}^{NLS}$	α	$\Delta_{top,\mathrm{infill}}^{NLS}$ [cm]	max	NLTH analysis: bare $\Delta_{top,bare}^{NLTH}$ [cm]	NLTH analysis: infilled $\Delta_{top,\mathrm{infill}}^{NLTH}$ [cm]
X	mod X+	5.089	1.78E+05	8.72E+05	0.204	1.038	1.038	3.518	1.079
	mod X-	5.025	1.80E+05	8.76E+05	0.205	1.032			
	unif X+	4.046	1.64E+05	7.65E+05	0.214	0.865			
	unif X-	3.987	1.65E+05	6.50E+05	0.254	1.014			
Y	mod Y+	5.846	1.25E+05	1.03E+06	0.121	0.705	0.705	3.191	1.037
	mod Y-	5.846	1.24E+05	1.04E+06	0.119	0.698			
	unif Y+	4.081	9.28E+04	8.46E+05	0.110	0.448			
	unif Y-	4.081	9.24E+04	8.44E+05	0.109	0.447			

Table 2: Numerical results of the analyses for the Limit State SLD.

Direction		NLS analysis: bare (N2 Method) $\Delta_{top,bare}^{NLS}$ [cm]	A_{bare}^{NLS}	NLS analysis: infilled $A_{\mathrm{infill}}^{NLS}$	α	$\Delta_{top,\mathrm{infill}}^{NLS}$ [cm]	max	NLTH analysis: bare $\Delta_{top,bare}^{NLTH}$ [cm]	NLTH analysis: infilled $\Delta_{top,\mathrm{infill}}^{NLTH}$ [cm]
X	mod X+	15.197	1.05E+06	2.73E+06	0.383	5.827	5.827	9.256	4.014
	mod X-	15.052	1.05E+06	2.78E+06	0.378	5.691			
	unif X+	12.452	1.07E+06	2.48E+06	0.432	5.376			
	unif X-	12.343	1.08E+06	2.49E+06	0.434	5.358			
Y	mod Y+	19.481	1.07E+06	2.80E+06	0.381	7.426	7.598	10.796	3.634
	mod Y-	19.481	1.11E+06	2.84E+06	0.390	7.598			
	unif Y+	13.601	7.96E+05	2.41E+06	0.330	4.489			
	unif Y-	13.601	7.95E+05	2.40E+06	0.331	4.498			

Table 3: Numerical results of the analyses for the Limit State SLV.

Finally, Figure 18 demonstrates a good agreement between the proposed procedure and the NLTH analyses considered in this study, the former being slightly conservative at the SLV.

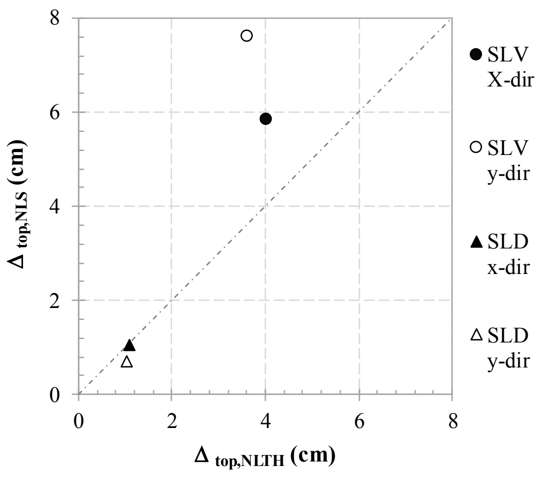


Figure 18: Comparison of the proposed procedure against NLTH analyses for the case of study.

6 CONCLUSIONS

This paper has proposed a further validation of a simplified procedure recently formulated by the Authors for simulating seismic response and determining displacement demand on masonry infilled RC frames. A wide parametric analysis has been proposed with the aim to support the proposed formulation, which is further validated on a relevant case study.

The proposed analyses point out that simplicity is the key attractive features of the proposed procedure, which prove to be also very accurate, as demonstrated in a wide number of applications covering a significant set of relevant quantities (in a total of 148 structures) and their variability in a wide range of values. Moreover, the procedure has been assessed in two different levels of seismic intensity (with PGA of 0.10g and 0.35g) corresponding to two relevant limit states in medium-seismicity regions.

Finally, further validations will be presented in the future with the aim to assess the stability of the proposed procedure with respect to several typologies of structural members and masonry types.

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