Calibration of an Equivalent Strut Model for the Nonlinear Seismic Analysis of Infilled RC Frames

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SUMMARY
The objective of this paper was to simulate the seismic response of RC infilled frames. In order to evaluate the behaviour of these structures, it was fundamental to define an appropriate analytical model. Several models were proposed in literature for the masonry infills. In this study the equivalent strut model was considered. The study started from the model for monotonic loading proposed by Al-Chaar. 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. This calibration was performed by comparing the analytical results with the ones of available experimental tests on infilled frame. 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 to assess the configuration of collapse.

Keywords: Infilled Frames; Equivalent Strut Model; Non Linear Dynamic Analysis

1. INTRODUCTION

The Italian existing buildings are characterized by numerous examples of RC frames with masonry infills. Various studies (Bertoldi et al., 1993; Comite Euro-International Du Beton, 1996) 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. 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 aim of this study was to evaluate how the presence of infills could modify the structural response of the frames. To this purpose it was necessary to calibrate an analytical reliable model which could take into account the presence of the infills. (Al-Chaar, 2002). The calibrated model was then used to perform various dynamic and pushover analyses. The objective of these analyses was to determine the response in terms of base shear-top displacement and to assess the configuration of collapse. The deformed configuration and the plastic hinge distribution were examined.

2. MODELS FOR THE INFILL PANEL

The first problem was to identify a reliable and simple model which could represent the masonry infill. Many difficulties were due to the intrinsic characteristics of masonry. As it is a non homogeneous and anisotropic material, it is difficult to find a generally valid constitutive law. Furthermore the masonry shows significant degradation of stiffness and strength under cyclic loading. In the seismic design it is necessary to take into account all these aspects but often the constitutive laws proposed in literature have a limited validity. Among the models proposed in literature, the most used is the equivalent strut (Fig. 2.1). In this model the infill is replaced by a single compression element. The choice of the diagonal strut derived from the modes of failure of the infills. The diagonal, in fact, is the only portion of the panel that continues to sustain stresses also after the detachment. The characteristics of the strut (width, stiffness and strength) depend on the geometrical and mechanical properties of the masonry. The method used to calculate these characteristics varies according to the author of the model. Models for monotonic or cyclic loads were proposed in literature. In the first case, the infill is replaced by a single strut with bilinear not degrading constitutive law. In the second case, instead, two struts are considered, so that each one responds to only one way of loading. In this last case the constitutive law takes into account the strength and stiffness deterioration due to loading and unloading cycles.

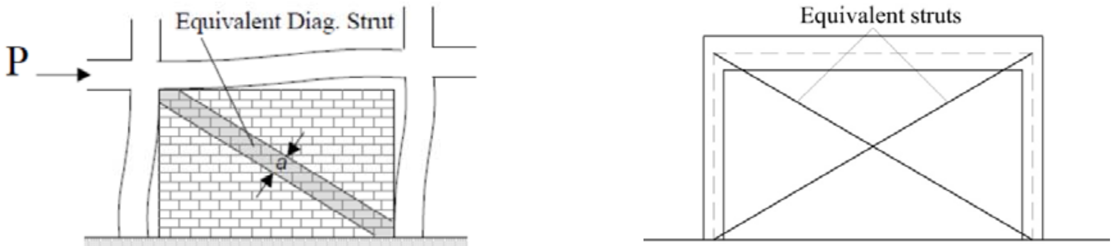

Figure 2.1. Equivalent diagonal struts

As models for monotonic loading it is possible to mention the ones proposed by Bertoldi et al. (1993) and by Al-Chaar (2002). As models for cyclic loading it is possible to mention the ones proposed by Klingner and Bertero (1978), by Cavaleri et al. (2005) and by Decanini et al. (2004). The calibration of the analytical model, which was carried out in this study, started from the models of Al-Chaar (2002) for monotonic loading and of Cavaleri et al. (2005) for cyclic loading.

2.1. Al-Chaar Model For Monotonic Loading

Al-Chaar used the dimensionless parameter λh in order to consider all the factors that determine how the infills affect the overall response of the frame; this parameter was introduced by Stefford-Smith (1961) for the first time and is expressed as follows:

$$\lambda h = \sqrt[4]{\frac{E_m t sen(2\theta)}{4 E_c I_p h}}\, h_p \tag{2.1}$$

where E_c and E_m are the modulus of elasticity respectively of concrete and masonry, I_p is the moment of inertia of the column, t is the thickness of the infill, θ is the inclination angle of the diagonal strut, h is the infill height, h_p is the height between beams axis. Al-Chaar referred to the expression proposed by Mainstone (1971) for determining the width w of the strut:

$$w = 0{,}175 D (\lambda h)^{0{,}4} \tag{2.2}$$

where D is the length of the strut. For masonry panels with openings it was introduced a reduction factor R_I for the width w. This factor depends on the ratio between the size of the openings and that of the infill. Through experimental tests Al-Chaar noted that Eqn. 2.2 lead to underestimate the actual stiffness of the infill. For this reason he suggested to use a modified modulus of elasticity for the masonry equal to $3E_m$. Finally, in order to determine the strength of the strut, he considered two modes of failure: compression and shear. The strength was set equal to the lesser of the compressive strength R_{cr} and the shear strength of the infill R_{shear}:

$$R_{u,ass} = min\left\{R_{cr} = wt\sigma_{m0}; \frac{R_{shear}}{cos\theta} = \frac{lt\tau_{m0}}{cos\theta}\right\} \tag{2.3}$$

where σ_{m0} e τ_{m0} are respectively the compressive and the shear strength of masonry.

2.2. Cavaleri-Fossetti-Papia Model For Cyclic Loading

Concerning the model for the cyclic behaviour of the infill, these authors started from the model proposed by Klingner and Bertero (1978): they observed some limits to the aforementioned model and proposed some changes in order to obtain a more general validity. With reference to the cyclic constitutive law in terms of axial force-deformation, the common features of the two models are:

- first linear elastic loading curve which continues until the panel reaches the total cracking
- strength envelope curve which accounts for the strength and stiffness degradation due to cracking; this curve considers also the residual strength of the infill after the cracking
- no tensile strength
- reloading branches with reduced stiffness compared to the elastic one
- unloading branches with slope equal to the linear elastic loading curve

In particular the modifications introduced by Cavaleri, et al. (2005) are:

- double slope of the unloading branch before the restoring force vanishes
- new loading branch characterized by a zero value of the restoring force before the system begins to exhibit non zero stiffness
- envelope strength curve for the strut characterized by exponential degrading, according to the experimental results

The model suggested by Cavaleri, et al. (2005) is shown in Fig. 2.2. The characteristics of the model are defined by a series of parameters, which depend on experimental results.

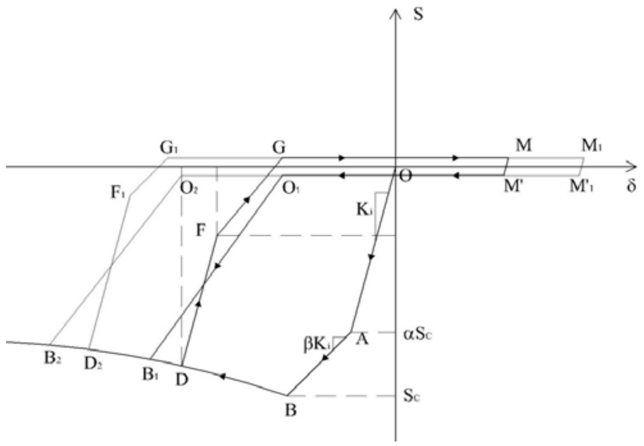

Figure 2.2. Cyclic behaviour of the strut according to the Cavaleri et al. (2005) model

3. CALIBRATION OF THE ANALTYICAL MODEL

3.1. RC frame members

In this study a distributed plasticity model with fibre sections, implemented in OpenSees (McKenna and Fenves, 2005), was adopted. The elements of the RC frames were modelled by a single finite element for each beam or column. For each element 5 control sections were adopted. These sections were modelled through a fibre discretization characterized by a width of the single fibre equal to 1 cm. A bilinear stress-strain relationship with hardening ratio equal to 0.005 and Baushinger effect was assumed for the steel fibres. A constitutive law which includes the stiffness and strength degradation due to cyclic loading was considered for the concrete. Different types of behaviour were adopted for the cover concrete and the concrete core: in the first case the effect of confinement was neglected, in the second case it was included according to the model proposed by Mander et al. (1988). In order to validate the model two comparisons with experimental results were carried out (Fig. 3.1). The first comparison regarded the moment-curvature response of a section and the test carried out by Kent (1969). The second comparison regarded the force-displacement response of a cantilever and the test carried out by Ma et al. (1976).

In the modelling of all the tested frames an element was added at the top of the columns in order to account for possible shear failure in the column due to the interaction with infill. This is a zero length element with shear strength equal to the column one.

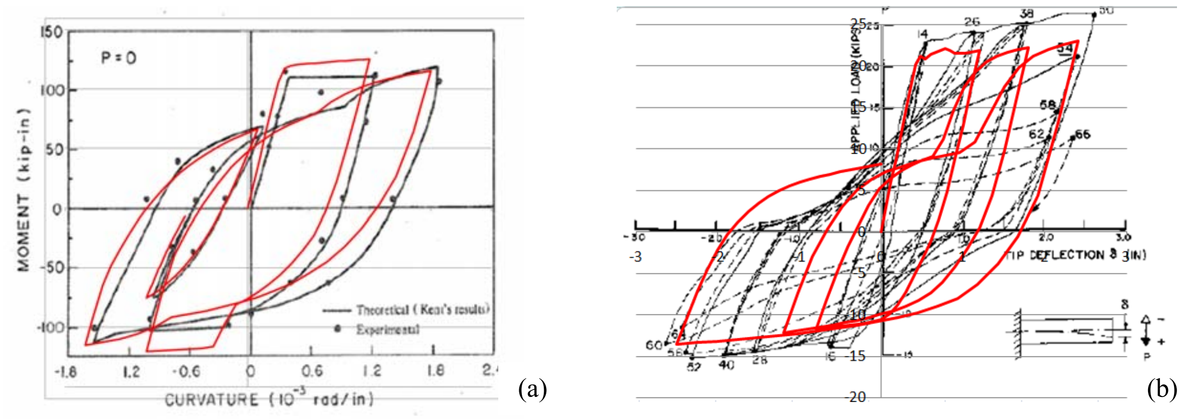

Figure 3.1. Analytical results compared to the experimental ones by Kent (1969) (a) and by Ma et al. (1976) (b)

3.2. Infill

The infill was replaced by two struts. The width and strength were calculated according to the Al-Chaar model (2002). Once these characteristic were determined, the infill was modelled in OpenSees through a truss element that works only for axial stress. The stress-strain law of the strut was determined according to the axial force-displacement response derived from the Al-Chaar model (Fig. 3.2). In order to complete the modelling of the strut it was necessary to examine the cyclic behaviour. In particular it was necessary to define the parameter for determining the strength degradation and the loading and unloading cycles. The application of the model proposed by Cavaleri et al. (2005) required the calibration of these parameters through comparisons with experimental results. The experimental tests considered were two: the first was conducted by the National Laboratory of Civil Engineering (LNEC) and it is available in Pires (1990), the second was carried out by Colangelo (2004).

3.2.1. Calibration of strength degradation

Initially it was necessary to determine the points which define the monotonic material behaviour assigned to the strut as shown in Fig. 3.2. In particular the problem was to identify as the constitutive law changes after yielding as a consequence of the strength degradation shown by the masonry. To this aim the slope of the second branch and the residual strength were varied in order to obtain a force-displacement response from the analytical model close to the envelope curve of experimental tests. The examined structures were subjected to a lateral force just in one way because only the force-displacement envelope curve was considered. Once the analyses were carried out, the constitutive law which provided the best agreement with experimental results was the one showed in Fig. 3.2. This law is characterized by a residual strength equal to the 25% of the ultimate strength and by a slope of the softening branch equal to 2% of the initial stiffness.

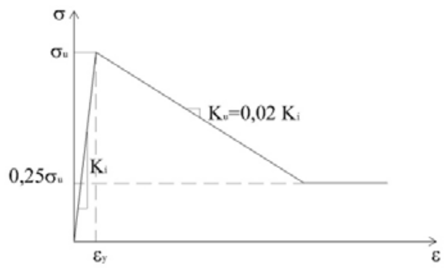

Figure 3.2. Constitutive law assigned to the strut

3.2.2. Calibration of cyclic behaviour

The cyclic behaviour was analyzed using an hysteretic constitutive law implemented in OpenSees for the truss element. This law is characterized by three parameters related to the definition of the unloading and reloading branches: β, used to determine the degraded unloading stiffness; p_x, pinching factor used to reduce the deformation during reloading; p_y, pinching factor used to reduce the axial force during reloading. These three parameters were calibrated in order to obtain a cyclic behaviour from the analytical model as close as possible to the one indicated by Cavaleri et al. (2005). The first examined aspect was the influence of the variation of the parameter β on the unloading branch of the strut (Fig. 3.3a). Cavaleri et al. (2005) proposed a double slope of the unloading branch: the first slope equal to the elastic stiffness, the second slope lower than the first. With a value of β equal to 0.5 the constitutive law implemented in the software resulted similar to the one proposed in literature.

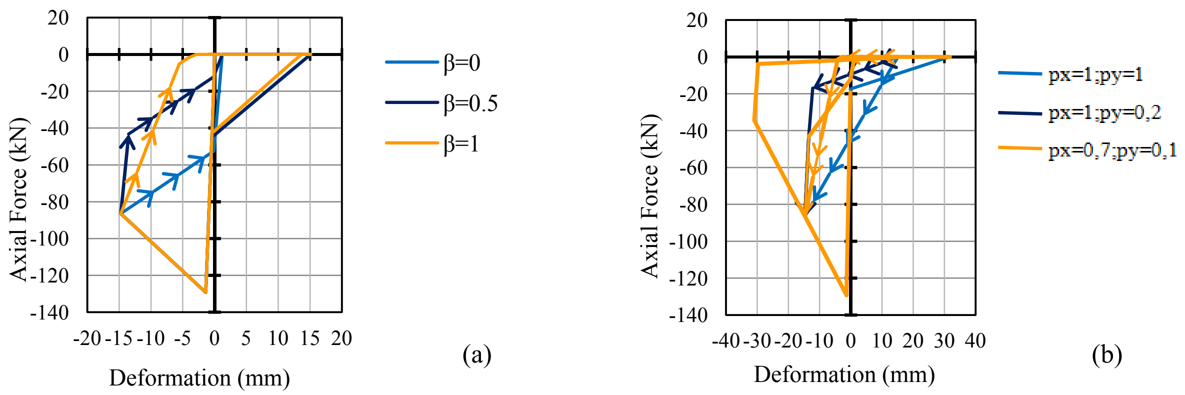

Figure 3.3. Strut response modified by the variation of the parameter β (a) and of the parameters p_x and p_y (b)

Once β was defined, the subsequent examined aspect was the influence of the variation of the parameters p_x and p_y on the reloading branch (Fig. 3.3b). According to the model proposed by Cavaleri et al. (2005), the reloading branch should have a reduced stiffness compared to the initial one. This branch should point toward the maximum deformation which was reached at the previous cycle. Furthermore the authors identified a first reloading branch characterized by no strength but compressive strain different from zero (MO_1 in Fig. 2.2). With the values p_x=0.7 and p_y=0.1 the implemented constitutive law was similar to the one proposed in literature.

3.2.3. Comparisons between analytical and experimental results

Finally, the implemented hysteretic model was verified and improved through comparisons with experimental results. Using the parameters previously defined (β=0.5, p_x=0.7, p_y=0.1), some little differences were observed between the numerical and experimental results. For this reason the model was updated in order to obtain a better correspondence between the two results: the new assumed parameters were β=0.8, p_x=0.8 and p_y=0.1. In Fig. 3.4 the numerical results obtained with these new values of the three parameters are compared with the experimental ones of Pires (1990) and Colangelo (2004). From the figure it is possible to observe a good agreement between the numerical and experimental results.

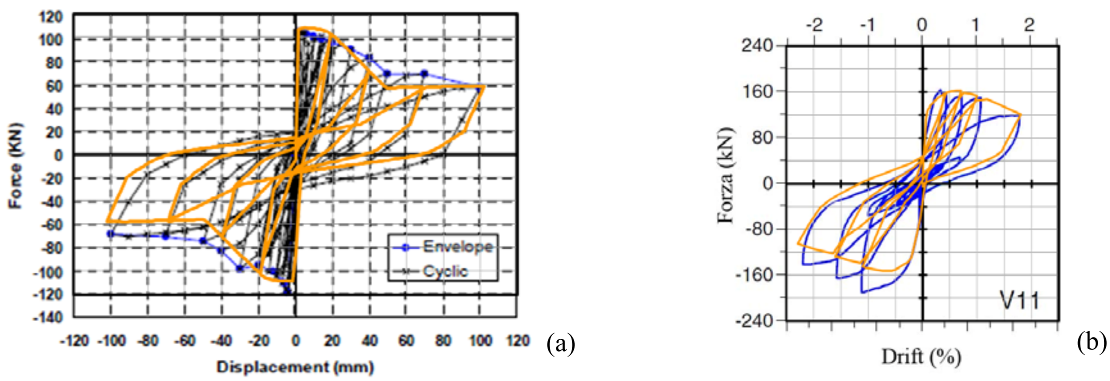

Figure 3.4. Numerical results for cyclic loading (orange curve) compared with the experimental results (blue curve) of Pires (1990) (a) and of Colangelo (2004)(b)

4. EXAMINED CASES

Once the model was calibrated, several pushover and nonlinear dynamic analyses of existing frames were carried out in order to study the mechanisms of failure and the deformed configurations. The analyses were performed on bare and infilled RC frames in order to evaluate how the masonry panel could modify the collapse mechanism. The examined bare frame was a five storey and symmetrical structure which was designed only for gravity loads. The adopted dimensions of beams were: width equal to 300 mm and depth equal to 500 mm. The adopted dimensions of columns were variable and are indicated in Fig. 4.1a. 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 beams and columns, the infilled frame was assumed identical to the bare one. Two types of masonry were examined: the infilled frame VAR has masonry characterized by compressive strength of 4.1 N/mm^2 and shear strength of 0.3 N/mm^2; the infilled frame UNIF has masonry characterized by 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 with thickness variable between 24 cm and 15 cm; instead the infilled frame UNIF has all masonry panels with thickness of 15 cm. As shown in Fig. 4.1, three cases were examined for both infilled frames: totally infilled frame, totally infilled frame with square openings of 150 cm in all the panels, infilled frame without panels at the ground floor.

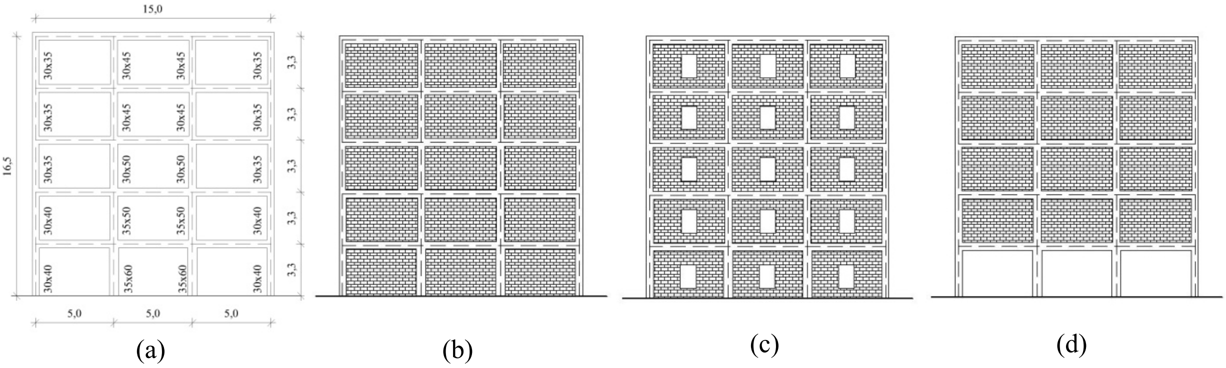

Figure 4.1. Representation of the examined frames: bare frame (a), totally infilled frame (b), totally infilled frame with openings (c) and infilled frame without panels at the ground floor (d)

5. PERFORMED ANALYSES

The objective of these 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:

a) achieving of the ultimate strain in confined concrete of columns
b) achieving of the ultimate interstorey displacement
c) achieving of the ultimate shear strength

The type of collapse "a" was associated to the first occurrence, in a confined concrete fibre, of the ultimate strain. The ultimate strain was determined using the relationship proposed by Mander et al. (1988). 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_{pl}) ones. The yield drift was calculated as a function of the geometry of the beam with the following expression proposed in literature (Priestley et al., 2007):

$$\theta_y = 0{,}5\varepsilon_y \frac{L_b}{h_b} \tag{5.1}$$

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

$$\theta_{pl} = l_p(\phi_u - \phi_y) \tag{5.2}$$

The yield drift of the examined frames resulted the same for all the storeys. Instead, the plastic drift was different for the external and the internal columns: for this reason the ultimate drift at each storey was assumed as the lower between the one of the external columns and the one of the internal columns. Once the ultimate drift was determined, the maximum displacement was calculated.

Pushover analyses were carried out using two set of forces: the first one with forces proportional to the first modal deformations and the second one with forces proportional to the masses. For each analysis, the displacements associated to the collapse conditions were identified. In correspondence of these displacements the plastic hinges were observed and the displacement profile was represented. This procedure allowed to find out which mechanism of collapse occurred. Subsequently incremental dynamic analyses were performed using three accelerograms (called here S1, S2, S3): each ground motion was scaled until the failure of the structure was reached. These three accelerograms are recorded ground motions characterized by an average displacement spectrum consistent with type 1 Eurocode 8 design spectrum. For each application of the accelerogram the maximum top displacement and the maximum base shear were obtained: in this way it was defined a point in the diagram top displacement-base shear. Once a point was determined for each dynamic analysis, we assessed which type of collapse occurred. At first we determined the intensity of the accelerogram which caused for the first time one of the defined collapse conditions. In correspondence of these values, the distribution of plastic hinges and the maximum displacement profile were evaluated. Furthermore each element of the frames was investigated in order to determine if it was collapsed according to one of the three conditions. The dynamic analyses were carried out only on the bare frame and on the totally infilled one.

6. RESULTS OF ANALYSES

Because of the large number of obtained results, only those related to the type of collapse "b" are illustrated in detail: in fact this type of collapse is the most significant as usually it is associated with a storey mechanism. We noted that the collapse "a" was reached for smaller displacements than the collapse "b" in all the conducted analyses, as it is shown in Fig. 6.1. The collapse "c", associated to the achieving of the ultimate shear strength, did never occur.

6.1. Pushover Analyses

The results of the pushover analyses are shown in Fig. 6.1. In general these results show that the contribution of the infill, in terms of base shear, was significant just for displacement lower than about 5 cm. For greater displacements the infills collapsed and their strength experienced a significant reduction. The collapse of the infilled frames always occurred at displacements smaller than the ones of the bare frame. However, for a given distribution of forces, the mechanism of collapse of the infilled and of the bare frame always occurred at the same storey. Furthermore, the results relative to the infilled frame show that the presence of the openings did not modify substantially the response of the frame. The openings just 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. At last we could note that the presence of the infill did not change the mechanism of collapse but affected the displacement profile: in fact, in the examined cases, the collapse occurred at the first or the third floor for both the bare and the infilled frame, while the displacement profile relative to the infilled frames was always characterized by lower values than the one of the bare frame (Fig. 6.2). This fact was related with the plastic hinges which, in the infilled frames, occurred only at the storey of collapse. With this distribution of hinges the collapse displacement was close to the ultimate displacement of the single storey, without taking advantage from the deformations of other the storeys.

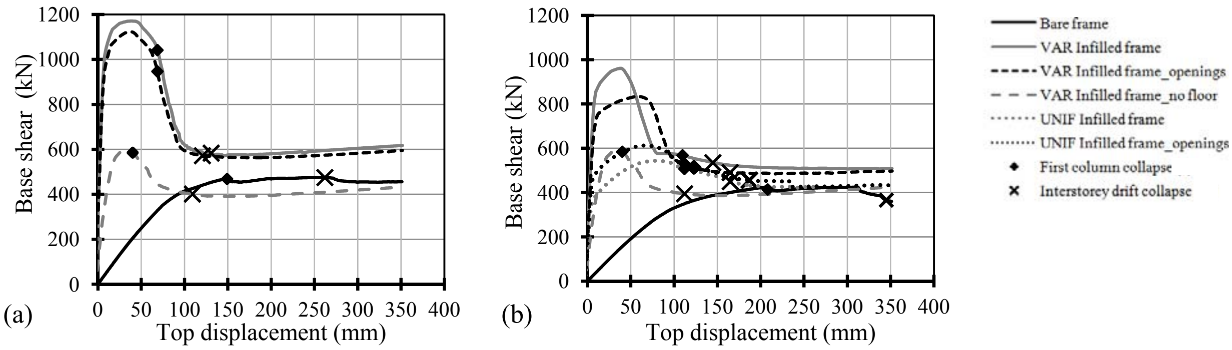

Figure 6.1. Pushover curves obtained with forces proportional to masses (a) and to the first mode (b)

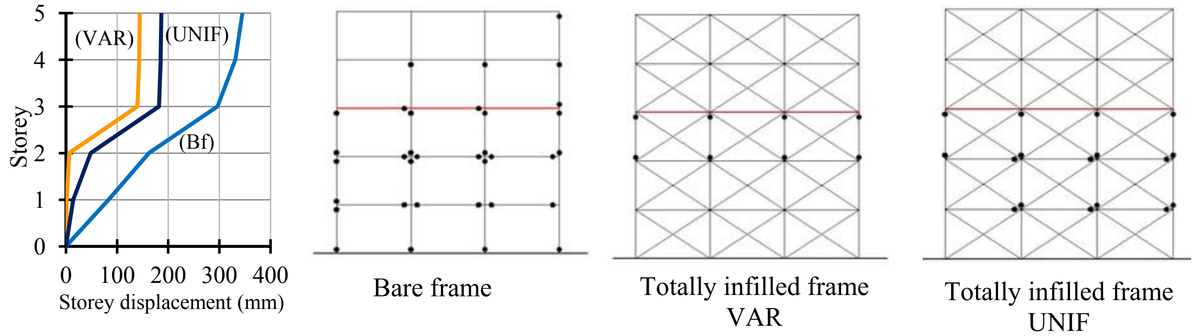

Figure 6.2. Displacement profile and distribution of plastic hinges at the collapse condition “b” (achievement of the ultimate interstorey drift) in the case of force proportional to the first mode

6.2. Dynamic analyses

In the results obtained from dynamic analyses the infilled frames were characterized by a greater strength but a lower collapse displacement than the bare frames. This result is similar to the one obtained from the pushover analyses (Fig. 6.3). With reference to the location of the storey mechanism, a different result than the one from pushover analyses was obtained. In particular with the same accelerogram the collapse of the infilled and of the bare frames occurred at different storeys. Therefore we could not conclude that the collapse mechanism was not modified by the presence of infills. However the dynamic analyses determined in the infilled frames a more spread distribution of plastic hinges at all the storeys than the pushover analyses. The displacement profile was affected by the displacements at all the storeys and not only by the one where collapse occurred. As a consequence of this distribution of plastic hinges the displacement profiles of the infilled frames resulted more similar to the one of the bare frame compared to the results of the pushover analyses. For example Fig. 6.4 shows the results relative to the application of the accelerogram S1.

With the incremental dynamic analyses it was possible to evaluate the peak ground acceleration at the collapse. In Table 6.1 the values of peak ground acceleration are illustrated for the different examined cases. In particular the average of the values of the three accelerograms are shown. For all the cases the seismic input necessary to reach the collapse of the infilled frame resulted larger than the one required for the bare frame. It could be noted that the values relative to the infilled frame are about 70% higher than the ones relative to the bare frame at collapse “a” and about 40% higher at collapse “b”. This result is related to the reduction of displacement demand in the infilled frames compared to the bare frame.

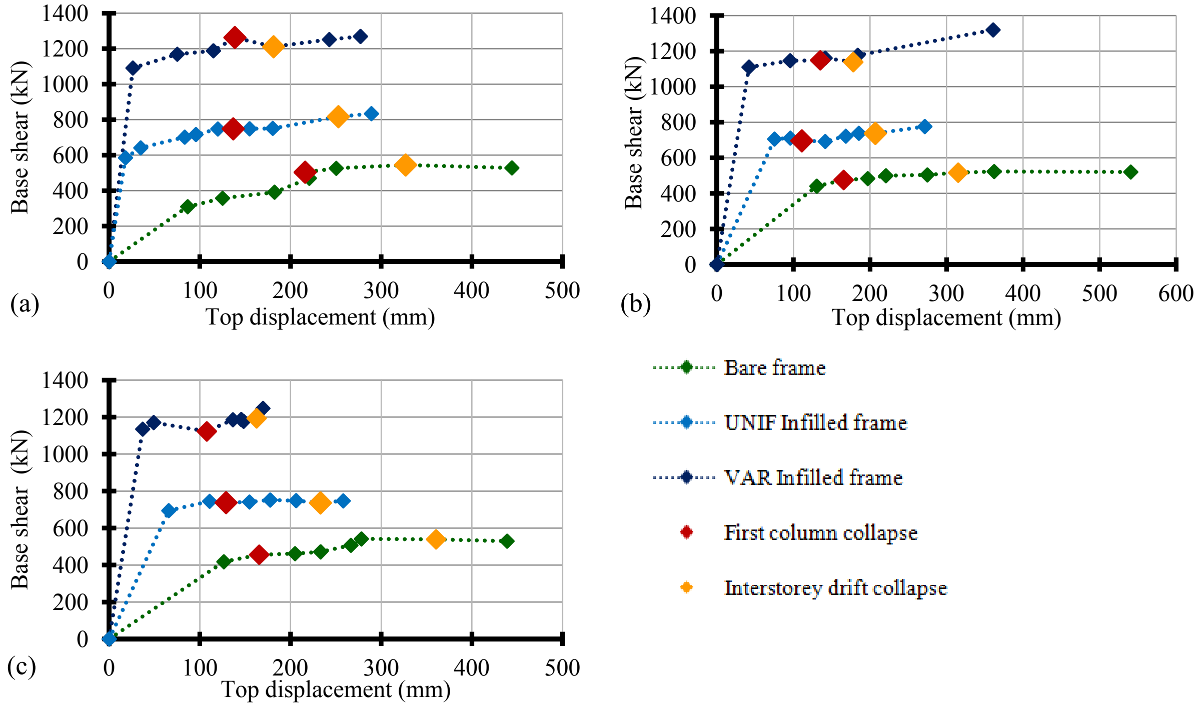

Figure 6.3. Top displacement-base shear curves from incremental dynamic analyses: S1 (a), S2 (b), S3 (c)

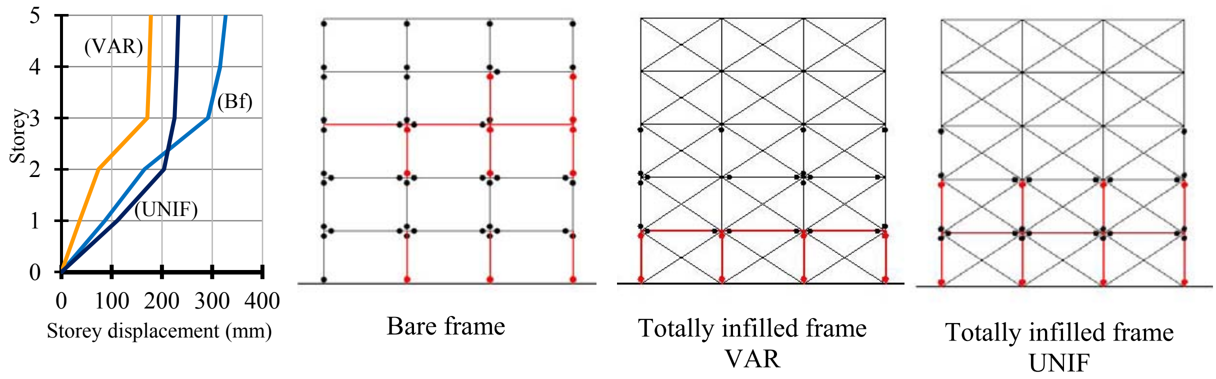

Figure 6.4. Displacement profiles and distribution of plastic hinges at collapse condition “b” for the accelerogram S1

Table 6.1. Values of peak accelerations at collapse and relative average values

		Bare frame	Infilled frame VAR	Infilled frame UNIF
Collapse “a” (ε_u)	S1	0,408g	0,68g	0,612g
	S2	0.268g	0,54g	0,469g
	S3	0,26g	0,455g	0,52g
	$a_{average}$	0,312g	0,558g	0,534g
Collapse “b” ($\Delta_{u,inter}$)	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

7. CONCLUSIONS

The analyses of the infilled frames revealed several difficulties which were especially due to the many variables related to the models of the masonry infills. The response of the infill is affected by many parameters related to the mechanical and geometrical characteristics of the masonry which are difficult to generalize. In order to resolve these difficulties the comparison with experimental results was fundamental. This comparison allowed to calibrate a reliable and general model for the infills. This model is characterized by an equivalent strut with a constitutive law for cyclic loading. This model could be used also for further studies because it relies on an extensive investigation.

Once the model was calibrated, the study continued with a series of nonlinear analyses of infilled frames. The initial aim was to determine the response of structures in terms of base shear-top displacement and to evaluate the collapse displacement. At the collapse the distribution of plastic hinges and the displacement profile were analysed. In this way it was possible to determine which mechanism of collapse occurred. The obtained results show that the presence of the infill increase significantly the response in terms of strength and stiffness, while it reduced the deformation capacity. All the analyses showed a collapse of the infilled frame at displacements smaller than the ones of the bare frames. This result was related with the distribution of plastic hinges, which in the infilled frame were present only near the collapsed storey.

Furthermore the larger stiffness and strength of the infilled structures caused a reduction of the displacement demand. For this reason the infilled frame was able to sustain larger peak ground accelerations than the bare frame.

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