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FLOOR ACCELERATION DEMAND IN REINFORCED CONCRETE FRAME STRUCTURES WITH MASONRY INFILL WALLS

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ABSTRACT

This study deals with the assessment of floor acceleration demands on reinforced concrete moment-resisting frames (RCFs) representative of typical building inventory in Italy. Floor acceleration can be used to estimate damage to nonstructural components. To this purpose we have considered 2-, 4-, 6- and 10-story concrete frame buildings with regular plan and stiffness and mass distributions along the height and designed for different base shear values. For each height we modeled three different frame/wall configurations: bare frame, frame with masonry infill walls at all stories, and frame with infill walls at all stories but ground level. In this article we have examined under what circumstances the presence of infill walls influences the peak and spectral floor acceleration demands along the height of these concrete frame buildings.

Introduction

Non-structural components are those parts of a building that are not meant to withstand gravity load or lateral loads. They include, for example mechanical and electrical equipment, piping, partitions, non-load bearing walls, and cladding. Recent earthquakes have demonstrated that damage to non-structural components constitutes a significant part of the total loss (Rodriguez et al. 2002, Villaverde 2004, Miranda and Taghavi 2005, Medina et al. 2006). Moreover, collapse of non-structural elements may cause injuries and fatalities. Although the importance of seismic design and assessment of non-structural elements is clear, in the last years only few studies have treated this topic (e.g., Sankaranarayanan and Medina 2006, 2008), particularly for existing buildings.

For damage assessment purposes non-structural components are usually classified into displacement-sensitive and acceleration-sensitive. The response of acceleration-sensitive non-structural components is correlated with the peak absolute acceleration or the spectral floor acceleration that they experience during the ground shaking. The design of concrete buildings in

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highly seismic areas often results in very strong frames that withstand well the seismic loading but cause extremely high floor accelerations. The consequence of such strong-frame design philosophy is that non-structural components (and contents) that make for most of the replacement value of the building suffer a disproportionately high amount of damage, if not adequately restrained.
Estimating peak or spectral floor acceleration, however, has proven to be a more challenging task than assessing maximum inter-story drifts. The difficulty of this task is also due to the presence of infill walls whose interaction with structural components during the shaking can modify significantly the acceleration demand at different floors. This paper presents the results from a parametric study aiming at investigating to what degree and under what conditions the peak absolute and spectral floor accelerations along the height of different concrete frame buildings are influenced by the presence of infill walls.

Structural models and Ground motions

This study considers reinforced concrete two-bay, two-dimensional frames designed for different seismic areas with different masonry infill wall configurations modeled both as simplified nonlinear shear-type models and more realistic distributed plasticity and fiber models with and without shear-failure springs and inelastic beams. The demand of these buildings measured in terms of floor peak absolute and spectral accelerations are computed via nonlinear dynamic analysis by applying a large ensemble of strong ground motions. The floor acceleration linear response spectra for 5% damping are obtained by applying the floor time histories to a suite of SDOF oscillators.

Table 1. Existing reinforced concrete buildings in Italy grouped according to: number of floors (*N*), seismic design (S=0: non seismic design, S=1: with seismic design), mean story area and relative percentage with respect of the total

	Mean Story area (m^2)					
***N*_S**	**0-50**	**50-100**	**100-200**	**> 200**	**TOT**	**%**
1_0	8382	118950	167952	26948	322232	11.64%
1_1	4793	81325	85467	10404	181989	6.57%
2_0	101304	506009	229612	33539	870464	31.45%
2_1	70188	272899	95281	13160	451528	16.31%
3_0	45825	128750	122600	42625	339800	12.28%
3_1	35364	74156	59908	17148	186576	6.74%
4_0	7555	23020	60448	45284	136307	4.92%
4_1	6304	13452	26669	14621	61046	2.21%
5_0	1166	4623	27919	37446	71154	2.57%
5_1	725	2311	9590	9125	21751	0.79%
6_0	569	1642	15108	29125	46444	1.68%
6_1	184	565	4119	6178	11046	0.40%
7_0	218	595	7725	18724	27262	0.98%
7_1	55	146	1631	3168	5000	0.18%
8_0	323	570	5849	25382	32124	1.16%
8_1	59	156	724	2543	3482	0.13%

The frame typologies were selected to be representative of a wide set of existing reinforced concrete buildings in Italy as described by the dwellings inventory recently collected by ISTAT

(Italian National Institute of Statistics). Industrial constructions or buildings used for functions other than housing were not included in that survey. Table 1 reports a preliminary estimate of the number of buildings subdivided in classes of mean floor area (Bramerini and Di Pasquale, 2006). Buildings of 5 or fewer stories constitute 95% of the whole stock. Low-rise typologies prevail also if the buildings are classified by number instead of floor area. The typical plan configurations considered here were derived by combining number of floors data with mean floor area data (Bazzurro et al. 2006).
In total we first considered 28 different two-dimensional frames (including also those containing the stairs shaft) by extracting the frames along the principal directions of each building typology. However, since additional parameters (such as the design base shear level and the configuration and lay-out of masonry infill walls) also needed to be taken into account in this study, the original number of frames was reduced to 21 to limit the number of computer analyses. However, to give breadth to the present research we decided to consider also building typologies that are infrequent in the ISTAT inventory (i.e., high-rise frames).

In the end, we evaluated frames having 2, 4, 6 and 10 stories with regular stiffness and mass distribution in plan and elevation. The bare frames were designed according to four different values of the base shear seismic coefficient, C_y: 0.10, 0.15, 0.25, 0.35. C_y is the ratio of the maximum base shear to the conventional weight of the building, which accounts for dead loads and a fraction of the live loads. The value of 0.10 is representative of the weakest Italian RCFs designed for gravity loads only (Bruno et al. 2000). The other three higher values refer to buildings with different combinations of lateral strength and designed according to past (1975-2003) and current (post 2003) Italian seismic codes in different seismicity zones.
Finally, three different wall configurations were considered (Fig. 1): i) no walls (i.e., bare frame, B); ii) frame with masonry infill walls at all stories (T); and iii) frame with infill walls at all stories but the ground level (i.e., *pilotis* frame, P), which is the classical soft-story case.

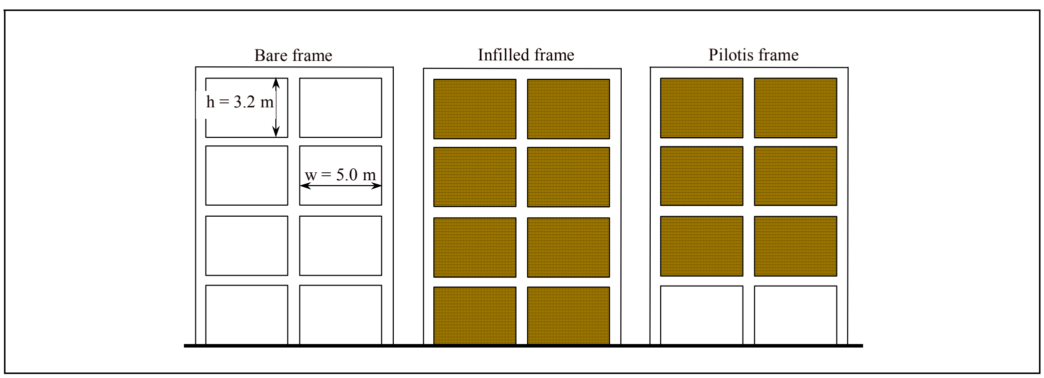

Figure 1. R.C. frame typologies: Bare (B), Infilled (T), e Pilotis (P)

A series of two-dimensional models were built and analized using OpenSees (McKenna et al., 2007). The nonlinear response was computed using nonlinear *Beam-Column* elements based on a force formulation, and considering the spread of plasticity along the element. The integration along the element is based on Gauss-Lobatto quadrature rule, with five integration points. The element section is discretized into fibers, each associated to the constitutive law that defines the stress/strain material (concrete and steel) response.

Since the objective of this study is to estimate the entire spectrum of building demand in terms of peak absolute and spectral floor accelerations, the 98 ground motions used in this study were selected in such a way that some of them would drive even the more modern of the considered buildings into the severe nonlinear response range. To ensure that the records were representative of moderate to extreme events, bounds were imposed on event magnitude, source-to-site distance, as well as peak ground velocity (PGV) and acceleration (PGA). The selected records have the following features: a) Moment Magnitude M_w, between 5.0 and 7.6; and b) Closest distance from the causative fault (D_f) from 0.7 to 21 km (several records belong to the near-fault, backward and forward directivity regions); c) all selected records are on sites located on C-D boundary of the NEHRP soil type categories; d) Peak ground horizontal acceleration between 150 and 1200 cm/s^2 and Peak ground velocity from 10 to 170 cm/s.

Peak floor acceleration demands

Peak floor acceleration, PFA, often referred to as the *zero-period acceleration* of the floor motion (i.e., the "anchor" point for floor response spectra), is the maximum acceleration demand of very stiff non-structural components. The knowledge of the distribution of PFAs for various building configurations (Singh et al. 1993; Rodriguez et al. 2002; Chaudhuri and Hutchinson 2004; Medina and Krawinkler 2004; Miranda and Taghavi 2005), can lead to a better characterization of the floor acceleration responses.

Figs. 2, 3, and 4 show the most interesting results of the analyses carried out on all the different concrete frame buildings investigated in the study. It can be seen (e.g., see Fig. 2 and 3) that for several ground motions the maximum accelerations in certain floors are smaller than the PGA. As the plots in Fig. 4 show, in the high nonlinear range the maximum PFA occurs at middle stories rather than at the top floor levels. Moreover, with the increase of the inelastic demand in the supporting structure, the maximum floor accelerations stabilize and remain almost constant over the height of the building. The mean values of the maximum normalized peak floor acceleration demands (PFA/PGA) evaluated for the studied buildings subjected to all the considered earthquake ground motions have been observed to range between 0.5 and 1.7, and 0.4 and 1.9 for the concrete frames designed for C_y =0.25 and C_y =0.35 respectively. The pilotis and infilled frames present similar trends.

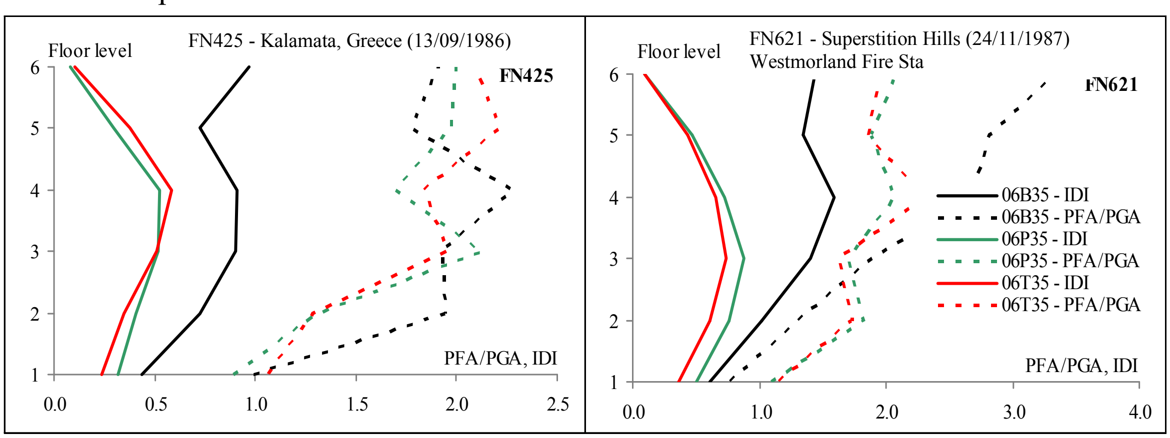

Figure 2. Peak floor accelerations (PFA) normalized by PGA and inter-storey drift demands, IDI (%), for the three configurations in which the strong, C_y =0.35 6-storey building has been modeled. Legend: 06B35=bare frame, 06P35= pilotis, and 06T35=frame with infill walls). FN425 and FN621 are two selected records.

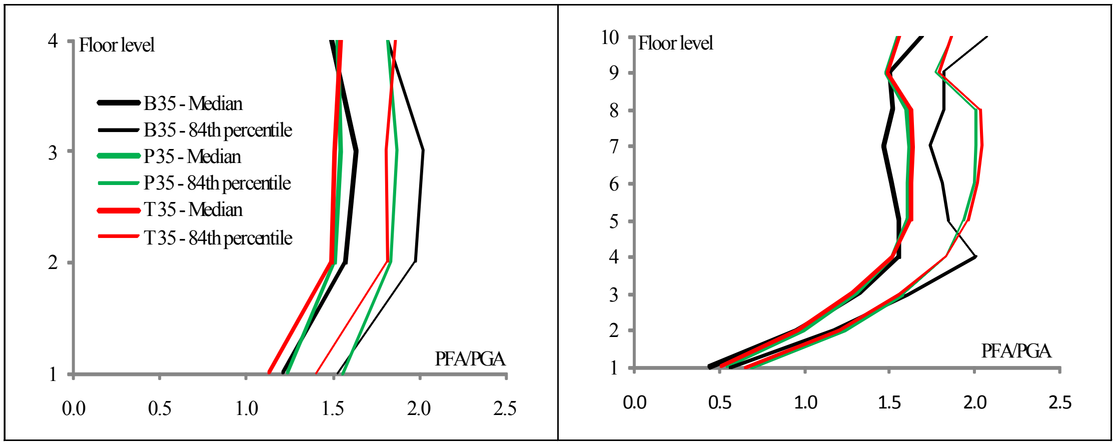

Figure 3. Mean and 84th percentile of the normalized peak floor acceleration ratio (PFA/PGA) for the strong (C_y=0.35) 4- and 10-storey buildings. Legend: Black=bare frame; Green= pilotis, Red=frame with infill walls.

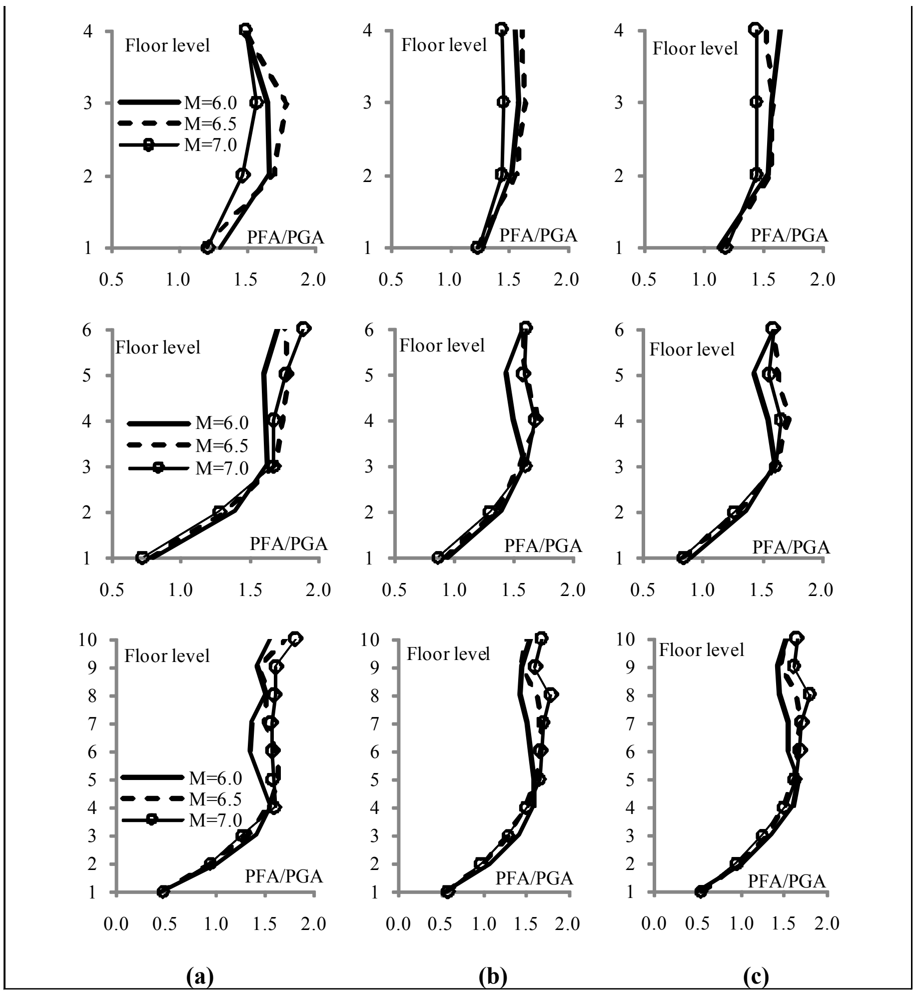

Figure 4. Mean curves of the normalized peak floor acceleration ratio (PFA/PGA) for the strong (C_y=0.35) 4-, 6- and 10-storey buildings. Legend: (a) bare frame; (b) pilotis; (c) frame with the infill walls. The three different curves were obtained by binning the results conducted using ground motion records from earthquakes with magnitude equal to 6.0 (25 records), 6.5 (24 records), and 7.0 (29 records).

Floor response spectra

In all the investigated cases this study confirms the influence of mode shapes and oscillatory periods of the supporting structure, and the location of the non-structural component along the height of the building on the 5%-damped elastic floor response spectra (FRS). Similar findings were observed previously by other researchers (e.g., Sewell et al. 1986, Singh et al. 1993, Medina et al. 2006). This influence can be clearly observed, for example, in the results of the analyses carried out on the 6-storey buildings (designed for high-seismicity areas) shown in Figs. 5, 6 and 7. Findings obtained in the study of the 2-, 4- and 10-storey structures also exhibited similar trends and they were omitted here for brevity.

Fig. 5 displays the 5%-damped acceleration spectra (S_{aC}) for the bare frame (left panels) and for the frame with infill walls uniformly distributed along the height (right panels). In particular, Fig. 5 shows the mean floor response spectra obtained by binning the input records by magnitude in the neighborhood of 6, 6.5 and 7, respectively. Looking at M binning is legitimate since the range of distances for these records is very limited. The FRS are plotted as a function of the T_C/T_{B1} ratio, where T_C is the fundamental period of the non-structural component and T_{B1} is the fundamental period of the supporting structure, and normalized for each considered floor level by the corresponding PFA value. In the case of the bare frame, the maximum amplification occurs when the component is in tune with the first (T_{B1}) or the second (T_{B2}) modal period of the supporting structure (T_C/T_B equal to 1 and 0.32 respectively). In particular, due to the damage of the structure during the shaking, the maximum accelerations occur at slightly elongated T_C values ranging from 1 to 1.2 times the elastic periods. It is interesting to note that the higher the floor level, the higher the amplification at periods around T_{B1}. The increase in amplification does not seem to scale with height for periods closer to the second period of the structure, T_{B2}.

This different trend with the floor level can be at least in part explained by the different contribution to the response of the building given by each single mode of vibration. While the first mode contribution increases with the height of the structure (i.e., the deflected shape of the first mode has no inflection points), the second mode contribution at the roof and at intermediate floor levels is comparable. In addition, the increase in the FRS amplification close to the first mode period is emphasized for records caused by larger magnitude events. On average, records from larger magnitude events are, in fact, richer in longer period content. Finally, it is interesting to observe that, as noted also in other studies (e.g. Medina et al. 2006), the maximum value in the mean FRS never exceed the value of 3.

Although similar trends have been identified in the frame structure with the infill walls as well, some differences can also be detected. First, many of the S_{aC}/PFA curves show a peak of amplification only around the second period of the building. In this case, the amplification around T_{B1} (i.e., 1.0s) become significant only for the top floor levels of the building and for ground motions from large magnitude earthquakes. Another difference can be noted in the plateau of amplification between T_{B1} and T_{B2} rather than a valley as in the bare frame case. As shown in Fig. 6, which displays the normalized acceleration FRS obtained for selected earthquakes only [FN275 (Obregon Park St., 1987 Whittier Narrows eqk, M_w=6.0), FN356 (Convict Creek St., 1980 Mammoth Lakes eqk, M_w=6.0), FN777 (Saratoga - Aloha Ave St., 1989 Loma Prieta eqk, M_w=6.9), FN851 (Duzce St., 1999 Duzce, Turkey, eqk., M_w=7.1)], well defined peaks can be easily identified for almost all of the input motions only in the case of the bare frame. In the frame with infill walls case the normalized FRS are often characterized by more than one peak (see the dashed curves). In the presence of infill walls the type and level of damage of the structural system is in fact more sensitive to the properties of the ground motion record. Another difference in the frame with infill walls case is that the acceleration peaks, due to the significant loss of stiffness produced in the building by the damage to the infill walls, can occur at T_C/T_{B1} and T_C/T_{B2} values much higher than those observed in the case of the bare frame structure. This is clearly shown in Fig. 6 (see the solid curves), which shows also peaks at T_C/T_B values greater than 1.5.

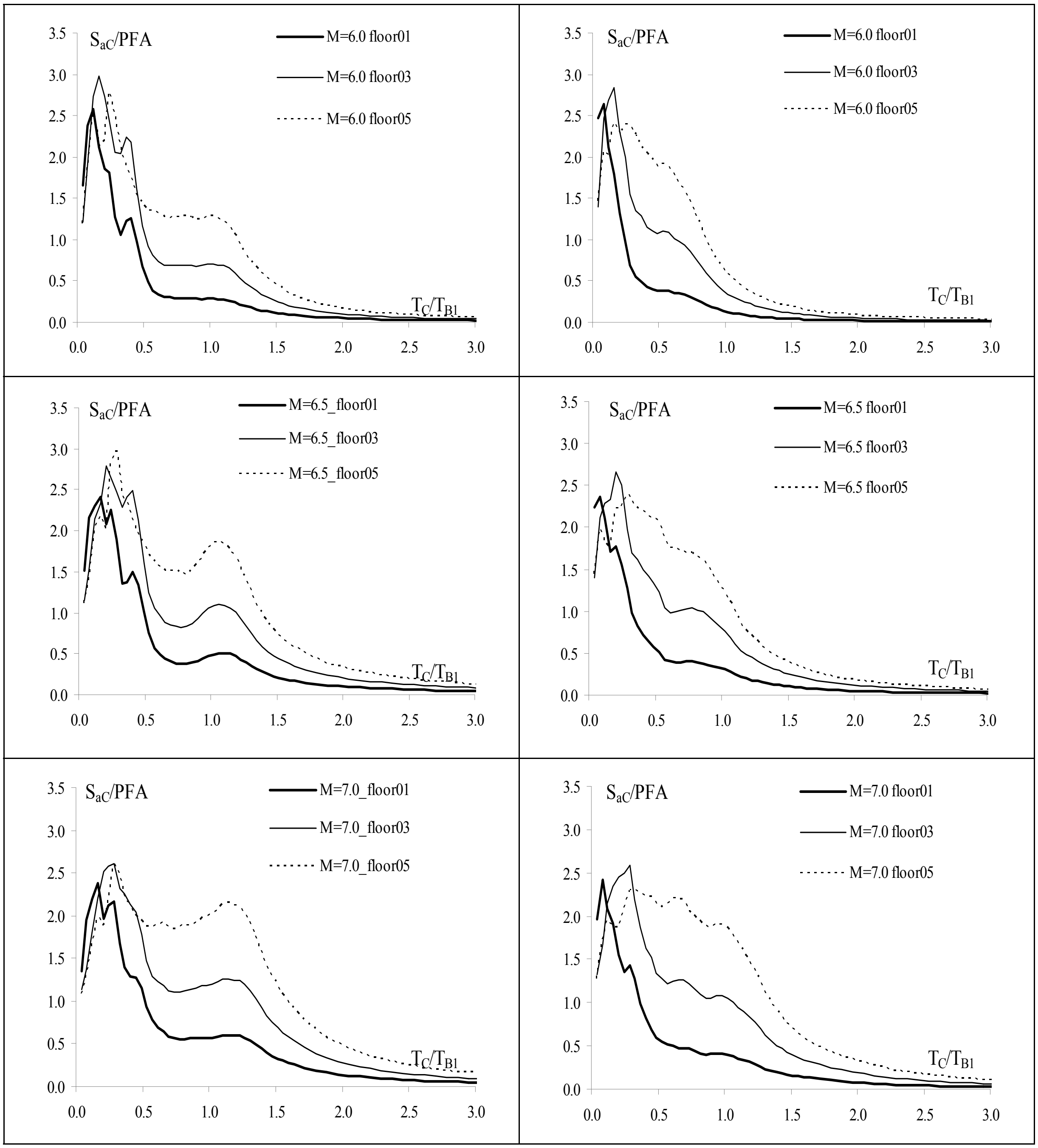

Figure 5. Mean 5%-damped floor acceleration response spectra for the strong (C_y=0.35) 6-storey bare frame (left column) and the strong 6-storey frame with the infill walls (right column).

The floor response spectra for the pilotis case are fairly similar to those for the frame with the infill walls case. The observed similarity is due to the fact that for this strong pilotis (C_y=0.35) the soft story behavior is hardly ever engaged. This is actually confirmed by the plot of Fig. 7, which shows the mean value (with respect to all the input records) of the maximum inter-storey drift demands (IDI) obtained for these two different structures. It can be observed that the IDI profiles are almost the same. Finally, note that the results on the FRS investigations described above, which refer to the studied frames characterized by higher base shear strengths, are representative also of those obtained for the buildings designed for lower seismic forces.

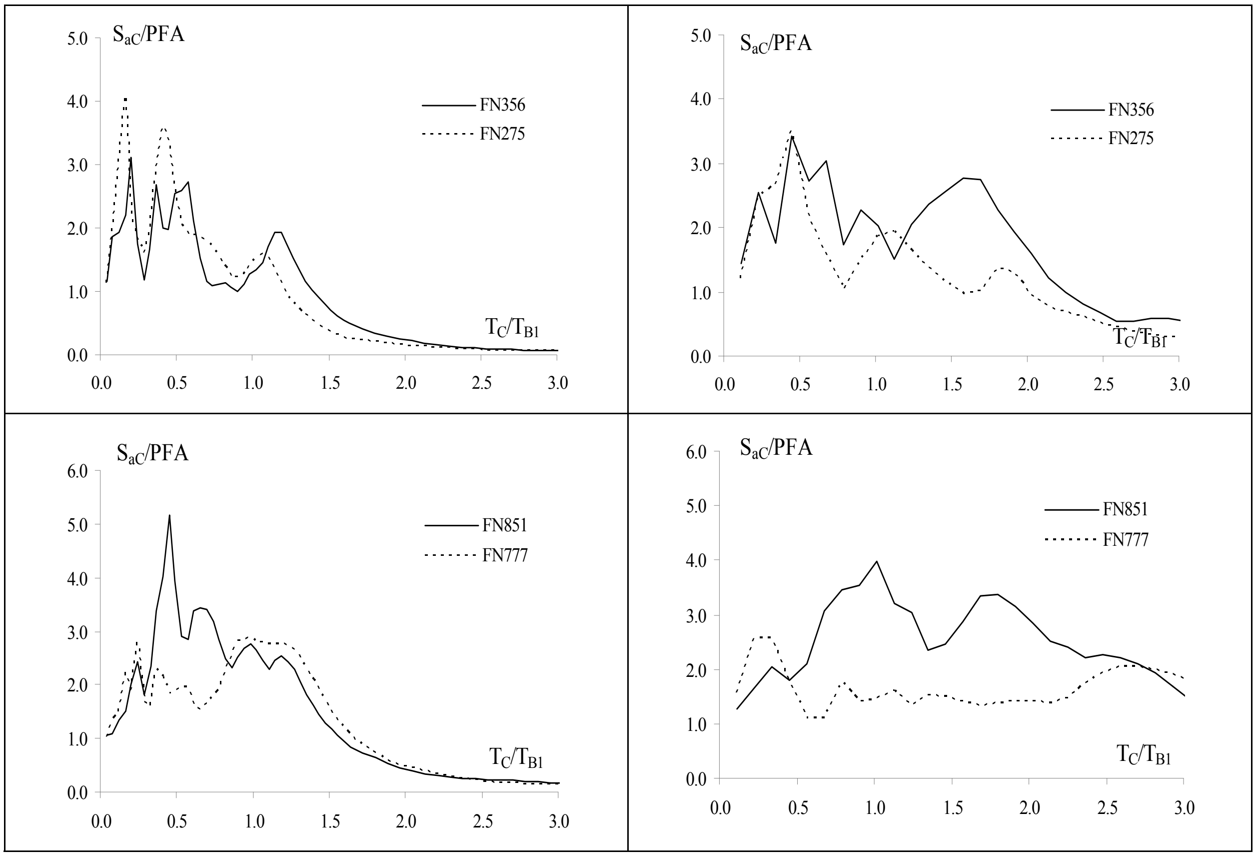

Figure 6. Selected floor acceleration response spectra for the strong (C_y=0.35) 6-storey bare frame (left column) and the strong 6-storey frame with the infill walls (right column).

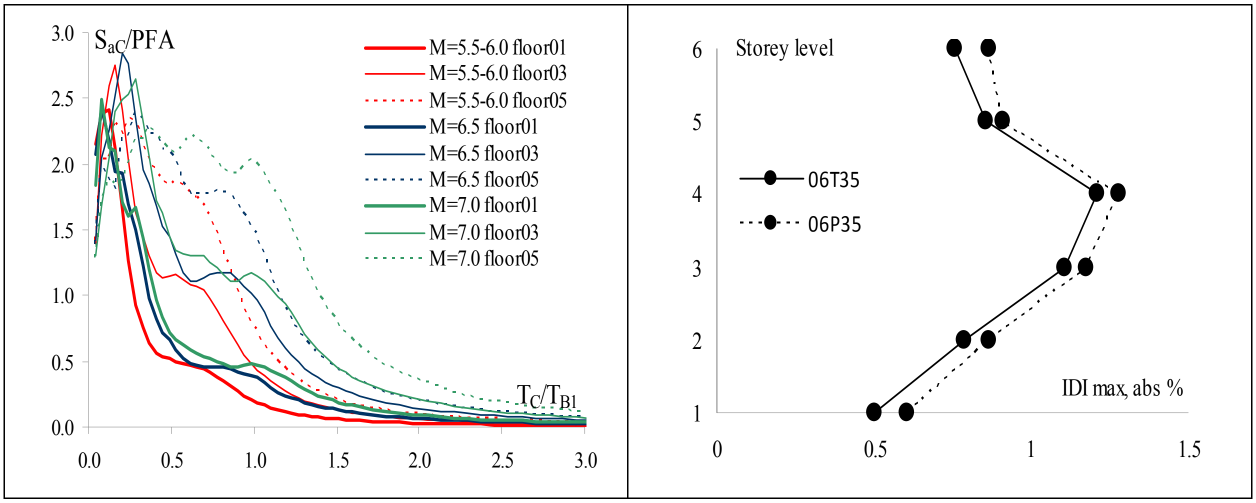

Figure 7. Mean 5%-damped normalized floor acceleration response spectra for the strong (C_y=0.35) 6-storey pilotis structure (left) and comparison between the mean values of the maximum inter-storey drifts for the pilotis structure (dashed line) and the frame with the infill walls (solid line) (right).

Conclusions

This study identifies some interesting peculiarities that characterize the floor acceleration demand on concrete frame structures with masonry infill walls. Investigations carried out on systems with different infill wall configurations have demonstrated that while the peak floor acceleration profiles present only slight differences with those obtained in the corresponding bare frame systems, the shapes of the floor acceleration response spectra (FRS) show more relevant differences. In particular, in the considered cases studied, significant peaks of amplification at the second period T_{B2} of the building only have been observed. In the case of the infill walls, the amplifications at the first mode period T_{B1} become considerable only for the top floor levels and for high-magnitude earthquakes. In these cases, rather than peaks, plateau between T_{B1} and T_{B2} can be observed. With the infill walls, in fact, the FRS evaluated for the single earthquake is frequently characterized by more than one peak. This can be probably explained by the fact that the type and level of damage for such structural system become more sensitive with respect to the bare frame to the properties of the ground motion record. Finally, it has been found that the acceleration peaks, due to the significant loss of stiffness produced in the building by the damage of the infill walls, can occur at T_C/T_{B1} and T_C/T_{B2} values much higher than those usually evaluated in the case of the bare frame structure. For the single earthquake, amplification for T_C/T_B values also greater than 1.5 have been observed.

Acknowledgments

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