SEISMIC PERFORMANCE OF PRE-CODE REINFORCED CONCRETE BUILDINGS

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SUMMARY

The present paper aims at deepening and extending the current knowledge about the evaluation of seismic vulnerability of existing reinforced concrete buildings. The object of the investigation consists in the analysis of the actual response and possible failure mechanisms of buildings constructed in the area of Southeast Sicily before adequate regulations for seismic design of structures came into effect, and representative of a widely diffused typological class. Nonlinear pushover and dynamic analyses were performed. As far as the seismic input is concerned, accelerograms both recorded during past events and generated according to the scenario earthquake expected for the area under investigation were utilized. The influence of masonry infilling was also taken into account resorting to a simplified model appositely developed in order to characterize both the strength and stiffness of the panels.

INTRODUCTION

The objective of the present research is to deepen and extend the current knowledge about the evaluation of seismic vulnerability of existing reinforced concrete buildings, through the analysis of the actual response and possible failure mechanisms. The investigation is mainly directed to buildings constructed before adequate regulations for seismic design of structures came into effect, and representative of a widely diffused typological class in the area of Southeast Sicily.

Nonlinear pushover and dynamic analyses were performed on pseudo-tridimensional models, identified thanks to the symmetry of the buildings examined. The influence of masonry infilling was also taken into account resorting to a simplified model appositely developed in order to characterize both the strength and stiffness of the panels. As far as the seismic input is concerned, accelerograms both recorded during past events and generated according to the scenario earthquake expected for the area under investigation were utilized. Strength demands in the reinforced concrete structural elements were checked with reference to the corresponding ultimate strength domains. The seismic performance was assessed through the analysis of the strength conditions of both the frame members and the masonry panels, varying the intensity level of the input.

DESCRIPTION AND MODELLING OF THE BUILDINGS

Characteristics of the materials and geometry

Two buildings belonging to a public housing complex in Catania (Southeast Sicily) and consisting respectively of an eight-story and a four-story reinforced concrete frame were analyzed. They both date back to the late 70s, therefore prior to the issuing of adequate provisions for seismic design of structures. The infill masonry panels

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consist presumably of two hangings made of tiles, with interposed hollow space. The first, eight-story building presents regular plan dimensions (11×22 m), with a typical story height of 3 m for a total height of 24 m. The structural configuration is characterized by three longitudinal and three transversal frames, with continuous footing built on piles. The structural members consist of rectangular columns, T beams (in the transversal frames) and rectangular beams contained in the floor thickness (in the longitudinal frames). The tile-lintel floors, (160+40) mm thick, are all equally oriented and resting on the longitudinal beams. The second, four-story building, though symmetric with respect to its minor axis, is less regular than the first, both in plan and in height. Plan dimensions are 3×10 m; the total height is 18 m along four stories, each 3.25 m high. The total mass is equal to 1864.0 kN for the eight-story building and to 1208.1 kN for the four-story building. The main properties of the materials are summarized as follows: *Steel* – characteristic value of the yield stress: $f_{yk} = 380$ N/mm^2; *Concrete* - characteristic value of the compressive strength: $R_{ck} = 25$ N/mm^2; *Masonry panels* - thickness: (120+80) mm, weight density: 16 kN/m^3, basic compressive strength: $\sigma_{m0} = 1.2$ N/mm^2; basic shear strength: $\tau_{m0} = 0.2$ N/mm^2, sliding strength in joints: $u = 0.133$ N/mm^2. The determination of the ultimate characteristics is affected by the large amount of uncertainty in the data available. Nevertheless, due to poor detailing (low percentage of longitudinal bars and stirrups, especially in the joints), scarce concrete strength and lack of confinement, a brittle failure due to concrete crushing is expected to occur.

Modeling of the frames

The hypothesis of rigid floor and the symmetry of the buildings examined allowed to reduce the study of the real tridimensional structures to that of a bidimensional model constituted by the transversal frames connected in parallel by means of inextensible links. Such a simplification appears correct on the basis of the coincidence of the values of the fundamental period observed for the frames modeled according to the two different schematizations. Moreover, a pseudo-tridimensional study limited to the minor plan dimension is justified both by the minor strength of the transversal beams with respect to that of the longitudinal beams and by the correspondence of the first natural mode with the flexural mode along that direction. A modified version of the nonlinear code *ANSR* [Mondkar and Powell, 1975] was adopted for the definition of the numerical model of the structures. Elements with lumped plasticity at both the ends of the structural members were used to schematize the behavior of beams and columns, while inelastic trusses apt to take pinching, softening and strength degradation into account were used for the resistant masonry, supposed to be present only in the lateral frames.

Infill masonry

It is well known that masonry infill can be modeled as equivalent struts hinged at the frame nodes and able to resist compressive loads only. In the present study a simplified model was adopted [Decanini et al., 1993, 1994], introducing significant modifications to the usual equivalent strut schematization, both in the definition of the strength, assumed as depending on several failure modes, and stiffness of the equivalent strut itself. Although the model aims at its use under monotonic loading conditions, the cyclic nature of the seismic action is taken into account through an adequate parameter calibration. Lateral strength and stiffness characteristics are considered taking complete or stabilized cracking conditions as a reference state for the formulation of the simplified model.

The equivalent strut is characterized by its dimension and compressive strength. The width (ω), depending on the mechanic characteristics of both masonry and concrete, and on the dimensions of both the panel and the frame, is expressed as a function of two semi-empirical coefficients (K_1 and K_2), a nondimensional parameter (λh), and the diagonal length of the panel (d):

$$(\omega/d)_{eq} = \frac{K_1}{\lambda h} + K_2 \tag{1}$$

The strength of the infill panel is simulated by a fictitious failure compressive stress, σ_{cu}, whose values depend on the most probable failure mode. Four basic rupture modes are considered, i.e. 1) diagonal tension failure; 2) sliding shear failure along the horizontal joints; 3) compression failure in the corner zone of contact with the frame; 4) diagonal compression failure. It should be noted that most rupture modes fall within the first two cases. The characterization of the failure modes is performed on the basis of the basic strength parameters of the masonry, i.e. a) the basic compressive strength (σ_{m0}), referred to prismatic specimen of masonry; b) the basic shear strength (τ_{m0}), determined through diagonal compression tests; c) the sliding strength in the joints (u), evaluated using specimens consisting in triplets subjected to different levels of lateral pressure. The minor failure compressive strength identifies the most probable failure mode, and is therefore attributed to the strut in the simplified modeling. In conclusion, the strength capacity of the equivalent strut at the ultimate limit state can be expressed as:

$$R_u = \sigma_{cu} \cdot e \cdot \omega \quad (2)$$

were e is the panel thickness.

RESPONSE ANALYSIS

Introductory remarks

The numerical models of the frames were analyzed in order to assess the seismic vulnerability of the structures in question from the study of the actual response and the identification of the possible collapse mechanisms. Both nonlinear static equivalent (pushover) and dynamic analysis were performed. Aiming to quantify the significant influence of the infill masonry on the seismic performance, such analyses were repeated either for bare and infilled frames. The preliminary determination of quantities such as vibration periods and excited masses was accomplished through modal analysis of the relevant spatial models. As anticipated, the principal vibration modes for the structures considered correspond to the flexural modes along the minor plan dimension. The relevant fundamental periods are equal to 1.76 s (bare frame) and 0.86 s (infilled frame) for the eight-story building; the corresponding values for the four-story building are 0.58 s and 0.42 s, respectively.

Pushover analysis

Nonlinear static analysis under monotonically increasing lateral loading constitutes an effective tool for the evaluation of the system demand in terms of forces and displacements imposed by the seismic event, as it accounts in an approximate manner for the redistribution of the internal forces developed within the inelastic range of the structural behavior. Pushover analysis represents therefore a powerful alternative to dynamic analysis in providing information on the global earthquake-resistant capacity and failure modes of the structures examined. It is based on the assumption that it is possible to relate the actual structural response to that of an equivalent SDOF system. This requires that the response is essentially controlled by a single mode, whose shape remains constant in time. From the modal analysis it was possible to verify such assumptions for the case under examination. The pushover analysis, performed with the application of a triangular load pattern, consented to bring to light design weaknesses and to predict unfavorable performances such as story mechanisms and excessive deformation demands, preliminarily to the dynamic analysis.

As far as the eight-story building is concerned, a nominal strength in terms of base shear equal to about 470 kN for the bare frame and 1030 kN for the infilled frame was determined (Figure 1). It appears that the presence of the infill masonry contributes to double the global resistance of the system. The values of the seismic coefficient C_y, defined as the ratio of base shear to gravity loads, are 0.026 and 0.056 for the bare frame and the infilled frame, respectively. A plastic hinge scenario is associated to each point of the base shear vs. roof displacement curve; the representation of such scenarios for the most significant points of the diagram consented to trace the occurrence of plastic hinges in the structural elements. Figure 2a illustrates the plastic hinge distribution in the infilled frame. The first damage develops in the third floor beams; then the plasticization propagates to the remaining beams, causing a remarkable reduction in the global stiffness. Subsequently to the plasticization of the beams, the collapse mechanism is activated as some of the base columns yield for concrete crushing, producing a sharp increase of the plastic curvature demand. The presence of the resistant masonry panels seems to contain the damage significantly, especially in the higher stories. Due to the rupture of the most stressed infill panels, and to the consequent presumable formation of a soft story, the damage concentrates particularly between the third and the fifth story. Such conjecture, as shown in a following section, was validated by the results of the dynamic analysis. With reference to the four-story building, a nominal strength in terms of base shear equal to about 900 kN for the bare frame and 1400 kN for the infilled frame was observed (Figure 1). The corresponding values of the seismic coefficient C_y are equal to 0.076 and 0.12, respectively. The damage scenario is characterized by an increasing spread of the plastic hinges, which come to affect almost the totality of the base columns at the structural collapse stage (Figure 2b). In this case the presence of the infill masonry, though increasing the global strength level, seems to modify neither the plastic hinge scenario nor the consequent collapse mechanism.

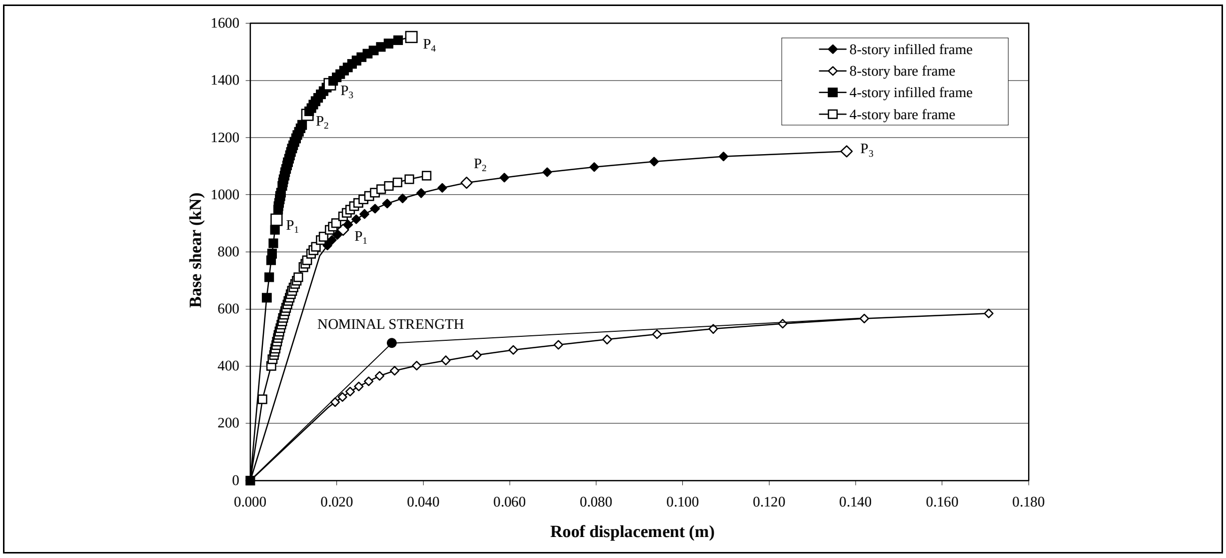

Figure 1: Pushover analysis: base shear vs. roof displacement for a triangular lateral force pattern.

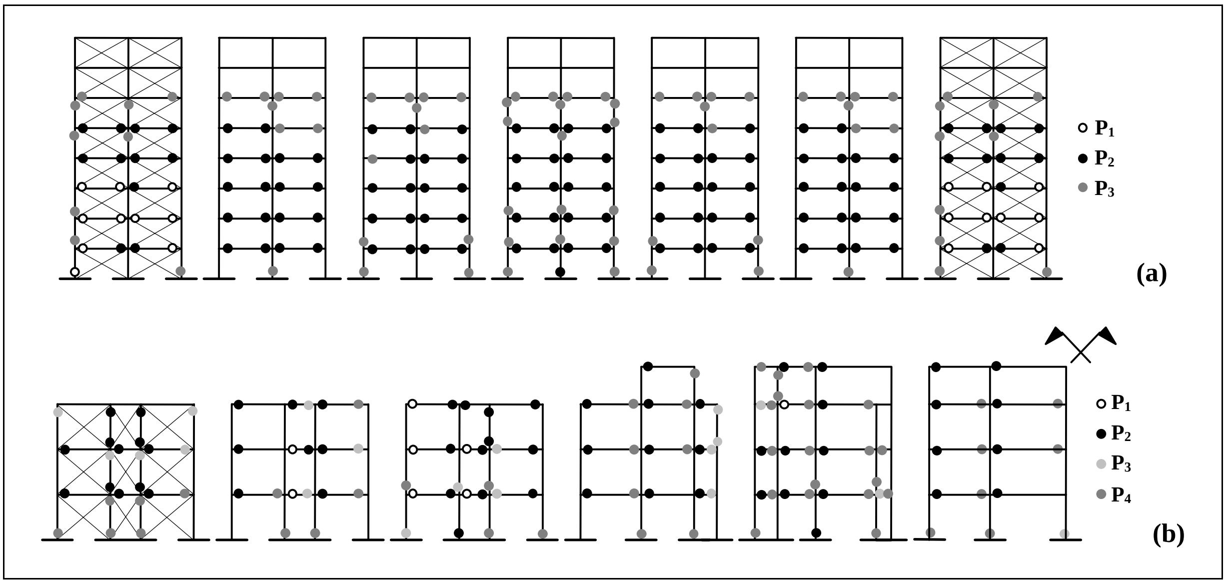

Figure 2: Pushover analysis: plastic hinge occurrence for 8-story (a) and 4-story (b) infilled frames.

Dynamic analysis

Definition of the seismic input

Many researches oriented to the study of the seismic action pointed out that the validity of the results may be heavily affected by the definition of the signals adopted in the evaluation of the structural system response. The choice of appropriate records for the case under examination required therefore a series of preliminary analyses, involving the classic elaboration of response spectra and the determination of the parameters characteristic of the ground motion damage potential. A broad set of strong motion records relevant to thirty-seven worldwide seismic events [Decanini and Mollaioli, 1998], together with a group of synthetic signals derived through two different methodologies (i.e., a simplified one [Romanelli et al., 1998], and a more detailed one [Priolo, 1999]), both referring to a scenario event comparable to the 11 January 1693, Southeast Sicily earthquake ($M \geq 7.1$). In conclusion, the analyses were performed using three strong motion records, i.e. El Centro N180, Tolmezzo EW, and Calitri EW, respectively recorded during the earthquakes of El Centro (1940, $M = 7.1$), Friuli, Italy (1976, M = 6.5) and Irpinia, Italy (1980, M = 6.8), and two synthetic signals denoted as S4_4 [Romanelli et al., 1998] and SEG_4R [Priolo, 1999]. Table 1 reports the value of some significant characteristic parameters of the accelerograms considered, i.e. the peak ground acceleration (PGA) and velocity (PGV), the Arias Intensity (I_A), the effective duration (t_D) [Trifunac and Brady, 1975], the area enclosed by the input energy elastic spectrum in the range of periods between 0.05 s and 4.0 s (AE_I), defined as a seismic hazard index in energy terms [Decanini and Mollaioli, 1998], and the value of the maximum ordinate of the input energy elastic spectrum, ($E_{I\,max,\,e}$).

Table 1: Parameters characteristic of the selected signals.

	El Centro N180	Tolmezzo EW	Calitri EW	SEG_4R	S4_4
PGA (cm/s^2)	342	348	177	356	829
PGV (cm/s)	38	32	32	49	33
I_A (cm/s)	174	119	136	154	228
t_D (s)	24.5	5.0	45.9	5.8	2.64
AE_I (cm^2/s)	17638	6966	20987	30612	5252
$E_{I\,max,\,e}$ (cm^2/s^2)	12219	16032	13657	32823	16531

The selected signals are representative of a wide class of properties. As far as the parameter t_D is concerned, it can be observed that while Tolmezzo EW, SEG_4R and S4_4 have short effective duration, El Centro N180 and Calitri EW have respectively medium and long effective duration. With reference to the energy aspects, the value of the index AE_I is quite high for SEG_4R, Calitri EW and El Centro N180, and low for the remaining two signals, these latter also characterized by a frequency content concentrated around the maximum energy value. In terms of damage potential it must be pointed out that critical conditions in a structure occur whenever a long (if compared with the system fundamental period) duration pulse has an average acceleration level of the same order of the seismic coefficient C_y. On the contrary, the high spectral accelerations typical of high frequency pulses are associated with modest quantities of energy effectively imparted to the structures. As a matter of fact, even signals characterized by a not particularly high absolute destructiveness power may determine critical conditions in the structure, being frequency content, sequence and duration of the pulses the decisive factors. In the present case, the low values of C_y, representative of the earthquake-resistant capacity of the buildings, induced to scale the accelerograms in amplitude. As the maximum resistance and energy demands associated with Tolmezzo EW and S4_4 appeared to concentrate around periods rather distant from the fundamental values determined for the buildings examined, while a greater destructiveness power was expected for Calitri EW, SEG_4R and El Centro N180, only the results obtained for the latter group of accelerograms will be reported and commented in the following.

Description of the results

The values of the peak ground acceleration reached in incipient collapse conditions are reported in Table 2. The evolution of the plastic hinge scenarios confirms the results of the pushover analysis. In the case of the eight-story building, the bare frame presents a widespread plasticization in the beams comprised between the third and the sixth floor, even when exposed to low PGA levels; the damage extends to the other floors and to the base columns for accelerations around 0.1 g. The behavior of the infilled frame follows the same evolutive trend, with an evident increase in the resistance to the seismic input (PGA = 0.15÷0.18 g). In both cases, collapse occurs due to concrete crashing in the base columns. Analogous considerations hold for the case of the four-story building; nevertheless it shows a resistance level lower than the eight-story building.

Table 2: PGA values in incipient collapse conditions.

Seismic input	El Centro N180		Calitri EW		SEG_4R	
8-story building	Bare	Infilled	Bare	Infilled	Bare	Infilled
PGA (g)	0.1	0.18	0.09	0.14	0.1	0.15
4-story building	Bare	Infilled	Bare	Infilled	Bare	Infilled
PGA (g)	0.05	0.12	0.07	0.10	0.07	0.13

Figure 3 illustrates the envelopes of the relative interstory drift for different levels of the seismic input. It appears that the introduction of the resistant masonry contributes to both limiting the drift value and regularizing the global behavior, particularly in the case of the eight-story building. Due to the scarce resistance of beams and columns, the masonry panels are not allowed to describe complete cycles including the degrading branches before the mechanism is activated for excessive damage in the frame members. Such a circumstance accounts for the good agreement with the results of the pushover analysis which, due to the fact that it was performed under monotonic loading, does not allow to detect the degrading behavior of the infill masonry. Figure 4 shows a comparison between the maximum base shear obtained from the dynamic analysis and the pushover curves. As expected, the pushover curve constitutes a lower bound to the values derived from the dynamic analysis.

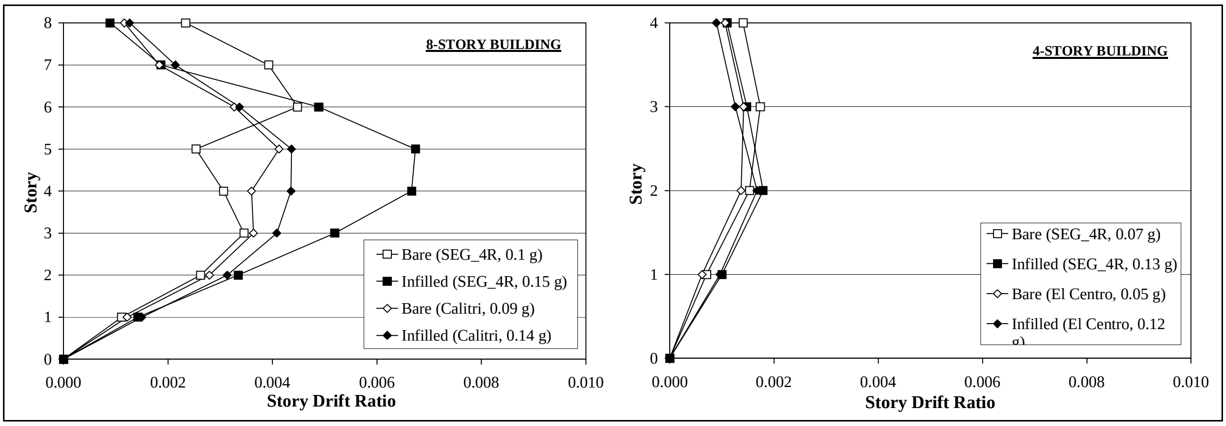

Figure 3: Evolution of the story drift ratio (envelopes).

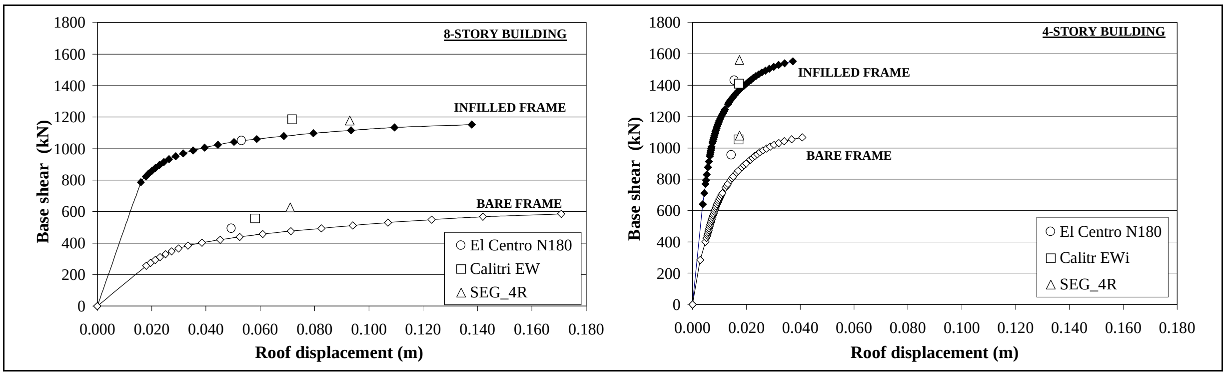

Figure 4: Comparison of dynamic and pushover analysis.

Some remarks on the earthquake-resistant capacity of the buildings

As stated previously, being performed with a monotonically increasing lateral load pattern, the pushover analysis did not allow to trace a softening branch due to the progressive structural damage. It was therefore difficult to define the structure ductility, either global or at story level, only relying on the results derived from the pushover analysis. In order to overcome such difficulty, being the values of the maximum dynamic base shear sufficiently close to those determined through the static analysis, it was possible to refer to the ultimate values of the displacements (δ_u) obtained from the dynamic analysis, taking instead as yielding limit displacement the values δ_y obtainable from the pushover curves. The maximum values of the available displacement ductility (μ) evaluated according to this criterion are reported in Table 3.

Further analyses on a set of elastoplastic SDOF systems with different mechanic properties, subjected to the accelerograms El Centro N180, Calitri EW and SEG_4R (scaled to the PGA corresponding to incipient collapse conditions), provided the relevant seismic coefficient, displacement and input energy spectra, for a 5% damping ratio. Five values of the displacement ductility were considered, i.e. 1 (elastic case), 1.5, 2, 4, 6. On the basis of such spectra, and taking into account the initial (T_i) and effective fundamental period (T_{eff}, derived from the secant stiffness corresponding to the ultimate displacement, δ_u), an attempt was made to check whether the values of the ductility determined through the procedure illustrated above were in agreement with the structure actual response. The following remarks, easily extensible to the case of the four-story frame, refer to the eight-story building, with particular reference to Calitri EW and SEG_4R, which provide limit conditions in terms of duration and performance in the inelastic range. Figure 5 illustrates the seismic coefficient spectra for Calitri EW and for different values of the displacement ductility. Comparing the maximum value of C_y derived from the pushover analysis and the ordinates of the spectra in the range of periods between T_i and T_{eff}, it can be seen that the maximum C_y spectral value is attained for ductilities ranging from 2 and 3 in the bare frame case, and varying between 3 and values even greater than 6 in the infilled frame case. With reference to the signal SEG_4R (Figure 6), the ductility ranges between 1 and 2 for the bare frame and between 2 and 6 for the infilled frame.

Table 3: Displacement ductility.

	8-story bare frame					8-story infilled frame				
	δ_y (cm)	δ_u (cm)	μ	T_i (s)	T_{eff} (s)	δ_y (cm)	δ_u (cm)	μ	T_i (s)	T_{eff} (s)
El Centro N180	3.5	4.9	1.4	1.76	2.1	2.5	5.3	2.1	0.86	1.25
Calitri EW	3.5	5.8	1.7	1.76	2.3	2.5	7.2	2.9	0.86	1.46
SEG_4R	3.5	7.1	2.0	1.76	2.5	2.5	9.3	3.7	0.86	1.65
Mean	3.5	5.9	1.7	1.76	2.3	2.5	7.3	2.9	0.86	1.46
	4-story bare frame					4-story infilled frame				
	δ_y (cm)	δ_u (cm)	μ	T_i (s)	T_{eff} (s)	δ_y (cm)	δ_u (cm)	μ	T_i (s)	T_{eff} (s)
El Centro N180	1.3	1.5	1.2	0.58	0.64	0.9	1.6	1.8	0.42	0.56
Calitri EW	1.3	1.7	1.3	0.58	0.66	0.9	1.7	1.9	0.42	0.58
SEG_4R	1.3	1.8	1.4	0.58	0.69	0.9	1.7	1.9	0.42	0.58
Mean	1.3	1.67	1.3	0.58	0.66	0.9	1.67	1.9	0.42	0.58

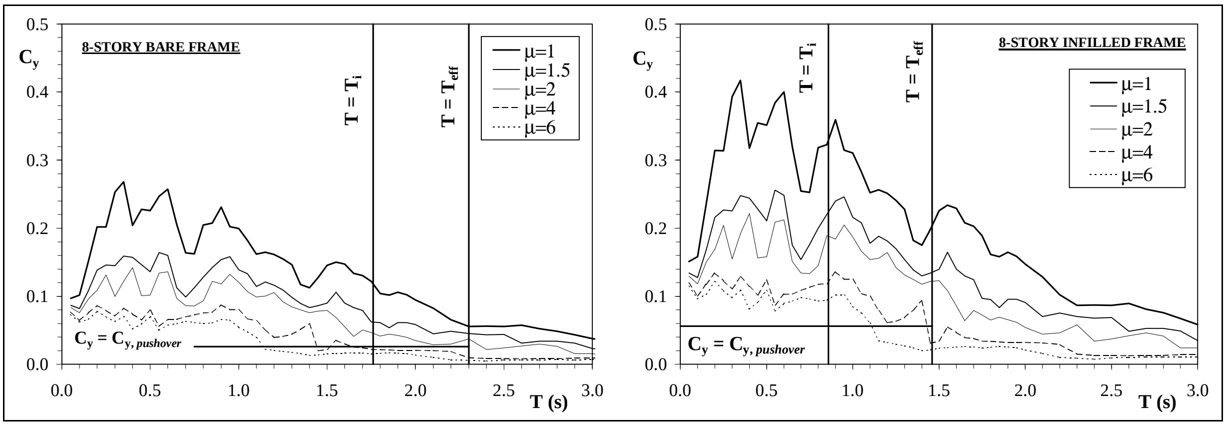

Figure 5: Calitri EW: PGA = 0.09 g, bare frame (left); PGA = 0.14 g, infilled frame (right).

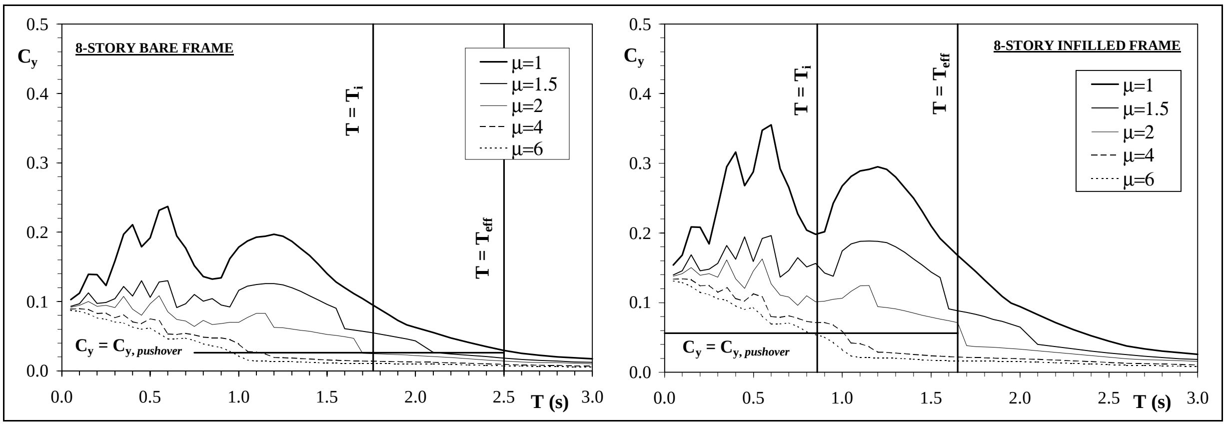

Figure 6: SEG_4R: PGA = 0.10 g, bare frame (left); PGA = 0.15 g, infilled frame (right).

The agreement of the values of the effective periods with the estimated ductility demand appears less satisfactory in the case of SEG_4R. However, it should be noted that, while the damage scenarios corresponding to the two signals are substantially similar, the way in which the damage is cumulated is remarkably different, as it can be easily verified from the analysis of the time histories of the input energy. In fact, in the case of SEG_4R almost the totality of the input energy is imparted to the system rapidly, bringing the building to collapse after a few damaging cycles. On the contrary, the damage produced by the Calitri EW signal develops rather gradually, due to the presence of several jumps in the energy time histories.

CONCLUSIONS

An overall evaluation of the results obtained allows to draw the following conclusions: a) the seismic performance of buildings without resisting masonry panels is very poor, and the EPA corresponding to collapse conditions does not exceed 0.1 g; b) the presence of infilling continuous in elevation reduces the vulnerability level. The EPA corresponding to collapse conditions reaches 0.2 g, but no difference in the collapse mechanism was detected; c) if not adequately distributed and located, the masonry panels may give raise to concentrated inelastic strain, though with a growth in strength with respect to the bare frames; d) the attitude to energy dissipation globally displayed by the typologies under investigation is extremely scarce; e) the collapse occurs as a consequence of the base columns yielding for concrete crushing; f) a comparison of the results of dynamic and pushover analysis allows to hypothesize that, depending on the energy characteristics of the seismic input, different ductility demands correspond to substantially identical collapse mechanisms; g) seismic retrofitting of such existing buildings is of problematic realization and doubtful effectiveness. Satisfactory solutions may consist in the introduction of shear walls and dissipative bracings.

Although the present research addressed the behavior of buildings realized in absence of appropriate seismic regulations, many results may be utilized in the evaluation of the vulnerability of those buildings constructed according to codes which do not prescribe constructive details apt to guarantee sufficient energy dissipation.

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REFERENCES

Decanini, L. Gavarini, C. and Bertoldi, S.H. (1993), «Telai tamponati soggetti ad azioni sismiche. Un modello semplificato. Confronto sperimentale e numerico», *6° Convegno Nazionale ANIDIS*: *L'Ingegneria Sismica in Italia, Perugia, 13-15 October 1993*, 2, pp815-824.

Decanini, L., Gavarini, C., Bertoldi, S. and Mollaioli, F. (1994), «Modelo simplificado de Paneles de Mamposteria con Aberturas incluidos en marcos de concreto reforzado y metalicos. Comparacion y calibracion con resultados experimentales y numericos», *9th International Seminar on Earthquake Prognostics, 19-23 September 1994, San Josè, Costa Rica*.

Decanini, L., Gavarini, C. and Mollaioli, F. (1994), «Strengthening of frame buildings with infill masonry». *Proc. of the Italian-French Symposium on Strengthening and Repair of structures in seismic areas, 17-19 October 1994, Nice, France*, pp175-184.

Decanini, L. and Mollaioli, F. (1998), «Formulation of elastic input energy spectra», *Earthquake Engng. Struct. Dyn.*, 27, pp1503-1522.

Mondkar, D.P. and Powell, G.H. (1975), *ANSR 1: General Purpose Program for Analysis of Nonlinear Structural Response*, Report No. EERC 75-37, University of California at Berkeley.

Priolo, E. (1999), «2-D spectral element simulations of destructive ground shaking in Catania (Italy)», *Journal of seismology* (in press).

Romanelli, F., Vaccari, F. and Panza, G. F. (1998), «Ground motions scenario in the Catania area for a Magnitude 7.0 earthquake on the Hyblean fault», *Journal of Seismology* (in press).

Trifunac, M.D. and Brady, A.G. (1975), «A study on the duration of strong earthquake ground motion», *Bulletin of Seismological Society of America*, 65, pp581-526.