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APPLICATION OF DISPLACEMENT-BASED PROCEDURES FOR THE ASSESSMENT OF INFILLED RC FRAMES

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ABSTRACT

The purpose of this research was to propose an extension of the displacement-based approach for the assessment of the seismic response of infilled RC frames. In order to evaluate the behaviour of these structures, it was fundamental to define an appropriate analytical model. In this study the equivalent strut model for monotonic loading proposed by Al-Chaar was considered. This model was extended to the case of cyclic loading by calibrating the degradation of strength and stiffness, the residual strength and the loading and unloading branches through comparisons with available experimental results. The model was then applied for investigating the seismic response of infilled RC frames: pushover and nonlinear dynamic analyses were carried out for obtaining the response in terms of base shear-top displacement and for estimating the configuration of collapse. On the basis of the results of these analyses, a displacement-based method for the assessment of infilled frames was proposed. The method is based in particular on simplified criteria for the estimate of the equivalent damping and of the collapse displacement profile of infilled RC frames. The effectiveness of the method was verified through comparison between the displacement demand estimated with the proposed procedure and the one obtained from nonlinear dynamic analyses.

INTRODUCTION

Many existing buildings, such as those in Italy, are built using RC frames with masonry infill. Various studies (Klingner and Bertero, 1978; Comité Euro-International Du Béton, 1996; Al Chaar, 2002; Cavaleri et al., 2003; Cavaleri et al., 2005; Crisafulli, 2007) showed how the presence of masonry infills may alter the overall behaviour of structures when subjected to seismic action. The presence of the masonry infills can produce positive or negative effects (Asteris, 2003; El-Dakhakhni et al., 2003; Korkmaz et al., 2007). In the first case it can increase the strength and the dissipative capacity of the structure, in the second case it can produce unexpected distributions of forces and consequent local phenomena of collapse. However, the most widespread and established practice of design does not consider the infills as structural elements, ignoring their strength and stiffness. For this reason, in the analytical models these elements are often neglected. The first step of this study was to evaluate how the presence of infills could modify the structural response of the frames. To this purpose it was first necessary to calibrate an analytical reliable model which could take into account the presence of the infills (Al Chaar, 2002; Cavaleri et al., 2005). The calibrated model was then used to perform several

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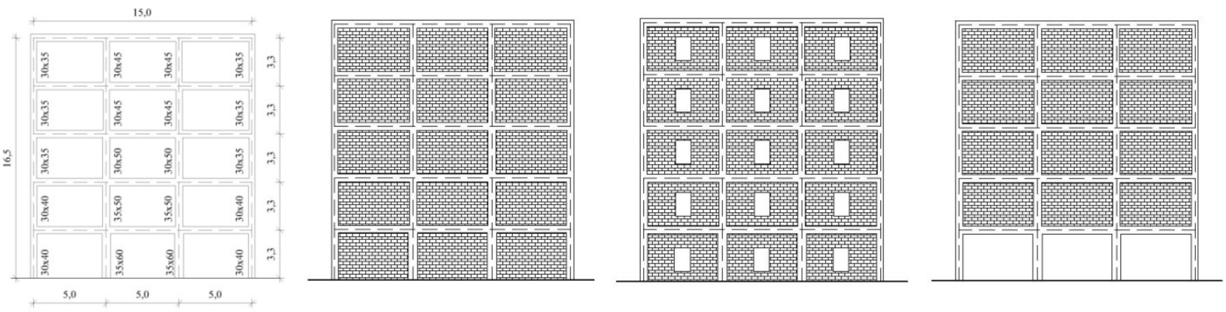
nonlinear dynamic and pushover analyses (Landi et al., 2013). The objective of these analyses was to determine the response in terms of base shear-top displacement and to investigate the configuration of collapse. In particular the lateral displacement profile at collapse and the plastic hinge distribution were examined. On the basis of the obtained results, a displacement-based method (Priestley et al, 2007) was proposed for the prediction of the seismic demand of infilled RC frames. The method is based also on simplified criteria proposed recently by the authors (Landi et al., 2012) for the estimate of the equivalent damping for infilled RC frames. This method was applied to the considered case studies and its effectiveness was tested through comparisons with nonlinear dynamic analyses.

NONLINEAR SEISMIC ANLYSIS OF INFILLED RC FRAMES

The analyses were performed on bare and infilled RC frames in order to evaluate how the masonry panel could modify the collapse mechanism. The bare frame being examined is a five-storey symmetrical structure designed only for gravity loads. The dimensions determined for the beams are: width equal to 300 mm and depth equal to 500 mm. The cross-sections of the columns are rectangular with different dimensions at the external and internal location at the different storeys (Fig. 1).



(a) (b) (c) (d)

Figure 1. Examined frames: bare frame (a), totally infilled frame (b), totally infilled frame with openings (d) and infilled frame without panels at the ground floor (d).

The assumed mechanical properties of materials are: concrete cylinder strength f_{ck} equal to 25 MPa and steel yield strength f_{yk} equal to 430 MPa. With reference to the beams and columns, the infilled frame was assumed to be identical to the bare frame. Two types of masonry were examined: the masonry of the infilled frame VAR has a compressive strength of 4.1 N/mm^2 and shear strength of 0.3 N/mm^2; the masonry of the infilled frame UNIF has a compressive strength of 3.3 N/mm^2 and shear strength of 0.2 N/mm^2. Furthermore, the infilled frame VAR has masonry panels of variable thicknesses between 240 mm and 150 mm; while, for the infilled frame UNIF, masonry panels are all of the same thickness of 150 mm. Three cases were examined for both types of infilled frames (Fig. 1): totally infilled frame, infilled frame with square openings of 1500 mm in all the panels, infilled frame without panels at the ground floor.

The nonlinear analyses were carried out in order to assess the performance of the structures under study. The OpenSees software (McKenna and Fenves, 2005) was used for the nonlinear analyses. The elements of the examined structures were modelled with a single finite element for each beam or column. For each element five control sections were adopted. A bilinear stress-strain relationship with hardening ratio equal to 0.005 was assumed for the steel fibres. A constitutive law, which includes the effect of confinement due to stirrup and the stiffness degradation due to cyclic loading, was considered for the concrete. Different types of behaviour were adopted for the cover concrete and the concrete core. The infills were modelled by replacing the panel with a system of two equivalent struts. Each masonry strut was considered to be effective only in compression. The width and the strength of the strut were determined according to the model proposed by Al-Chaar (2002). The constitutive law assigned to the strut accounts for the degradation of stiffness and strength typical of the masonry (Fig. 2). The response for cyclic loading was studied as suggested by Cavaleri et al. (2005). More details about the model and its calibration through comparisons with experimental results can be found in Landi et al. (2013).

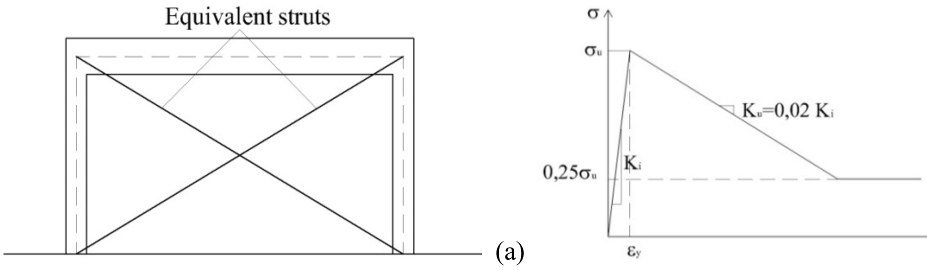


Figure 2. Equivalent diagonal struts (a) and stress-strain relationship assigned to the struts (b).

The objective of the analyses was to determine the response of the structures in terms of base shear-top displacement and to evaluate their displacement of collapse. At this displacement the configuration of collapse was examined by observing the distribution of plastic hinges and the displacement profile. The collapse, in this study, was defined according to three criteria: collapse “a” is the achieving of the ultimate strain in confined concrete of columns, “b” is the achieving of the ultimate interstorey displacement, “c” is the achieving of the ultimate shear strength. The second type of collapse was defined by the maximum interstorey displacement which was determined for each floor. This displacement was determined using the ultimate drift multiplied by the storey height. The ultimate drift was calculated as the sum of the yield (θ_y) and the plastic (θ_p) ones. The yield drift was calculated with the following relation proposed in literature (Priestley et al., 2007):

$$\theta_y = 0,5\varepsilon_y \cdot \frac{L_b}{h_b} \tag{1}$$

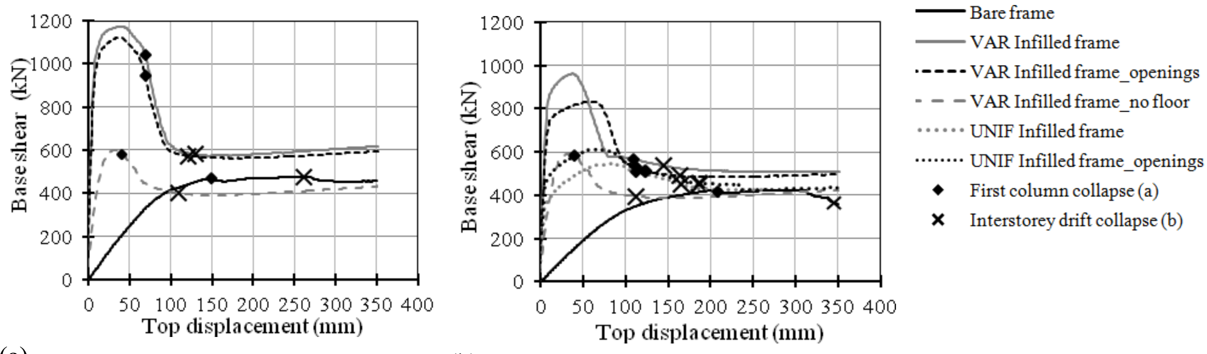
where L_b and h_b are beam length and section depth. The plastic drift θ_p was determined from the ultimate ϕ_{ls} and the yield ϕ_y curvatures and from the plastic hinge length l_p:

$$\theta_p = \left(\phi_{ls} - \phi_y\right) L_p \tag{2}$$

Once the ultimate drift was determined, the maximum displacement was calculated. The pushover analyses were carried out using two set of forces, the first with forces proportional to the masses multiplied by the first modal deformations, and the second with forces proportional to the masses. The displacements associated to the collapse conditions were identified for each analysis. In correspondence of these displacements, the plastic hinges were observed and the displacement profile represented. The nonlinear dynamic analyses were performed using three accelerograms (here S1, S2, S3): each ground motion was applied with increasing values of intensity until the failure of the structure was reached (incremental dynamic analyses). Once a point was determined for each dynamic analysis, it was possible to derive a base shear-top displacement curve. Then, for each ground motion, it was possible to determine the intensity that caused the first occurrence of each collapse condition. At the collapse condition, the distribution of plastic hinges and the maximum displacement profile were evaluated. The nonlinear dynamic analyses were carried out only for the bare frame and for the totally infilled frame.

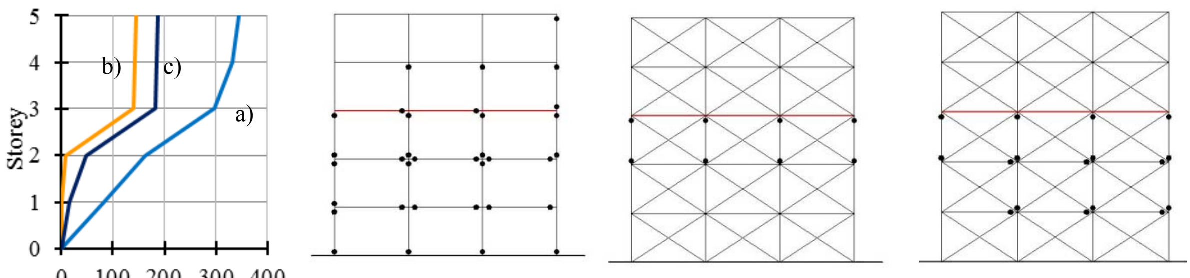
The collapse “b” is the most significant, as usually it is associated with a column-sway mechanism. We noted that collapse “a” was reached for smaller displacements than collapse “b” in all the analyses we carried out, as is shown in Fig. 3. Collapse “c”, associated with achieving the ultimate shear strength, never occurred.

The results of the pushover analyses are shown in Fig. 3. In general, these results show that the contribution of the infill, in terms of base shear, was significant only for a displacement lower than about 75 mm. For larger displacements, the infills collapsed and their strength was significantly reduced. The collapse of the infilled frames always occurred at displacements smaller than those of the bare frame. However, for a given distribution of forces, the mechanism of collapse of the infilled and the bare frame always occurred at the same storey. Furthermore, the results relative to the infilled frame show that the presence of openings did not substantially modify the response of the frame. The openings simply reduced the lateral strength of the structure. The absence of infills at the ground floor produced, as expected, a collapse mechanism at the first storey. We could finally see that the presence of infill did not change the mechanism of collapse, but affected the displacement profile. In the examined cases, the collapse occurred at the first or the third floor for both the bare and the infilled frames, while the displacement profile for the infilled frames had lower values than that of the bare frame **(**Fig. 4). This was linked with the plastic hinges which, in the infilled frames, were concentrated at the storey of collapse. With this distribution of plastic hinges, the collapse displacement was close to the ultimate displacement of the single storey, without significant effects of the deformation of the other storeys.



(a) (b)

Figure 3**.** Pushover analyses: force proportional to masses (a) or to first mode deformations (b).



(a) Bare frame (b) Totally infilled frame VAR (c) Totally infilled frame UNIF

Figure 4. Displacement profile and distribution of plastic hinges under collapse condition “b” (achievement of ultimate inter-storey drift) in the case of force proportional to the first mode.

In the results obtained from the nonlinear dynamic analyses (Fig. 5), the infilled frames had greater strength but a lower collapse displacement than the bare frames. This is similar to the result obtained in the pushover analyses. However, in the nonlinear dynamic analyses, there was a more widely spread distribution of plastic hinges for the infilled frames at all storeys than for the pushover analyses. For such frames, the top displacement was influenced not only by the inter-storey displacement where the collapse occurred, but also by the inter-storey displacements of other two storeys. As a consequence of the observed distribution of plastic hinges, the displacement profiles of the infilled frames were found to be more similar to the profile of the bare frame than the results of the pushover analyses. Fig. 6 shows, for example, the results relative to accelerogram S1 being applied.

Using incremental dynamic analyses, it was possible to evaluate the peak ground acceleration (PGA) at the collapse. In Table 1, the values of the peak ground acceleration are illustrated for the different cases examined, referring to the collapse “b”. In particular, the table shows the average values of the three accelerograms. For all cases, the seismic input necessary to reach the collapse of the infilled frame resulted to be larger than that required for the bare frame. It could be seen that the average values relative to the infilled frame are about 40% greater than those relative to the bare frame. This result is related to the reduction of the displacement demand for the infilled frames when compared to that for the bare frame.

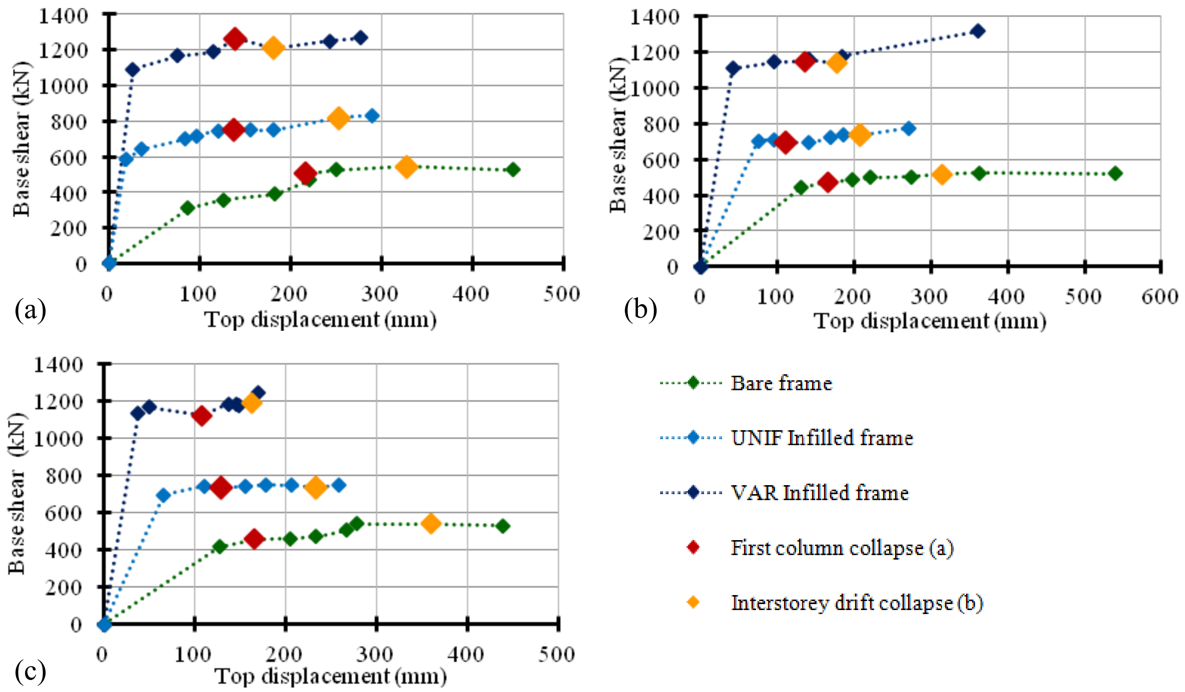
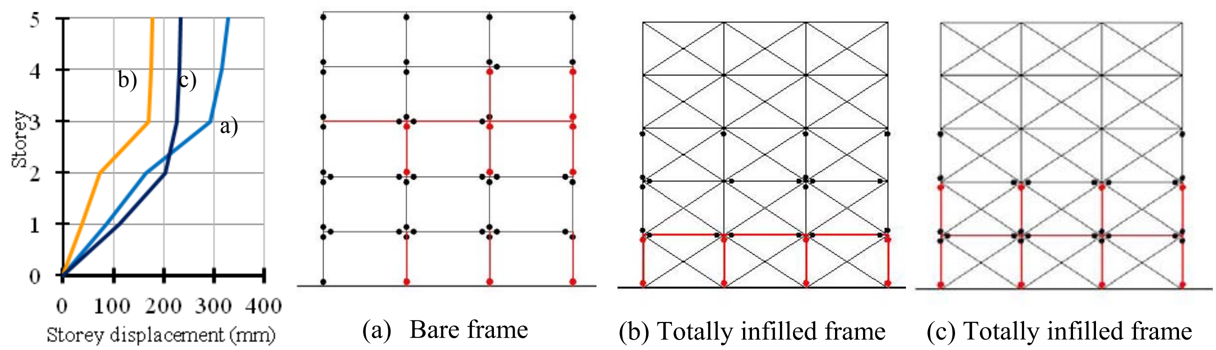


Figure 5. Top displacement-base shear curves from incremental dynamic analyses: S1 (a) S2 (b) S3 (c).



(a) Bare frame

(b) Totally infilled frame VAR

(c) Totally infilled frame UNIF

Figure 6. Displacement profiles and distribution of plastic hinges under collapse condition “b” for the ground motion S1.

Table 1. Values of peak ground accelerations at collapse “b" and relative average values

	Bare frame	Infilled frame VAR	Infilled frame UNIF
S1	0,544g	0,748g	0,952g
S2	0,502g	0,67g	0,737g
S3	0,585g	0.715g	0,78g
$a_{average}$	0,544g	0,711g	0,823g

PROPOSAL OF A DISPLACEMENT-BASED ASSESSMENT METHOD FOR INFILLED RC FRAMES

This section illustrates a proposal for an extension of the displacement-based assessment (DBA) procedure (Priestley et al., 2007) to infilled RC frames. As it is known, according to the displacement-based approach, a general MDOF structure is schematized by an equivalent linear SDOF system defined by the secant stiffness at maximum displacement and by equivalent damping. The ductility-damping laws derived in previous study of the authors for infilled frames (Landi et al., 2012) were implemented in the procedure together with a proposed method to predict the collapse mechanism and the displacement profile. The procedure was then applied to the multi-storey frames described previously, and its effectiveness was tested through comparisons with the results of nonlinear dynamic analyses. The DBA procedure extended to infilled RC frames is summarized in the steps reported in the following: each step is described with reference to the study cases previously described.

1. Assess the moment-curvature response of the critical sections of the structural elements.
2. Determine the plastic rotation capacity of the plastic hinges (Eq. (2)).
3. Determine the expected inelastic mechanism. This mechanism can be defined on the basis of simple evaluations of particular indices or, alternatively, on the basis of the results of pushover analyses. In this work, the inelastic mechanism resulting from pushover analyses was assumed. With reference to the study cases previously presented, it was possible to obtain a column-sway mechanism at the third floor for both the bare and infilled frame (Fig. 4).
4. Define the limit-state displacement profile. For the bare frame, the limit-state displacement profile can be defined, as proposed in literature (Priestley et al., 2007), by adding yield displacement to the plastic displacement. The storey displacement at yield is assumed to be linear along the height, and can be determined as the product of the yield rotation (Eq. (2)) with the height. Assuming a column sway-mechanism at the floor at which the collapse occurs (level *i*), the plastic displacement may be calculated as:

$$\Delta_P = \theta_P \cdot H_{i-1i} \tag{3}$$

where θ_P is the lower of the plastic rotation capacity of the columns at level *i* and $H_{i\text{-}1\mathrm{i}}$ is the inter-storey height between levels *i* and *i*-1. The limit-state displacement is thus obtained by adding the yield displacement to the plastic displacement at the levels above the one in which the collapse occurs. Below this level, the limit-state displacement corresponds to the yield displacement. Regarding the infilled frame, a new method was adopted for this study. In particular, the method is based on defining the values of yield displacements of the infilled frame, as a fraction of the yield displacements of the corresponding bare frame and is based on the assumption that the collapse mechanism of the infilled frame is similar to that of the bare frame. This method was then studied and applied to the study cases presented previously. Initially, the displacement profile obtained from pushover analyses, for both bare and infilled frames, was observed. In particular, the displacement profile observed at the instant before the column-sway mechanism occurred was assumed to be the yield displacement profile. The displacement profiles obtained from pushover analyses are the ones represented by dotted lines in Fig. 7.

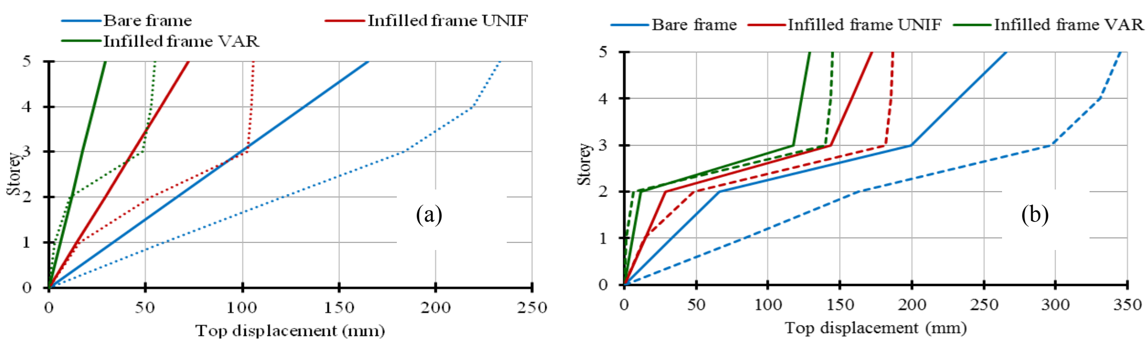


Figure 7. a) Yield displacement profile from pushover analyses (dot lines) and from linearization (continuous lines). b) Limite-state displacement profile from pushover analyses (dot lines) and from proposed procedure (continuos lines).

In order to linearize the displacement profile, a ratio was calculated between the displacement from pushover analysis of the infilled frame and the displacement of the bare frame, for each floor. The mean of the values obtained for these ratios at each level was then applied to the values of displacement obtained on the basis of Eq. (1). A mean ratio equal to 0.18 was derived for the infilled frame VAR, while a value equal to 0.44 was obtained for the infilled frame UNIF. The linear displacement profile determined in this way was assumed to be the yield displacement of the infilled frame (continuous lines in Fig. 7a). In terms of the limit-state displacement profile, the procedure used was the same used for the bare frame, and the yield displacement was added to the plastic displacement at the levels above the one at which the collapse occurred. In the examined cases, the collapse occurred at the third floor for both bare and infilled frames. Fig. 7b shows the comparison between pushover analyses and the proposed procedure.

5. Determine the base-shear capacity. As proposed in literature (Priestley et al., 2007), if a column-sway mechanism is predicted, the base shear can be determined from the column shears at the storey where the collapse occurs. With reference to the study cases, considering a column sway mechanism occurring at the third storey, the total shear at this storey was calculated as:

$$V_{3rd} = \sum_{j=1}^{m} \left(M_{Cj,b} + M_{Cj,t} \right) \Big/ H_{23} \tag{4}$$

where $M_{Cj,b}$ and $M_{Cj,t}$ are the column moment capacities at the bottom and top of the columns and H_{23} is the inter-storey height of the third storey. In order to determine the base shear, it was necessary to determine the loading factor λ at the collapse condition:

$$\lambda = V_{3rd} \Big/ \sum_{i=3}^{5} F_i \tag{5}$$

where F_i is the force applied at the storey i in a generic step of the pushover analysis. Once λ was determined, it was possible to calculate the distribution of forces $F_{i\lambda}=\lambda \cdot F_i$ applied at the collapse condition and the corresponding base shear V_{base}:

$$V_{base} = \sum_{i=1}^{5} F_{i\lambda} \tag{6}$$

In the case of an infilled frame, the total shear at the collapsed storey was determined by also including the shear strength of the infill panels V_{inf}:

$$V_{3rd} = \sum_{j=1}^{m} \left(M_{Cj,b} + M_{Cj,t} \right) \Big/ H_{23} + V_{inf} \tag{7}$$

6. Define the equivalent SDOF system. The parameters that define the equivalent SDOF system are the equivalent mass m_e and effective height H_e calculated as suggested in (Priestley et al., 2007). The yield displacement Δ_y of the equivalent SDOF system can be derived as:

$$\Delta_y = \theta_y \cdot H_e \tag{8}$$

7. Determination of the displacement demand for the equivalent SDOF system ($\Delta_{dem,\mathrm{SDOF}}$): this calculation can be performed following the iterative procedure described in (Priestley et al., 2007). In this step the μ-ξ relationship was defined accounting for the effects of the infills using the laws derived in previous studies by the authors (Landi et al., 2012) with reference to a set of spectrum-compatible ground motions that include the three ground motions here considered (S1, S2 and S3). In the assessment of the bare frame it was assumed the law of Eq. (9). Two method were considered for the assessment of the infilled frame. In the first method the assessment was performed considering the base shear strength and the stiffness of the bare frame, while in the second method the assessment was performed considering the base shear strength and the stiffness of the infilled frame. In the assessment of the infilled frame based on the stiffness of the

bare frame $K_{bare,}$ it was assumed the relationship of Eq. (10). In the assessment of the infilled frame based on the stiffness of the infilled frame $K_{inf,}$ it was assumed the relationship of Eq. (11). Furthermore the law of Eq. (12) was tested. This law is similar to Eq. (11), but for large values of ductility tends to the same values of the bare frame, thus neglecting any residual strength of the infill panel.

$$\xi_{hyst} = 0,794\left(\frac{\mu - 1}{\mu\pi}\right) \tag{9}$$

$$\xi_{hyst} = 0,804\left(\frac{\mu + 0,83}{\mu\pi}\right),\ \mu \geq 1 \tag{10}$$

$$\xi_{hyst} = 0,83\left(\frac{\mu - 0,07}{\mu\pi}\right),\ \mu \geq 0 \tag{11}$$

$$\xi_{hyst} = 0,794\left(\frac{\mu - 0,07}{\mu\pi}\right),\ \mu \geq 0 \tag{12}$$

8. Transform the displacement demand for the SDOF structure in the displacement demand for the MDOF structure. The effectiveness of the method was verified by comparing the displacement demand obtained from nonlinear dynamic analyses for the study cases with the displacement obtained in the assessment procedure. Once the assessment of the SDOF structure was carried out, the displacement demand $\Delta_{dem,\mathrm{SDOF}}$ was determined. It was then necessary to determine the displacement demand at the top of the five storey frame being considered. The relation between the displacement demand for the equivalent SDOF structure and the displacements at the different levels $\Delta_{dem,i}$ can be written as:

$$\Delta_{dem,\mathrm{SDOF}} = \frac{\sum_i m_i \Delta^2_{dem,i}}{\sum_i m_i \Delta_{dem,i}} \tag{13}$$

In this case, the unknown terms are the displacements of the storeys. However, in the nonlinear range, considering a prefixed column-sway mechanism, all these displacements depend upon a single parameter, the plastic rotation of the yielded columns. Therefore, an inverse expression of Eq. (13) was used to determine the plastic rotation associated to the displacement demand. By knowing this rotation, it was then possible to derive the displacement demand at each storey.

VERIFICATION OF THE PROCEDURE

The procedure above was applied to the selected bare and infilled frames. Regarding the infilled frames, different cases were examined. In particular, two different seismic intensities were analysed, one with peak ground acceleration (PGA) equal to 0.53 g and the other with PGA equal to 0.74 g. The two cases of stiffness previously mentioned were also examined. In the first case, the stiffness of the infill panels was neglected and the base shear was obtained considering only the RC frames (K_{bare}). In the second case, the stiffness of the infills was considered and the base shear was obtained, which also included the shear strength of the infill panels (K_{inf}). In this last case, the shear strength of the infill panels was evaluated in two different ways: considering peak strength (V_{max}) or considering residual strength, equal to 25% of the peak strength ($V_{25\%}$). All the analyses were repeated for the two different infilled frames denominated VAR and UNIF.

At the end of the analyses, for each case being examined, it was possible to compare the displacement obtained with the DBA procedure with the displacement obtained from nonlinear dynamic analyses. Table 2 shows all the results in terms of top displacement, together with the corresponding ductility and damping values. In Fig. 8, the results of DBA and the ones of nonlinear dynamic analyses are shown superimposed on the pushover curves.

It is possible to note that the estimates obtained with the DBA procedure were close to the results of the nonlinear dynamic analyses, except when V_{max} was considered. The agreement between the proposed DBA and nonlinear dynamic analyses was very good, especially when K_{inf} and $V_{25\%}$ were considered. In these cases, the estimate obtained with DBA was almost always conservative. The estimates obtained considering V_{max} were not very close to the results of the nonlinear dynamic analyses, since the responses of the infilled frames, for the considered seismic intensities, largely over-passed the peak point. The estimates obtained when considering the stiffness of the bare frame for the infilled frames were not conservative, but they were, however, quite good and not overly different to the results of nonlinear dynamic analyses, especially for PGA=0.74 g. When the stiffness of the infilled frame was assumed in the assessment, DBA was repeated for comparison, first applying the law in Eq. (11) (C_{inf}), and then the law in Eq. (12) (C_{bare}). The difference between the results obtained using Eqs. 11 or 12 was not particularly significant. With Eq. (12), the results of DBA were more conservative than with Eq. (11).

Table 2. Comparison between DBA and nonlinear dynamic analyses (DYN)

	Analysis case	**PGA = 0,53g**					**PGA = 0,74 g**				
		μ	ξ_e	DBA (mm)	DYN (mm)	$\Delta_{DBA}/\Delta_{DYN}$	μ	ξ_e	DBA (mm)	DYN (mm)	$\Delta_{DBA}/\Delta_{DYN}$
Bare	K_{bare}	2,32	19%	340	322	106%	-	-	-	-	-
Infilled VAR	K_{bare}	0,53	52%	79	115	69%	1,12	49%	151	177	85%
	K_{inf}, V_{max}, C_{inf}	0,20	22%	35	115	30%	0,78	29%	109	177	62%
	K_{inf}, V_{max}, C_{bare}	0,21	22%	37	115	32%	0,82	28%	114	177	64%
	K_{inf}, $V_{25\%}$, C_{inf}	0,75	29%	105	115	91%	1,45	30%	193	177	109%
	K_{inf}, $V_{25\%}$, C_{bare}	0,91	23%	126	115	110%	1,51	29%	201	177	114%
Infilled UNIF	K_{bare}	0,60	52%	98	130	75%	1,28	47%	185	202	92%
	K_{inf}, V_{max}, C_{inf}	0,53	28%	88	130	68%	0,96	30%	145	202	72%
	K_{inf}, V_{max}, C_{bare}	0,56	27%	92	130	71%	1	29%	149	202	74%
	K_{inf}, $V_{25\%}$, C_{inf}	0,95	29%	143	130	110%	1,28	47%	231	202	114%
	K_{inf}, $V_{25\%}$, C_{bare}	1,19	24%	173	130	133%	1,66	30%	237	202	117%

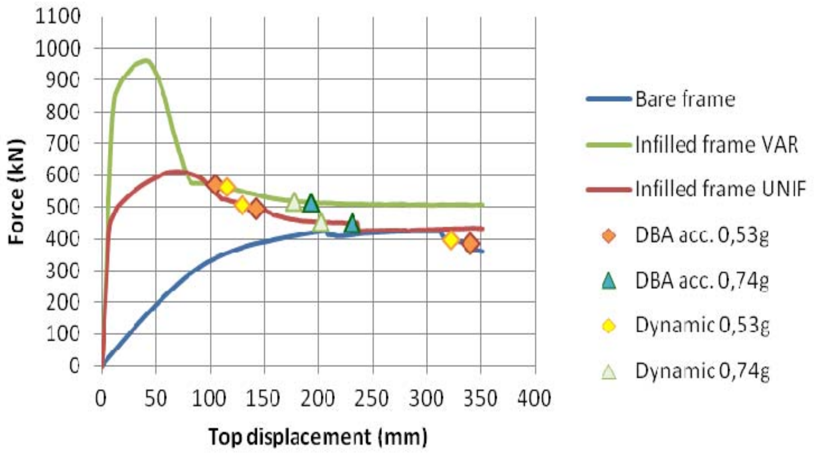


Figure 8. Representation of the results in the pushover curves.

CONCLUSIONS

The initial aim was to determine the response of the infilled frames in terms of base shear-top displacement curve and to evaluate the limit-state displacement profile. The distribution of plastic hinges and the displacement profile were analysed under collapse conditions. In this way, it was possible to determine the type of collapse mechanism. The results obtained show that the presence of the infills increased the response significantly in terms of strength and stiffness, while it reduced deformation capacity. All the analyses showed a collapse of the infilled frame at displacements smaller than those of the bare frames. Furthermore, the larger stiffness and strength of the infilled structures caused a reduction of the displacement demand. For this reason, the infilled frames were able to sustain larger peak ground accelerations than the bare frames.

On the basis of these results, a simplified method was proposed to predict the displacement profile of infilled frames under yield and collapse conditions. This method, together with the ductility-damping laws obtained in previous studies, was implemented in the displacement-based assessment procedure, and subsequently applied to the examined cases. In verifying the proposed DBA for infilled frames, it showed a good agreement with the results of nonlinear dynamic analyses.

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