

Alma Mater Studiorum Università di Bologna
Archivio istituzionale della ricerca

On the peak inter-storey drift and peak inter-storey velocity profiles for frame structures

This is the final peer-reviewed author's accepted manuscript (postprint) of the following publication:

Published Version:

Palermo, M., Silvestri, S., Trombetti, T. (2017). On the peak inter-storey drift and peak inter-storey velocity profiles for frame structures. SOIL DYNAMICS AND EARTHQUAKE ENGINEERING, 94, 18-34 [10.1016/j.soildyn.2016.12.009].

Availability:

This version is available at: <https://hdl.handle.net/11585/594120> since: 2017-06-12

Published:

DOI: <http://doi.org/10.1016/j.soildyn.2016.12.009>

Terms of use:

Some rights reserved. The terms and conditions for the reuse of this version of the manuscript are specified in the publishing policy. For all terms of use and more information see the publisher's website.

This item was downloaded from IRIS Università di Bologna (<https://cris.unibo.it/>).
When citing, please refer to the published version.

(Article begins on next page)

This is the final peer-reviewed accepted manuscript of:

Palermo, Michele, Stefano Silvestri, and Tomaso Trombetti. "On the Peak Inter-storey Drift and Peak Inter-storey Velocity Profiles for Frame Structures." *Soil Dynamics and Earthquake Engineering* 94 (2017): 18-34.

The final published version is available online at:

<https://doi.org/10.1016/j.soildyn.2016.12.009>

Rights / License:

The terms and conditions for the reuse of this version of the manuscript are specified in the publishing policy. For all terms of use and more information see the publisher's website.

This item was downloaded from IRIS Università di Bologna (<https://cris.unibo.it/>)

When citing, please refer to the published version.

On the peak inter-storey drift and peak inter-storey velocity profiles for frame structures

Michele Palermo ⁽¹⁾, Stefano Silvestri and Tomaso Trombetti

*Department DICAM, University of Bologna, Viale Risorgimento 2, 40136
Bologna, Italy*

ABSTRACT

It is well known that peak inter-storey drifts and peak inter-storey velocities are fundamental response quantities to assess the seismic response of a frame structure equipped with added viscous dampers. In the present work, analytical estimations, grounded on the first mode response, of the two response quantities are introduced. Then, a large parametric study is carried out to assess the effectiveness of the proposed predictions. A special attention is devoted to the peak inter-storey velocities and on their sensitivity on the higher modes contribution.

1. INTRODUCTION

Since the 1980s the peak inter-storey drift profile has been used in earthquake engineering to estimate damages in building structures due to the earthquake ground shaking (Sozen (1983), Moehle (1984), Bozorgnia and Bertero (2001), Akkar et al. (2005)). Iwan (1997), by making use of waves propagation theory through continuum shear beams, introduced the concept of the inter-storey drift spectrum. The main advantage in the use of the inter-storey drift spectrum with respect to the conventional displacement response spectrum is that it accounts for the higher modes contribution, thus resulting in a more precise estimation of the seismic displacement responses. Chopra and Chintanapakdee (2001) showed that the inter-storey drift spectrum may also be computed by classical modal analysis. Few years later, Miranda and Akkar (2006), generalized the inter-storey drift spectrum to

¹ Corresponding author. Phone: +39 051 20 9 3232; Fax: +39 051 20 9 3236; e-mail: michele.palermo7@unibo.it

shear-flexural building behaviour accounting for different beam-to-column stiffness ratios.

All the mentioned studies showed that the predictions of the peak inter-storey drifts based on the first mode response only are, in general, relatively accurate for structures characterized by fundamental periods ranging in the so-called acceleration sensitive spectral region, while they may become quite un-conservative for structures with larger fundamental periods. Nonetheless, despite the potential advantages of the inter-storey drift spectrum, it remained quite confined among the academia and practically unknown to professional engineers. A possible reason is the absence of schematized (code-like) expressions of the inter-storey drift spectrum which could be used by professional engineers in the practical design.

In case of structures equipped with added inter-storey viscous dampers, in addition to the peak inter-storey drifts, also the peak inter-storey velocities play an important role in the evaluation of the structural response, given that the forces in the viscous dampers directly depend on the inter-storey velocities (Christopolous and Filiatrault (2006)). Adachi et al. (2013) recently showed that, for high-rise buildings the peak inter-storey velocities due to earthquake ground motions may become very large, especially at the lower storeys, due to the higher modes effects. In a recent work, the authors (Palermo et al. 2015) introduced the so-called Seismic Modal Contribution Factors (SMCFs) to estimate the contribution of the higher modes to a given response quantity (e.g. inter-storey drifts, inter-storey velocities) due to the seismic input. SMCFs represent an improvement of the well-known Modal Contribution Factors (MCFs, Chopra 2001) which are grounded on the concept of modal static response and do not account for the nature of the dynamic input.

The present paper focuses on the study of peak inter-storey drifts and peak inter-storey velocities developed under seismic excitation in frame structures equipped with inter-storey viscous dampers are presented. The main aim is to assess the effectiveness of simple analytical predictions which could be useful for professional engineers, especially in the preliminary design phase.

2. AN ANALYTICAL ESTIMATION OF PEAK INTER-STOREY DRIFTS AND VELOCITIES

The analytical formulations which are here used for the prediction of peak inter-storey drifts and peak inter-storey velocities developed by multi-storey frame structures during an earthquake are based on an assumed analytical first mode shape and has been first derived by the authors in a recent work (Palermo et al. 2016) with the purpose of simplifying a procedure for the sizing of viscous dampers inserted in frame structures (known as the five-step procedure, Silvestri et al. 2010).

In detail, two analytical first mode shapes (shape A and B) were considered (see Figure 1) even though, in principle, the same procedure can be adopted for a generic first mode shape. For the sake of conciseness, the readers may refer to the work by Palermo et al. (2016) for the derivations. According to the above mentioned procedure, the following analytical estimations ($\delta_{\max,A}^1$, $v_{\max,A}^1$, $\delta_{\max,B}^1$ and $v_{\max,B}^1$) of the maximum actual first-mode peak inter-storey drift (δ_{\max}^1) and peak inter-storey velocity (v_{\max}^1) at the ground floor were derived assuming Type A and Type B profiles.

For Type A profile:

$$\delta_{\max,A}^1 = \frac{12N}{(2+5N+5N^2)} \cdot \frac{S_a(T_1)}{\omega_1^2} \quad (1)$$

$$v_{\max,A}^1 = \frac{12N}{(2+5N+5N^2)} \cdot \frac{S_a(T_1)}{\omega_1} \quad (2)$$

For Type B profile:

$$\delta_{\max,B}^1 = \frac{2}{N+1} \cdot \frac{S_a(T_1)}{\omega_1^2} \quad (3)$$

$$v_{\max,B}^1 = \frac{2}{N+1} \cdot \frac{S_a(T_1)}{\omega_1} \quad (4)$$

where N indicates the total number of storeys, ω_1 is the first circular frequency of the structure, $S_a(T_1)$ is the ordinate of pseudo-acceleration spectrum at the structure fundamental period T_1 . Note that the ratio between the two estimations ($\delta_{\max,A}^1 / \delta_{\max,B}^1$ or equivalently $v_{\max,A}^1 / v_{\max,B}^1$) is between 1.0 and 1.2 (e.g. maximum discrepancies in the order of 20%).

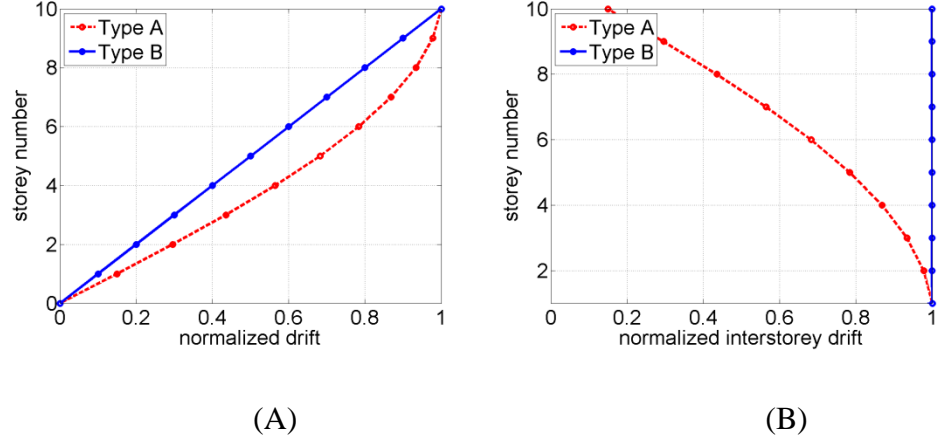


Figure 1: First mode drift profiles (A) and inter-storey drift profiles (B) for Type A and Type B 10-storey structures, normalized to a unit value at the roof level (adapted from Palermo et al. 2016).

3. THE OBJECTIVES OF THE STUDY

With the purpose of verifying the effectiveness of the analytical estimations of the peak inter-storey drift and velocities recently proposed by the authors (Palermo et al. 2016) a comprehensive parametric study has been developed. In more detail:

- A first parametric study (section 4) is developed with reference to idealized structures (i.e. uniform shear type structures) subjected to a reference seismic input (an ensemble of 10 artificial earthquakes with average spectrum compatible with the design spectrum) and is aimed at: (i) obtaining the trends of the response quantities (peak inter-storey drift and velocities), and assess the influence of the key parameters and evaluate their effect on the response quantities and predictions; (ii) comparing the trends obtained from the time-history analyses with the predictions of the peak inter-storey drifts and velocities; (iii) introducing/calibrating correction factors (section 5) to improve the accuracy in the estimation of the peak inter-storey velocity.
- A second parametric study (section 6) is aimed at assessing the influence of the seismic input on the trends of the correction factors of the peak inter-storey velocity.
- A third parametric study (section 7) is aimed at assessing the influence of different along-the-height lateral stiffness distributions and dampers

placements on the trends of the correction factors of the peak inter-storey velocity.

- Four realistic reinforced concrete moment-resisting frames equipped with inter-storey viscous dampers are analysed in section 8 and the proposed predictions are compared with those suggested in the ASCE7-10 building code.

4. THE PARAMETRIC STUDY ON THE UNIFORM FRAME STRUCTURES

4.1. The uniform shear-type structures

Shear-type structures with uniform mass and stiffness distribution (along the height) and equipped with uniform (along the height) inter-storey viscous dampers are the frame structures investigated in this section.

The frame structures are characterized by:

- uniform floor mass $m = 100$ ton;
- uniform storey stiffness $\rho \cdot k$ (with $k=1000000$ kN/m and); ρ values are set equal to 0.5, 1.0, 2.0 and 5.0 in order to cover a wide range of fundamental periods T_1 between 0.1 s to 5.0 s;
- storey damping coefficient c leading to damping ratios $\xi = 0.05, 0.15, 0.3$ according to the five-step procedure by Silvestri et al. (2010):

$$c = \xi \cdot \omega_1 \cdot m_{tot} \cdot (N + 1) \quad (5)$$

- total number of storeys N equal to 5, 10, 20, 30.

A total number of 48 frame structures have been analysed.

4.2. The synthetic spectrum compatible ground motions ensemble

An ensemble of 10 artificial ground motions generated using the software SIMQKE (Vanmarcke et al. (1990)) is here considered as the reference seismic input. The 10 artificial ground motions were generated in order to obtain a mean pseudo-acceleration spectrum compatible with the elastic design spectrum at 5% damping ratio according to the Italian building code (NTC08 (2008)). An average PGA equal to 0.25g, soil type B and topographic condition T1 have been considered. The

average response spectra (for damping ratios equal to 5% and 30%) of the 10 artificial records are displayed in Figure 2.

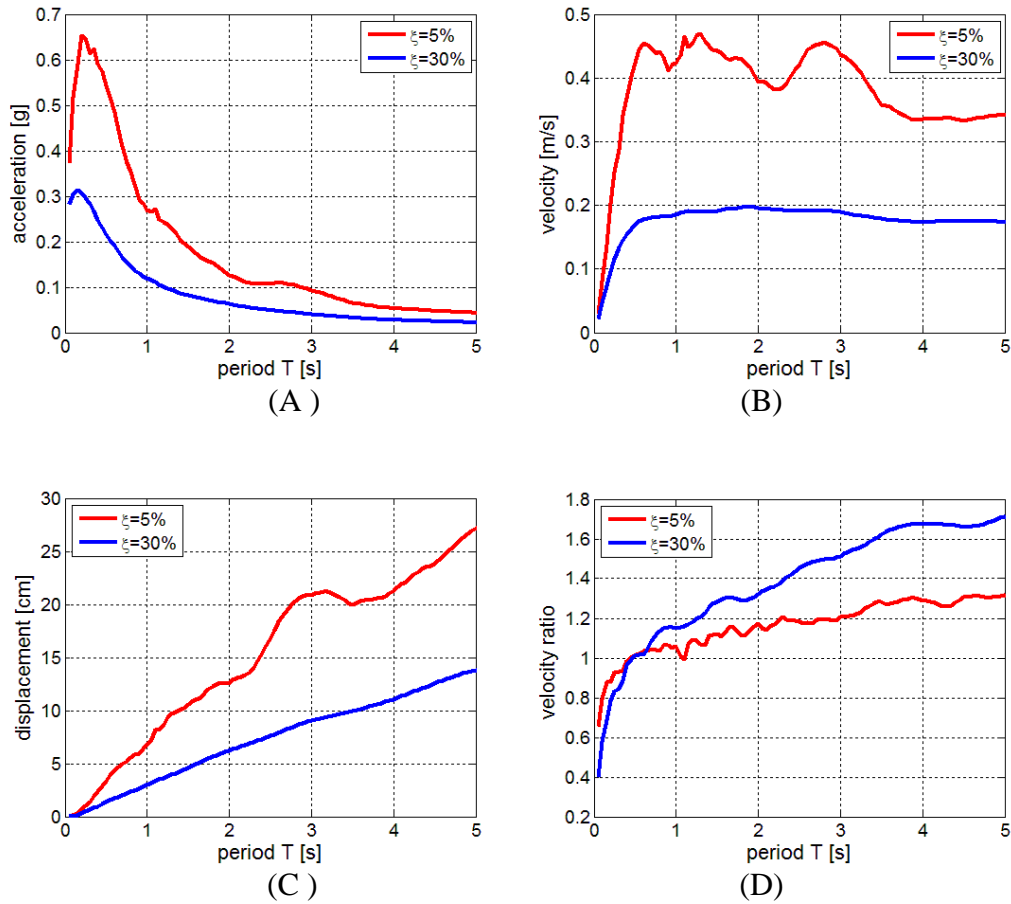


Figure 2: Mean spectra of the 10 artificial ground motions: (A) pseudo-acceleration spectrum; (B) pseudo-velocity spectrum; (C) displacement spectrum (D) velocity/pseudo-velocity spectrum

4.3. Total response history analyses and first modal response analyses

Linear time-history analyses have been developed:

- by direct integration of the equations of motion using the unconditionally stable Newmark method (Hilber et al. (1977)). These analyses will be hereafter also referred to as Response History Analyses, RHA (Chopra (2001)); the related structural response will be also referred to as “total response”.
- by integration based upon the response of the first mode of vibration only. These analyses will be hereafter referred to as 1st Modal Response Analysis,

1-MRA (Chopra, 2001); the related structural response will be also referred to as “first mode response”.

A total number of 1200 time-history simulations has been conducted.

The comparison between total and first mode responses allows to directly quantify the contribution of the higher modes.

In order to reduce the computational time, the numerical simulations and the post process have been developed through a specific MATLAB code implemented directly by the authors. The code has been validated by comparisons with SAP2000 (v 16). For instance, the total and first mode displacement responses at the first floor of a 5-storey frame structure with added viscous dampers are displayed in Figure 3.

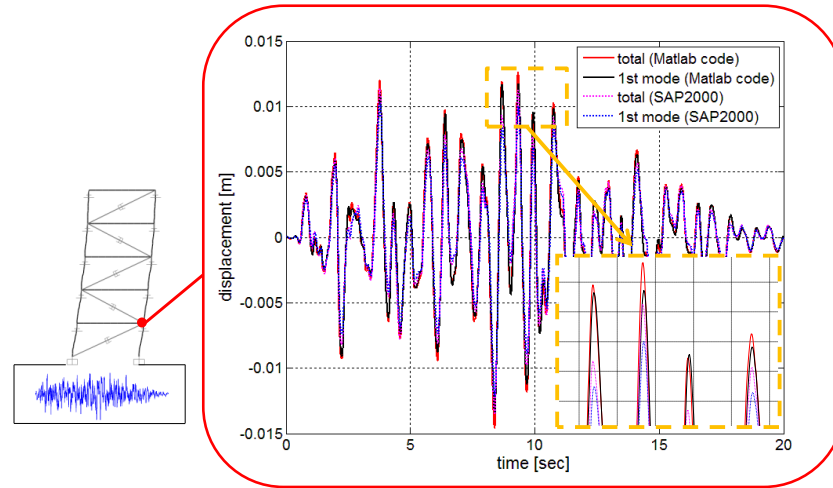


Figure 3: Total and first mode time-history response of a 5-storey structure with added dampers: SAP2000 vs Matlab code.

4.4. Main response quantities

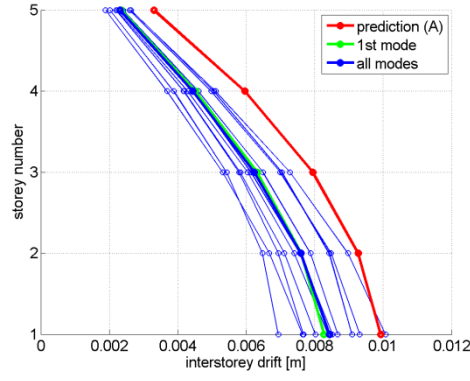
In the next subsections the results of the numerical simulations in terms of both maximum recorded (e.g. peak) inter-storey drifts and inter-storey velocities are analysed and compared to the predictions derived in section 2.

4.4.1. Peak inter-storey drift profiles

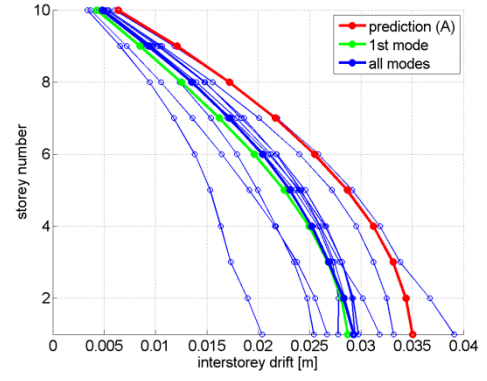
Figures 4 and 5 compare the total and first mode average (over the 10 input) peak inter-storey drift profiles with the corresponding average Type A predictions for a 5% and 30% damping ratio, respectively. Four cases are shown:

- The 5-storey structure with the shortest natural period ($T_1=0.3s$) ;
- The 10-storey structure with natural period $T_1=1.3s$
- The 10-storey structure with natural period $T_1=1.3s$

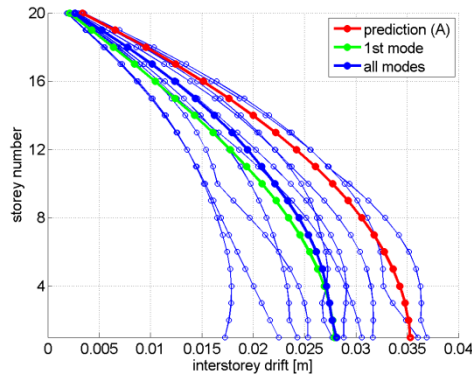
- The 20-storey structure with natural period $T_1=2.6\text{s}$
- The 30-storey structure with the largest natural period ($T_1=5.5\text{s}$).



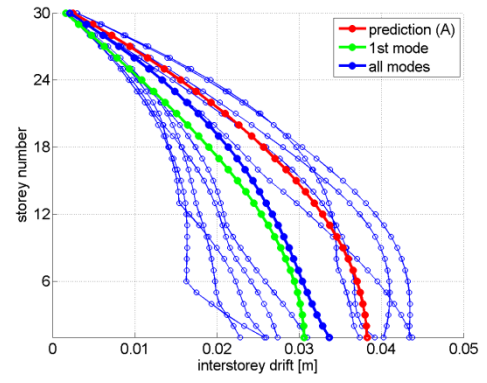
(A)



(B)

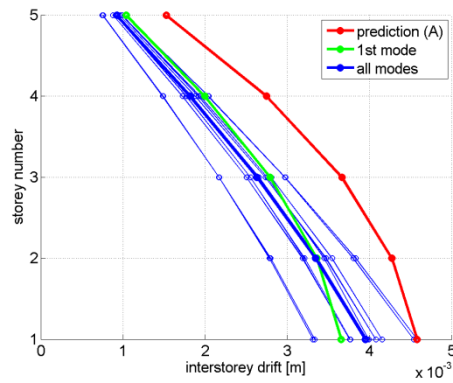


(C)

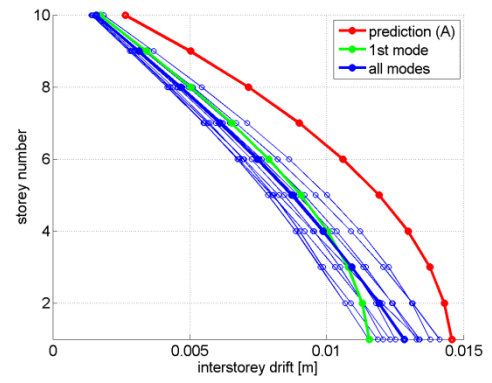


(D)

Figure 4: Peak inter-storey drift profiles for the 5% damped structures: (A) 5-storey structure; (B) 10-storey structure; (C) 20-storey structure; (D) 30-storey structure.



(A)



(B)

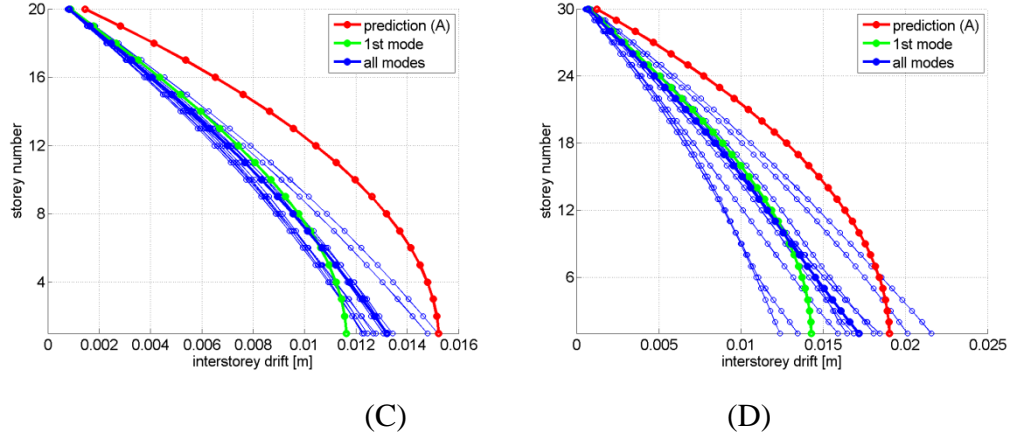


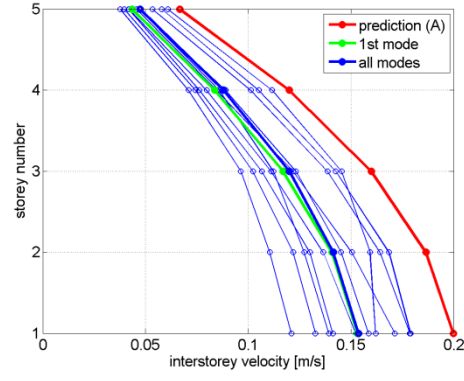
Figure 5: Peak inter-storey drift profiles for the 30% damped structures: (A) 5-storey structure; (B) 10-storey structure; (C) 20-storey structure; (D) 30-storey structure.

The following observations can be made:

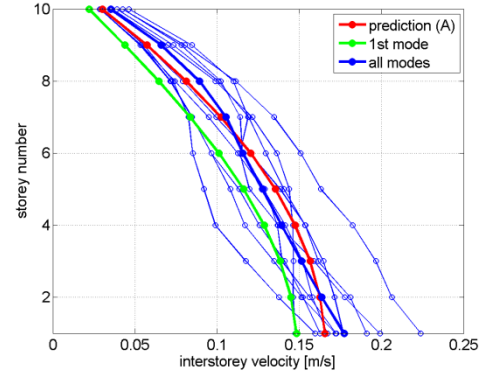
- As expected, on average, the maximum displacement responses of the 30% damped structures are sensibly reduced (around 50% less) with respect to the corresponding 5% damped structures.
- The displacement response of the 30% damped structures exhibit a reduced variability with respect to the response of the 5% damped structures.
- On average, the average first mode response is quite close to the corresponding total response, meaning that the higher modes have a reduced contribution. Minor higher modes effects (less than 10%) are observed for the 30-storey structures.
- On average, the predictions are generally conservative with the maximum overestimations (around 30%) observed at the bottom storeys.

4.4.2. Peak inter-storey velocity profiles

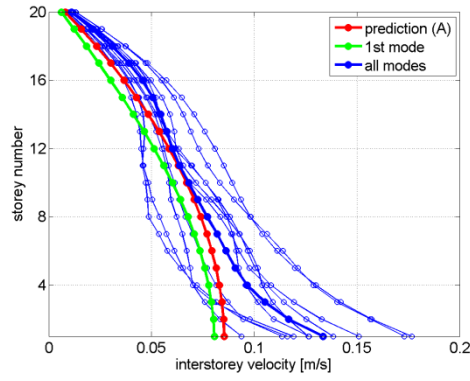
Figures 6 and 7 compare the average total and first mode (over the 10 input) peak inter-storey velocity profiles with the corresponding average Type A predictions for a 5% and 30% damping ratio, respectively. The same frame structures encompassed in Figures 4 and 5 are considered.



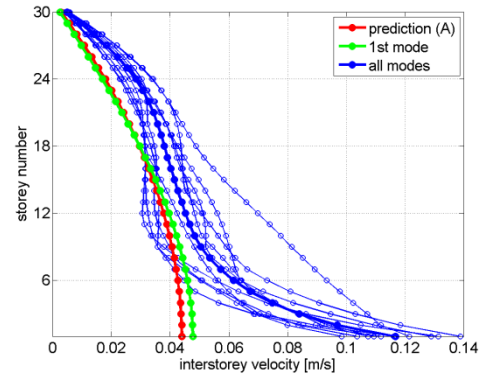
(A)



(B)

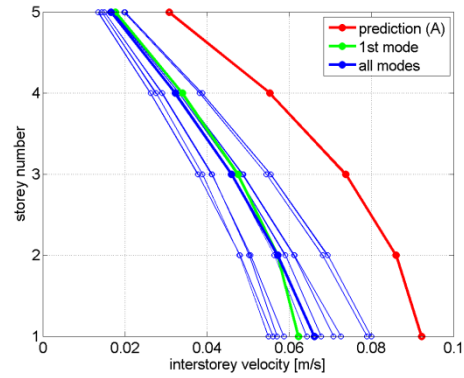


(C)

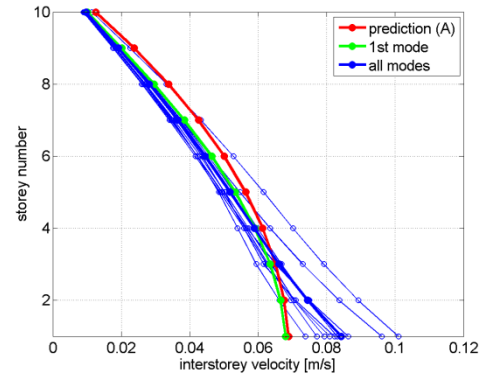


(D)

Figure 6: Peak inter-storey velocity profiles for the 5% damped structures: (A) 5-storey structure; (B) 10-storey structure; (C) 20-storey structure; (D) 30-storey structure.



(A)



(B)

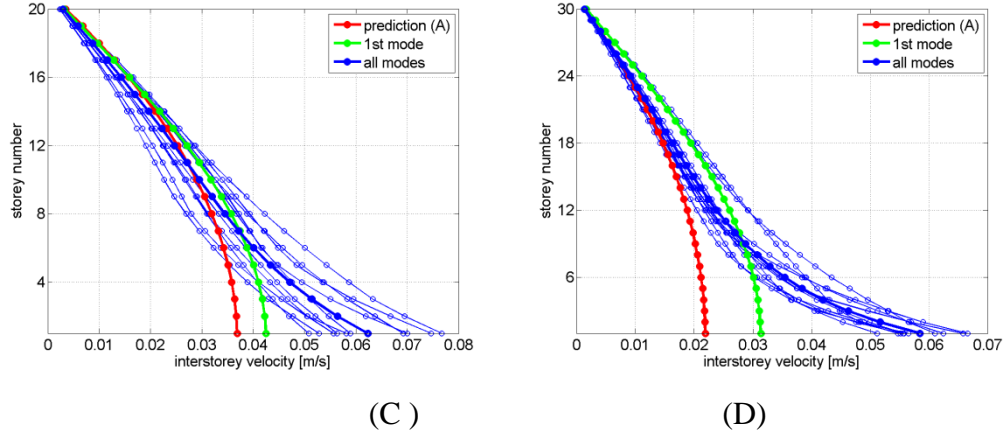


Figure 7: Peak inter-storey velocity profiles for the 5% damped structures: (A) 5-storey structure; (B) 10-storey structure; (C) 20-storey structure; (D) 30-storey structure.

The following observations can be made:

- Again, increasing the amount of damping ratio from 5% to 30% a significant reduction of the maximum velocity responses is observed (around 50%).
- For the five-storey structure the trends of total response, first mode response, and predictions (in terms of peak inter-storey velocities) are similar to those observed with reference to the displacement response:
 - on average, the total response is close to the first mode response;
 - on average, the average predictions are slightly conservative.
- From the 10-storey structures up to the 30-storey structures, the average total inter-storey velocity profile gets farther and farther with respect to both the average first mode inter-storey velocity profile and the average prediction, especially at the bottom storeys.
- At the bottom storeys of the 30-storey structures, the average total inter-storey velocities may become even 3 times larger than the average predictions, which become also smaller than the average first mode inter-storey velocities.

4.5. Total vs first mode response

Figure 7 shows the ratios between the average total and 1st mode responses (both inter-storey drifts and inter-storey velocities) for a 5-storey ($T_1=0.7$ s), 10-storey ($T_1=1.3$ s), 20-storey ($T_1=2.6$ s) and 30-storey ($T_1=3.9$ s) structure with 5% damping ratio.

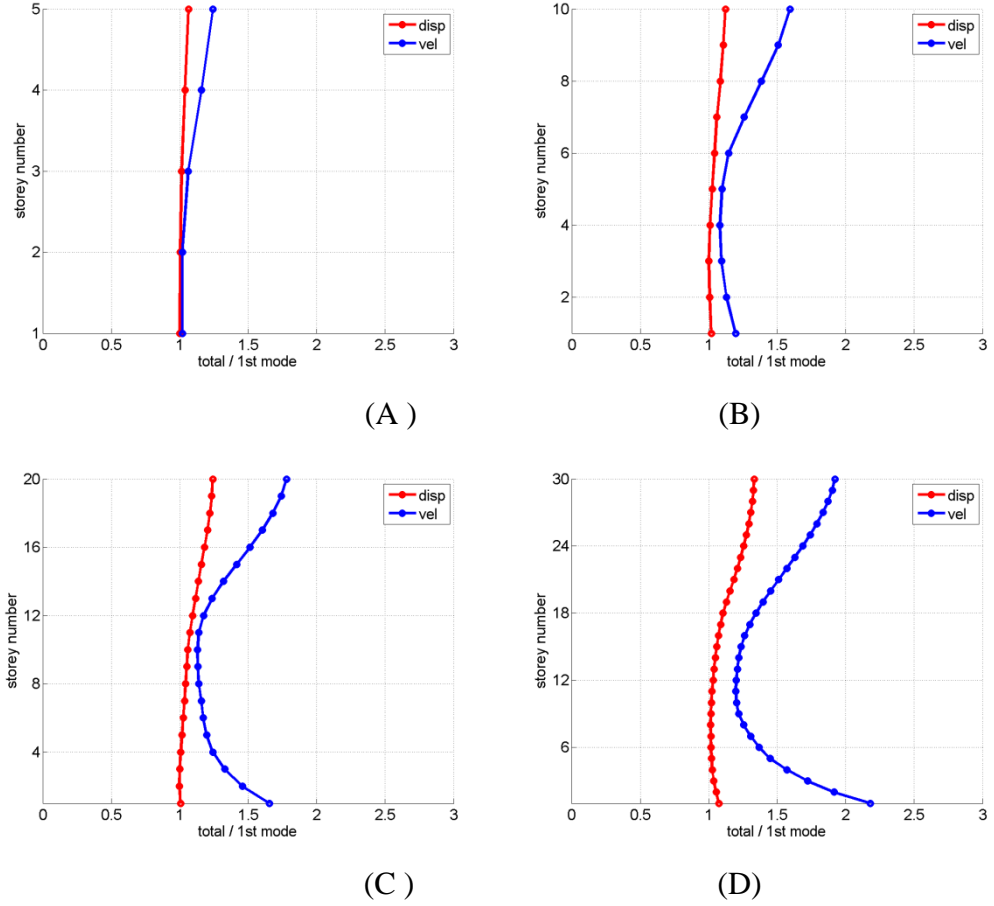


Figure 7: Total / 1st mode ratios of peak inter-storey drifts and velocities (5% damped structures): (A) 5-storey structure; (B) 10-storey structure. (C) 20-storey structure. (D) 30-storey structure.

It can be noted that, for the peak inter-storey drifts, the maximum ratios are limited to values less than 1.5 and are achieved at the upper stories. On the contrary, for the peak inter-storey velocities, large values of the ratios are achieved at both the higher and the lower stories and, for the 30-storey structure ($T_1=3.9$ s), they can become even larger than 2.5 at the bottom storey.

4.6. Remarks

The fundamental results of the first parametric study may be summarized as follows:

- The contribution of the higher modes to the peak inter-storey drifts appears not significant (less than 10%) for all the analysed structures (with

fundamental periods up to 5.0 s). This finding is in line with other results available in the scientific literature (see Chopra (2001)).

- The proposed analytical estimation of the peak inter-storey drifts is, on average, quite accurate within the entire investigated period range (generally conservative) and adequate for design purposes.
- On the contrary, the contribution of the higher modes to the peak inter-storey velocities is in general not negligible and becomes fundamental for structures with first periods larger than 1.0 s.
- The predictions of the peak inter-storey velocities become quite un-conservative (especially at the bottom stories) for buildings with fundamental periods larger than 1.0 s and correction factors need to be introduced. Similar results were also obtained by Adachi et al. 2013.

5. CORRECTION FACTORS FOR THE PEAK INTER-STOREY VELOCITIES

5.1. Correction factors for the actual peak inter-storey velocity at the ground floor

In the present section the attention is devoted to the peak inter-storey velocity at the first inter-storey, where, according to the results of the previous section, the largest magnifications are expected. With specific reference to the peak responses at the ground floor, a further investigation on the trends and correlations between total peak inter-storey velocity (v_{\max}^{tot}), first mode peak inter-storey velocity (v_{\max}^1) and the predictions according to the assumed first mode shapes ($v_{\max,A}^1$ or, alternatively, $v_{\max,B}^1$) is here conducted. First of all, the three quantities of above (v_{\max}^{tot} , v_{\max}^1 and $v_{\max,A}^1$ or, alternatively, $v_{\max,B}^1$) may be related to each other through the following relation:

$$v_{\max}^{tot} = \frac{v_{\max}^{tot}}{v_{\max}^1} \cdot \frac{v_{\max}^1}{v_{\max,A}^1} \cdot v_{\max,A}^1 = H \cdot L \cdot v_{\max,A}^1 = M \cdot v_{\max,A}^1 \quad (6)$$

where $H = \frac{v_{\max}^{tot}}{v_{\max}^1}$ can be interpreted as the correction factor accounting for the higher

modes contribution (it does not depend on the prediction), $L = \frac{v_{\max}^1}{v_{\max,A}^1}$ is the

correction factor accounting for the discrepancies between the actual first mode velocities and predictions. $M = H \cdot L = \frac{v_{\max}^{tot}}{v_{\max,A}^1}$ is the overall correction factor that

has to be applied to the predictions in order to directly obtain the actual total velocities. The results of the numerical simulations shown in the previous section are used to obtain the trends of the correction factors above introduced.

Figure 8 displays the average values of H and L for the 5% and 30% damped structures as a function of the fundamental period T_1 . The H factor (which quantifies the higher modes contribution) tends to linearly increase as the fundamental period increases. On the contrary, it tends to decrease as the damping ratio increases. Values of H at $T_1=5$ s are around 2.5 for $\xi=5\%$ and around 1.8 for $\xi=30\%$. The L factor follows the trend of the average velocity/pseudo-velocity spectra (Figure 2D): for $T_1 < 0.5$ s the values of L are less than 1; for $T_1 > 0.5$ s values of L tend to become larger than 1.0; for increasing damping ratios the values of L tend to increase. The discrepancies between the velocity and pseudo-velocity spectra were well described already in the work by Pekcan et al. (1999).

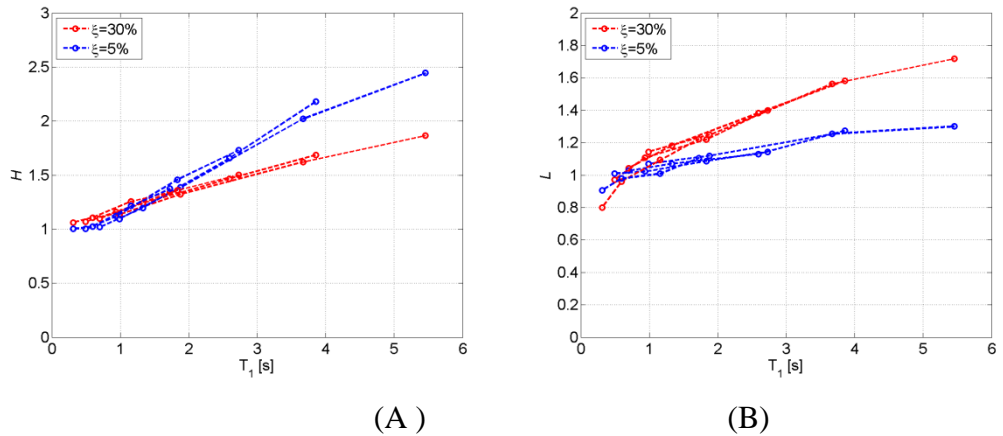


Figure 8: Average correction factors for the 5% and 30% damped structures: (A) H factor; (B) L factor.

5.2. An estimation of the actual peak inter-storey velocity at the ground floor

Figure 9 displays the average values of correction factor M for the 5% and 30% damped structures as a function of the fundamental period T_1 together with its linear regression (Least Square Fit):

$$M = \begin{cases} 1.0 & \text{for } T_1 \leq 0.5s \\ 0.44 \cdot T_1 + 0.78 & \text{for } 0.5 < T_1 \leq 5.0s \end{cases} \quad (7)$$

Note that the authors derived a similar expression of M (Eq. 16 of Palermo et al. 2016) based on a different set of ground motions (50 real records selected from the PEER database). The two linear fits of M lead to discrepancies in the order of 20% at a period of 5.0 s, thus meaning that the average characteristics of the ground motions ensemble may leads to different amplifications of the peak inter-storey velocities (i.e. different higher modes contribution).

If the analytical expression of Eq. 7 is used to evaluate the global correction factor M , the following analytical prediction of the actual (i.e. total, therefore accounting for the higher modes contributions) peak inter-storey velocity at the ground floor is obtained:

$$v_{\max,A}^{tot} = \begin{cases} \frac{12N}{(2+5N+5N^2)} \cdot \frac{S_a(T_1)}{\omega_1} & \text{for } T_1 \leq 0.5s \\ (0.44 \cdot T_1 + 0.78) \cdot \frac{12N}{(2+5N+5N^2)} \cdot \frac{S_a(T_1)}{\omega_1} & \text{for } 0.5 < T_1 \leq 5.0s \end{cases} \quad (8)$$

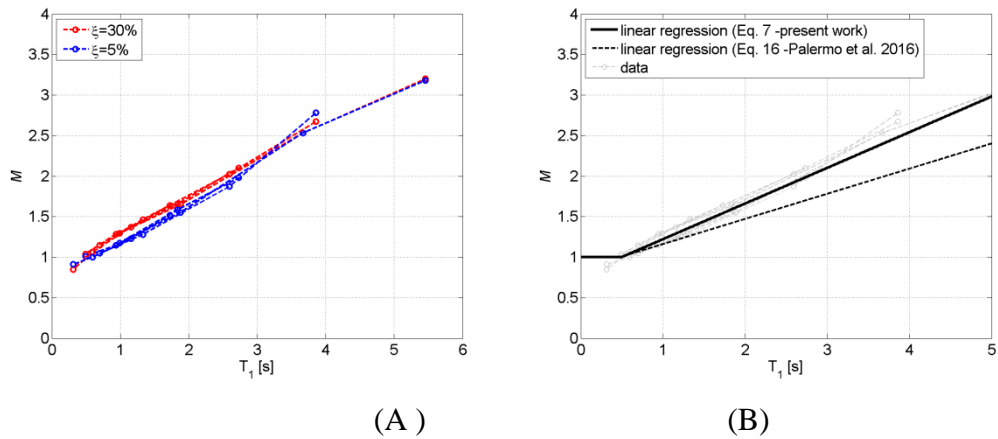


Figure 9: Average M correction factors for the 5% and 30% damped structures: (A) values from the numerical simulations; (B) linear regression lines.

6. ON THE INFLUENCE OF THE SEISMIC INPUT

6.1. G1 and G2 real ground motions ensembles

It is well known that the characteristics of the seismic input may significantly affect the dynamic response of building structures. In a recent work aimed at an engineering characterization of earthquake ground motions (Palermo et al. (2014)), the authors highlighted that the ground motion at a specific site can be adequately described in terms of the three peak ground parameters (namely the peak ground acceleration, PGA, the peak ground velocity, PGV, and peak ground displacement, PGD). As such, a ground motion selection criterion based on the expected PGA, PGV and PGD at the site (as obtained from site hazard analysis) would allow the analyst to obtain an ensemble of ground motions representative of the expected earthquake at the site (i.e. characterized by an average response spectrum characteristic of the site). This also means that ground motions ensembles with similar average PGA, but quite different average values of PGV (and/or PGD) are characterized by quite different frequency contents resulting in different spectral shapes.

In the light of these observations, in the present section, the influence of the earthquake input on the correction factors is investigated by considering two additional ground motions ensembles (the same considered in the work by Palermo et al. (2014), and referred to as G1 and G2). Each ground motion ensemble is composed by real seismic records (selected from the PEER strong motion database database) with an average PGA of about 0.25 g. In more detail:

- G1 ensemble consists of 10 records with PGV between 15 and 31 cm/s (with an average of 23 cm/s) and PGD between 8 and 12 cm (with an average of 10 cm).
- G2 ensemble consists of 10 records with PGV between 40 and 70 cm/s (with an average of 48 cm/s) and PGD between 30 and 40 cm (with an average of 34 cm).

The average (5% and 30 % damping ratio) pseudo-acceleration, pseudo-velocity, displacement, velocity and velocity/pseudo-velocity response spectra of G1 and G2 ensembles are displayed in Figures 10 and 11, respectively.

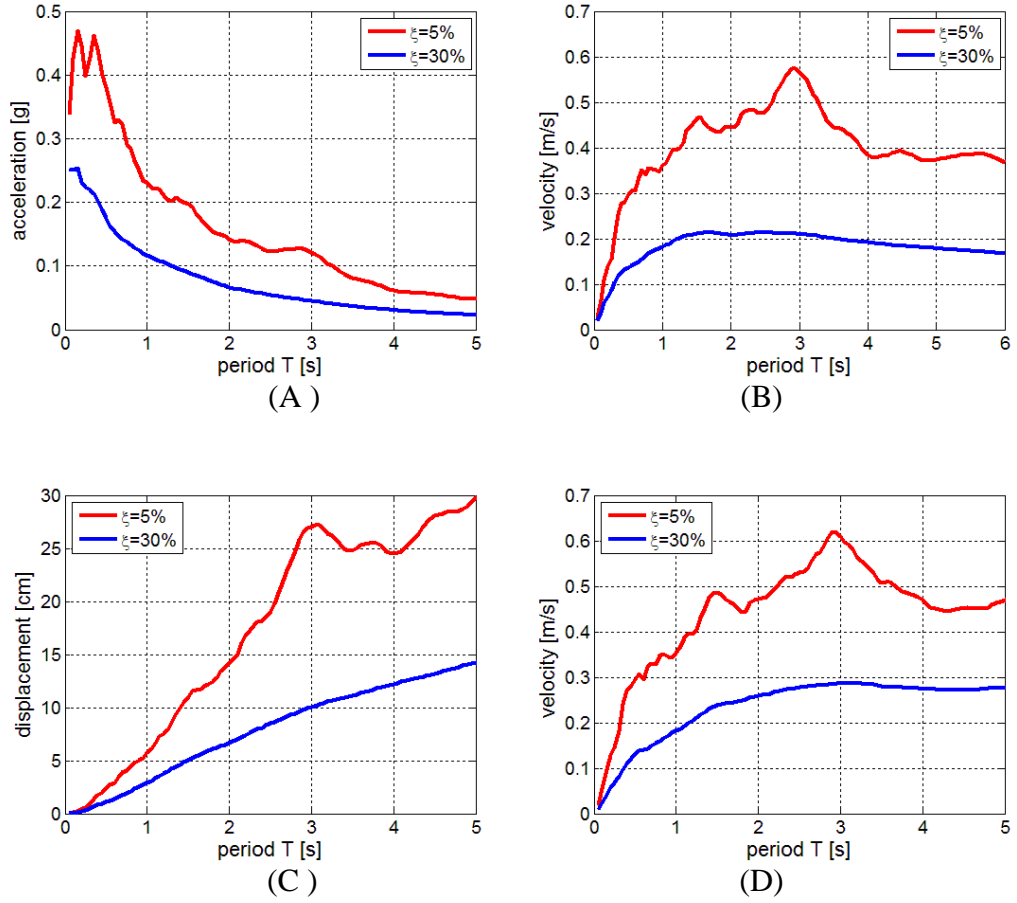
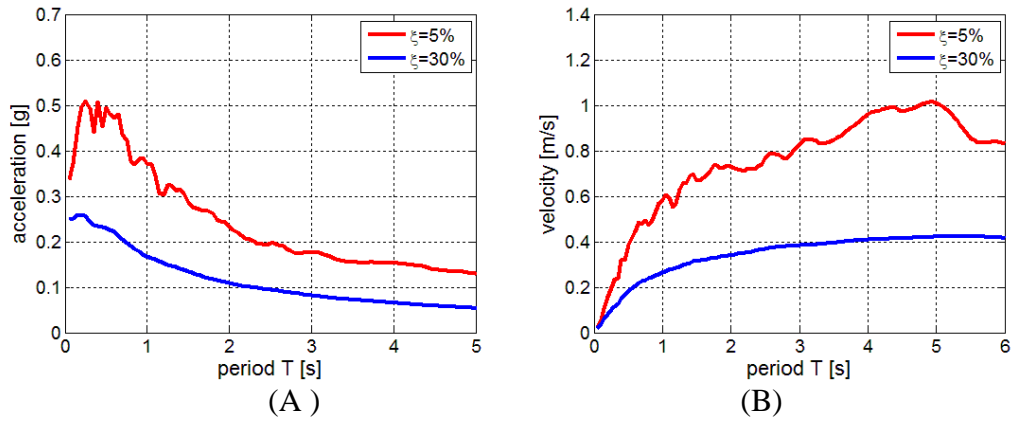


Figure 10: Mean spectra of the 10 ground motions of G1 ensemble: (A) pseudo-acceleration spectrum; (B) pseudo-velocity spectrum; (C) displacement spectrum (D) velocity spectrum.



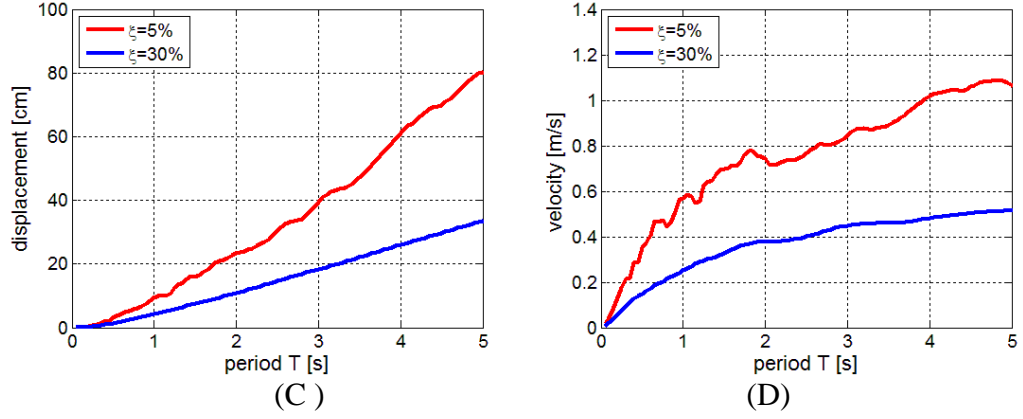
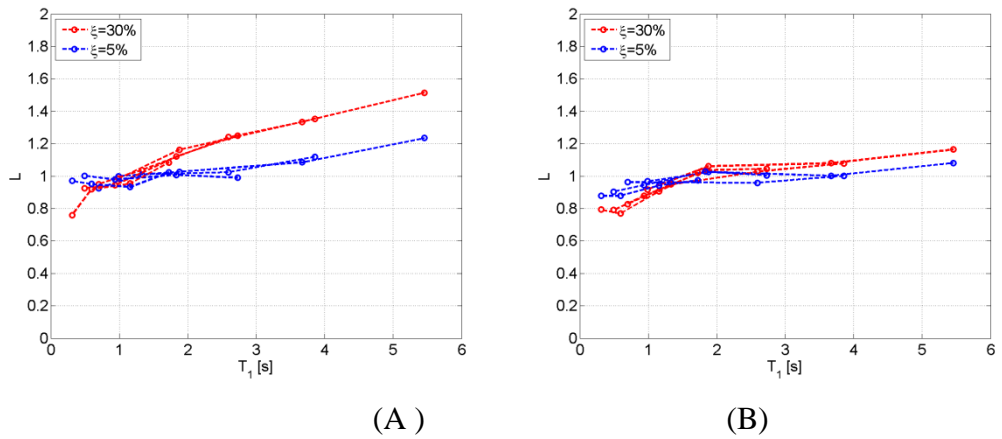
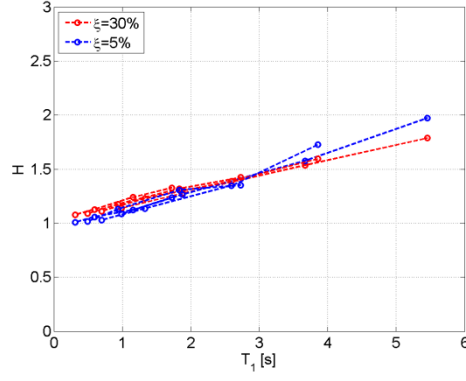


Figure 11: Mean spectra of the 10 ground motions of G2 ensemble : (A) pseudo-acceleration spectrum; (B) pseudo-velocity spectrum; (C) displacement spectrum (D) velocity spectrum.

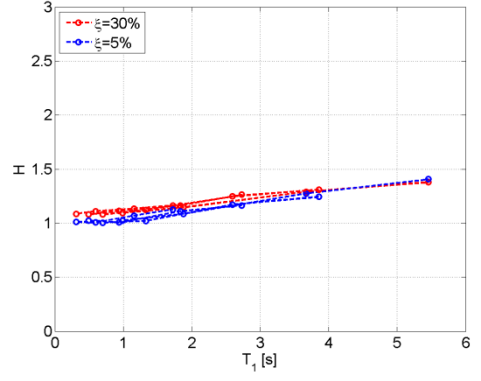
6.2. Correction factors for real ground motions ensembles

Figure 12 compares the average values of factors L , H and M (for 5% and 30% damping ratios) as obtained from the numerical simulations considering both G1 and G2 ground motions ensembles. Their trends and mutual discrepancies can clearly be related to the differences in the average spectral shapes, which are shown in Figure 13. In detail, Figure 13A displays the ratios between the average pseudo-acceleration spectra of G1 and G2 ground motions ensembles, while Figure 13B displays the ratio of the average velocity over pseudo-velocity spectrum for G1 and G2 ground motions ensembles.

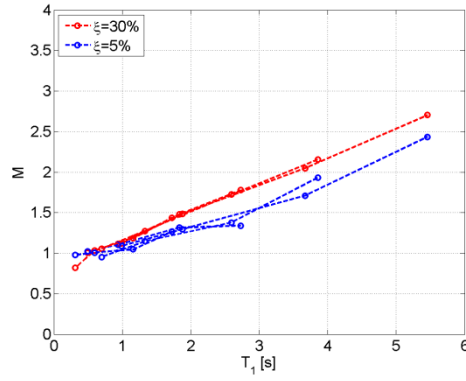




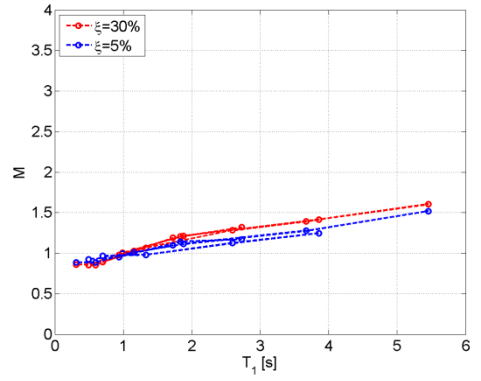
(C)



(D)

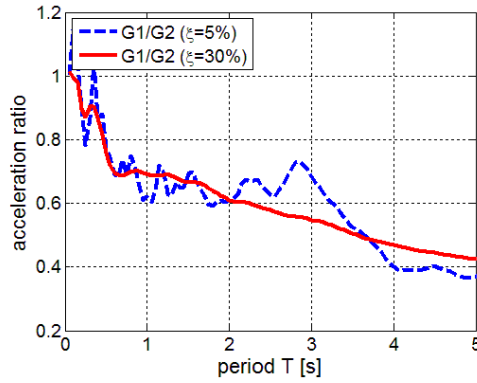


(E)

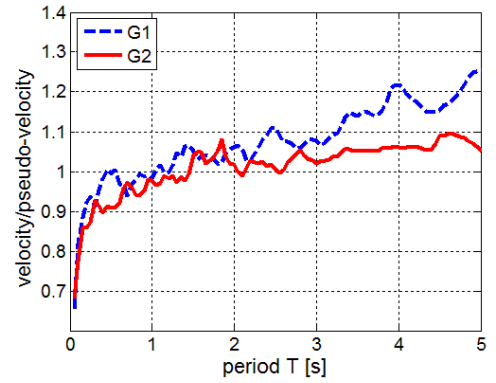


(F)

Figure 12: Mean values of the correction factors L , H and M for 5% and 30% damped structures: (A) L factor for G1 ensemble; (B) L factor for G2 ensemble; (A) H factor for G1 ensemble; (A) H factor for G2 ensemble; (A) M factor for G1 ensemble; (A) M factor for G2 ensemble.



(A)



(B)

Figure 13: (A) ratio of the average pseudo-acceleration spectra between G1 and G2 ground motion ensembles; (B) ratio of the average velocity over pseudo-velocity spectrum.

Inspection of Figures 12 and 13 leads to the following observations:

- Values of the L factor tend to be larger for G1 with respect to G2 and follows the shape of velocity/pseudo-velocity spectrum (Figure 16B);
- Values of the H factor are quite larger for G1 with respect to G2. This result is due to the fact that the amplitudes of the average pseudo-acceleration spectrum of G1 decreases faster than the amplitudes of the average pseudo-acceleration spectrum of G2 (Figure 16A);
- As a result, the M factors as obtained from G1 ensemble are quite larger than those obtained from G2 ensemble.

7. ON THE INFLUENCE OF THE LATERAL STIFFNESS DISTRIBUTION AND DAMPERS PLACEMENT

7.1. The parametric study

In this section the influence of the along-the-height lateral stiffness distribution and damper placement on the peak velocity profiles and correction factors is investigated. The lateral stiffness k_i (shear-type behaviour is considered) at the i -th storey is assumed to gradually decrease from the bottom to the top:

$$k_i = \lambda \cdot \frac{N(N+1) - i(i-1)}{2} \quad (9)$$

It can be shown that such stiffness variation leads to an almost linear first mode shape (λ is a constant given that Eq. 9 represents a shape).

The other main properties of the analysed structures are:

- uniform floor mass $m = 100$ ton;
- lateral stiffness values equal to ρk_i ; values of λ are set in in order to obtain structures with fundamental periods close to those of the corresponding reference structure (i.e. the structure with same number of storeys); values of k_i for the case of $\rho = 1.0$ are in Table 1. Four different ρ values have been considered (0.5, 1.0, 2.0 and 5.0).
- total damping coefficient c_{tot} leading to three different damping ratios $\xi = 0.05, 0.15, 0.30$ according to the five-step procedure by Silvestri et al. (2010).

- total number of storeys N equal to 5, 10, 20 and 30;
- two dampers placement:
 - Uniform damper placement (UD), i.e. dampers with equal damping coefficient at all storeys,
 - Stiffness proportional damper placement (SPD), i.e. dampers with damping coefficient proportional to the lateral stiffness.

Both RHA and 1-MRA have been carried out using the reference input of section 3.2. A total number of 1920 (4x3x4x2x2x10) time-history simulations have been performed.

It should be noted that for such class of structures, characterized by increasing lateral stiffness going from the top stories to the ground, Type B predictions should be more accurate than Type A.

Table 1: k_i values $\rho=1.0$.

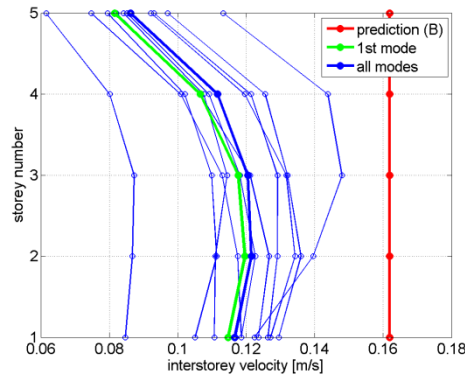
storey	$N = 5$	$N = 10$	$N = 20$	$N = 30$
1	2608203	2790622	2891556	2926826
2	2332847	2647417	2818340	2877632
3	2020305	2496008	2743171	2827583
4	1649572	2334802	2665883	2776631
5	1166424	2161607	2586286	2724727
6	/	1973268	2504161	2671814
7	/	1764944	2419249	2617833
8	/	1528487	2331247	2562714
9	/	1248004	2239789	2506384
10	/	882472	2144435	2448758
11	/	/	2044639	2389743
12	/	/	1939715	2329234
13	/	/	1828780	2267110
14	/	/	1710667	2203235
15	/	/	1583770	2137452
16	/	/	1445778	2069579
17	/	/	1293143	1999403
18	/	/	1119895	1926673
19	/	/	914390	1851087
20	/	/	646572	1772281
21	/	/	/	1689804
22	/	/	/	1603089
23	/	/	/	1511406
24	/	/	/	1413791

25	/	/	/	1308916
26	/	/	/	1194872
27	/	/	/	1068726
28	/	/	/	925544
29	/	/	/	755703
30	/	/	/	534363

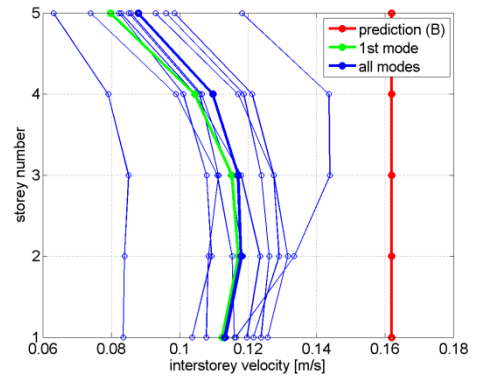
7.2. Peak inter-storey velocity profiles

Figures 14 and 15 compare the average (over the 10 seismic records) total and first mode peak inter-storey velocity profiles as obtained from the numerical simulations with the Type B predictions. The following cases are encompassed:

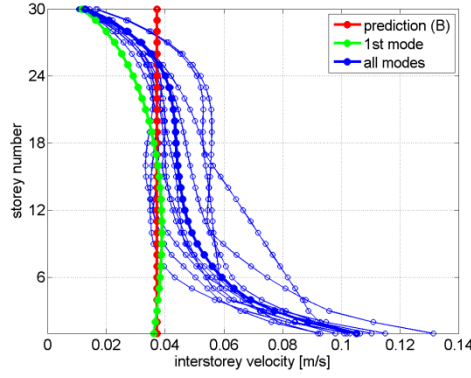
- the 5-storey structure ($T_1=0.30$ s) with UD placement (5% and 30% damped structures, see Fig. 14A and 15A, respectively);
- the 5-storey structure ($T_1=0.30$ s) with SPD placement (5% and 30% damped structures, see Fig. 14B and 15B, respectively);
- the 30-storey structure ($T_1=5.05$ s) with UD placement (5% and 30% damped structures, see Fig. 14C and 15C, respectively);
- the 30-storey structure ($T_1=5.05$ s) with SPD placement (5% and 30% damped structures, see Fig. 14D and 15D, respectively);



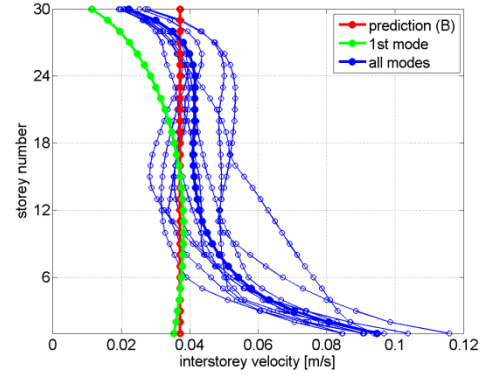
(A)



(B)

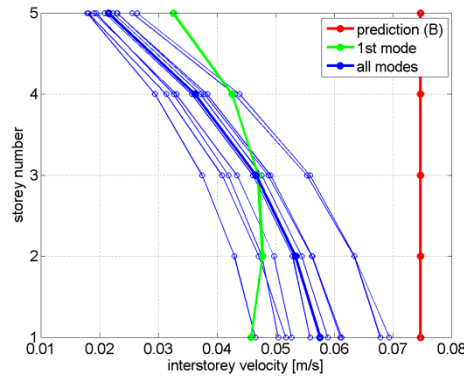


(C)

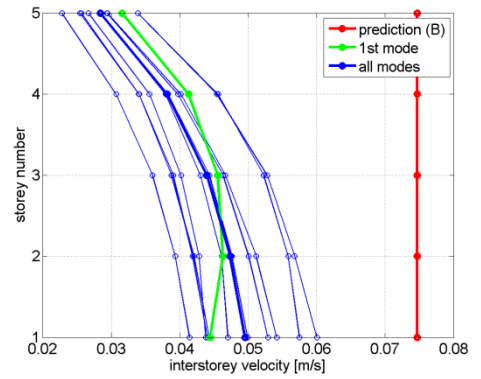


(D)

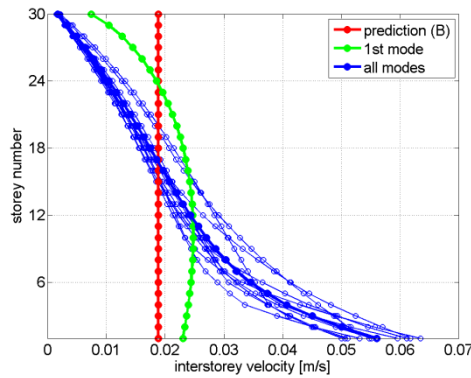
Figure 14: Peak inter-storey velocity profiles for the 5% damped structures: (A) 5-storey structure with UD placement; (B) 5-storey structure with SPD placement; (C) 30-storey structure with UD placement; (D) 30-storey structure with SPD placement.



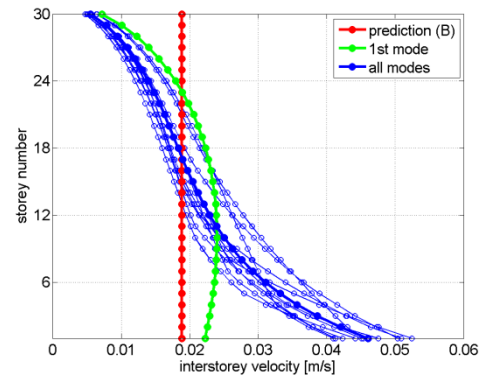
(A)



(B)



(C)



(D)

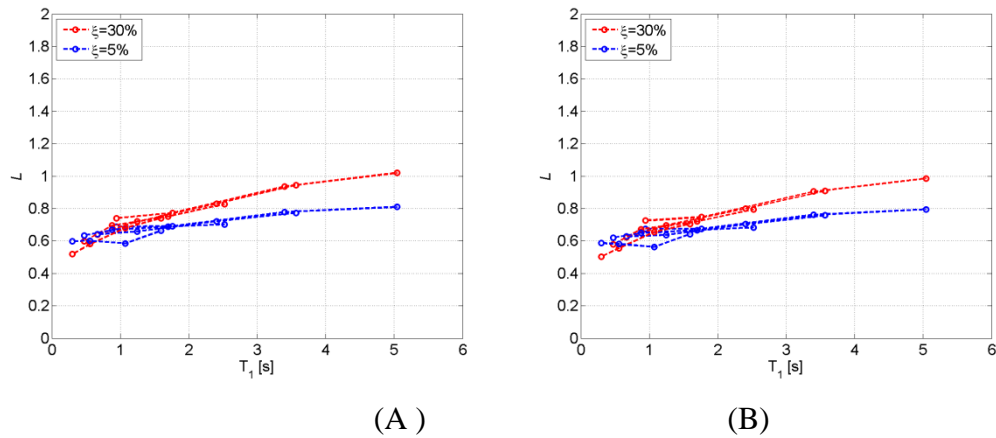
Figure 15: Peak inter-storey velocity profiles for the 30% damped structures: (A) 5-storey structure with UD placement; (B) 5-storey structure with SPD placement; (C) 30-storey structure with UD placement; (D) 30-storey structure with SPD placement.

The following observations can be provided:

- For the 5-storey structure with 5% viscous damping (Fig. 14A and B) the average total and first mode inter-storey velocity profiles are very close to each other, and almost uniform along the building height. The predictions are on average conservative (maximum overestimations are around 30-50%). The different dampers placements do not significantly affect the peak inter-storey velocity profiles.
- For the 30-storey structure with 5% viscous damping (Fig. 14C and D):
 - the predictions are close to the average first mode profile;
 - the average total peak inter-storey velocities significantly increase going from the top to the bottom and the shapes are far from being uniform. The SPD placement leads to a slightly reduction (less than 10%) in the peak inter-storey velocities at the lower stories.
- As expected, for the 30% damped structures (Fig. 15) the peak inter-storey velocities are, on average, reduced on around 50% with respect to those of the 5% damped structures. The SPD placement leads to a slightly reduction (less than 10%) in the peak inter-storey velocities at the lower stories.

7.3. Correction factors for different dampers placements

Figure 16 displays the trends of the average values of the correction factors L , H and M for the 5% and 30% damped structures with respect to T_1 .



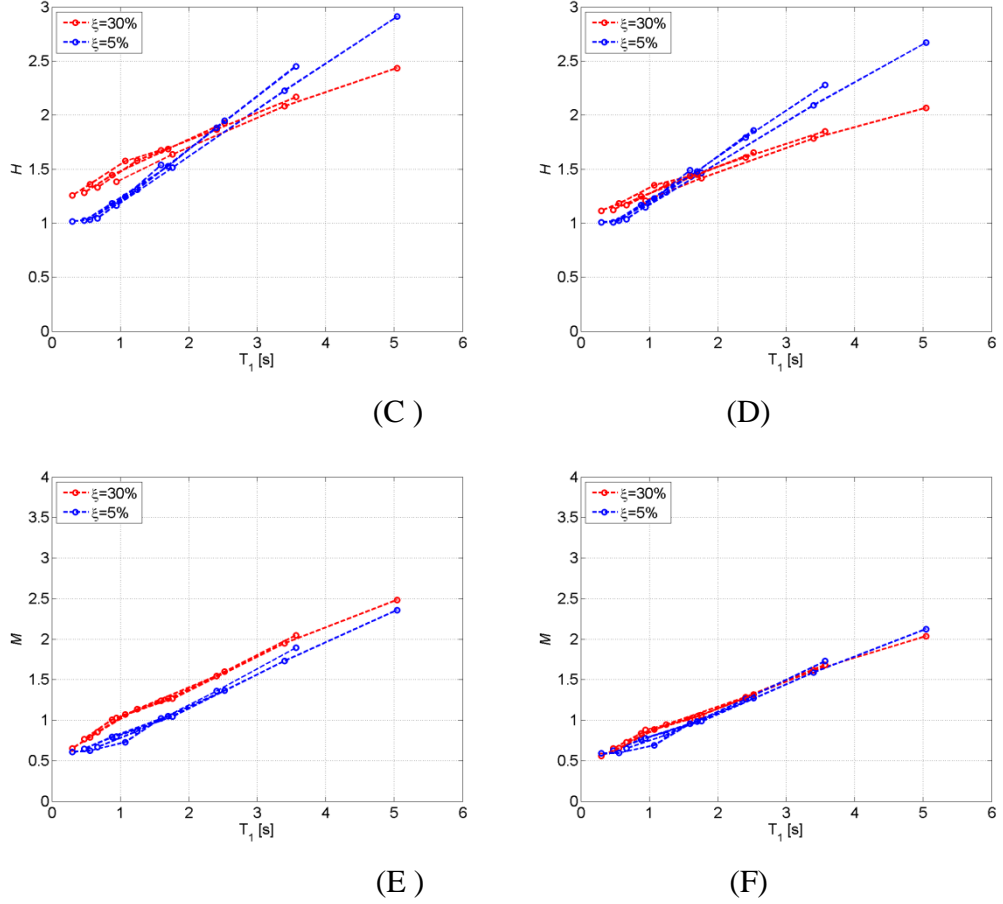


Figure 16: Mean values of the correction factors L , H and M for 5% and 30% damped structures: (A) L factor for UD placement; (B) L factor for SPD placement; (C) H factor for UD placement; (D) H factor for SPD placement; (E) M factor for UD placement; (F) M factor for SPD placement.

The dampers placement does not seem to have an influence on the trend of L . On the contrary, an effect on the trend of H is observed: the H values are on, average, slightly reduced for the SPD placement. Similarly, the M values are, on average, reduced for the SPD placement. Reductions are in the order of 10-15%. The expression of the correction factor M for the peak inter-storey velocity at the ground floor as given by Eq. 7 appears suitable to be used also for the Type B based estimations. In particular, the use of Eq. 7 in conjunction with Eq. 4 leads to the following analytical estimation of the actual peak inter-storey velocity at the ground floor:

$$v_{\max, B}^{tot} = \begin{cases} \frac{2}{N+1} \cdot \frac{S_a(T_1)}{\omega_1} & \text{for } T_1 \leq 0.5s \\ (0.44 \cdot T_1 + 0.78) \cdot \frac{2}{N+1} \cdot \frac{S_a(T_1)}{\omega_1} & \text{for } 0.5 < T_1 \leq 5.0s \end{cases} \quad (10)$$

8. RC MOMENT RESISTING FRAMES

In this final section, four realistic reinforced-concrete (RC) planar moment-resisting frames equipped with added viscous dampers are considered. The main objective is to evaluate possible discrepancies in the trends of the peak inter-storey velocity profiles with respect to those observed for the shear-type frame structures analysed in the previous sections. The influence of the beam-to-column stiffness ratio $\rho = EJ_{\text{beam}}/EJ_{\text{column}}$ has been deeply investigated by Miranda and Akkar (2006). Large values of ρ lead to a shear-type frame behaviour, while small values of ρ lead to a cantilever behaviour. In addition, the predictions of the peak inter-storey velocities profile according to the procedure here introduced are compared with those obtained according to the Equivalent Lateral Force (ELF) procedure of the ASCE 7-10 provisions (Ramirez et al. 2003).

8.1. The analysed structures

The analysed structures are characterized by: (i) the same number of bays (equal to 3), (ii) fixed inter-storey height (3 m) and span length (6 m), and (iii) equal beams at all floors (rectangular cross section of 70 cm height x 30 cm width). The total number of storeys ranges from 5 to 30. The columns have square cross sections decreasing each five storeys (from the bottom to the top) as it is usual for existing structures designed in low-intensity seismic regions. Uniform inter-storey viscous dampers are sized to achieve a target damping ratio equal to 30%. The structural models (Figure 17) and the seismic analyses have been developed using the commercial software SAP2000 (v16.1.1). For the analysed structures ρ values are between 0.33 and 1.5. Both RHA and 1-MRA have been carried out using the same reference input of section 3.2 (10 artificial ground motions).

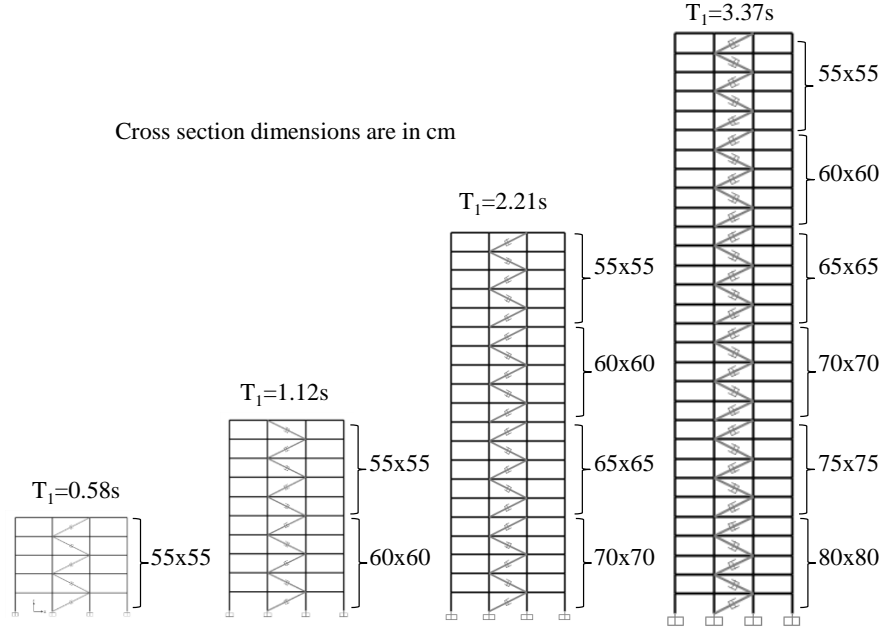


Figure 17: The four RC planar moment-resisting frame structures analysed.

8.2. The ASCE 7-10 ELF procedure

Chapter 18 of the ASCE 7-10 provisions provides simplified procedures for the design of structures equipped with added viscous dampers, including an Equivalent Lateral Force (ELF) procedure. The ELF procedure allows to estimate peak inter-storey drifts and velocities combining (SRSS rule) the contribution of the first mode with that of the so called “residual mode”. In particular, for relatively regular multi-storey frames, it is possible to use a linear first mode shape (e.g. Eq. 18.5-3 of ASCE 7-10). The ELF procedure is permitted for structures characterized by height above the base lower than 30 m (say 10 stories).

8.3. Main results

Figure 18 compares the average total and first mode peak inter-storey velocity profiles for the four RC frame structures as obtained from the time-history analyses, using Type A and Type B predictions of the present paper and following the ELF procedure. Figure 19 shows the ratios between the different predictions of the peak inter-storey velocity profiles and the average peak inter-storey velocity profiles as obtained from the numerical simulations (predictions/SAP2000 ratios). It has to be noted that the total responses according to Type A and Type B predictions are simply obtained by applying the correction factor M as given by Eq. 7 at all stories. This simplified assumption may lead to an overestimation of the peak inter-storey velocities at the upper stories, given that the expression of the M factor given by

Eq. 7 has been calibrated with specific reference to the ground floor where the largest amplifications are expected (see section 4.4).

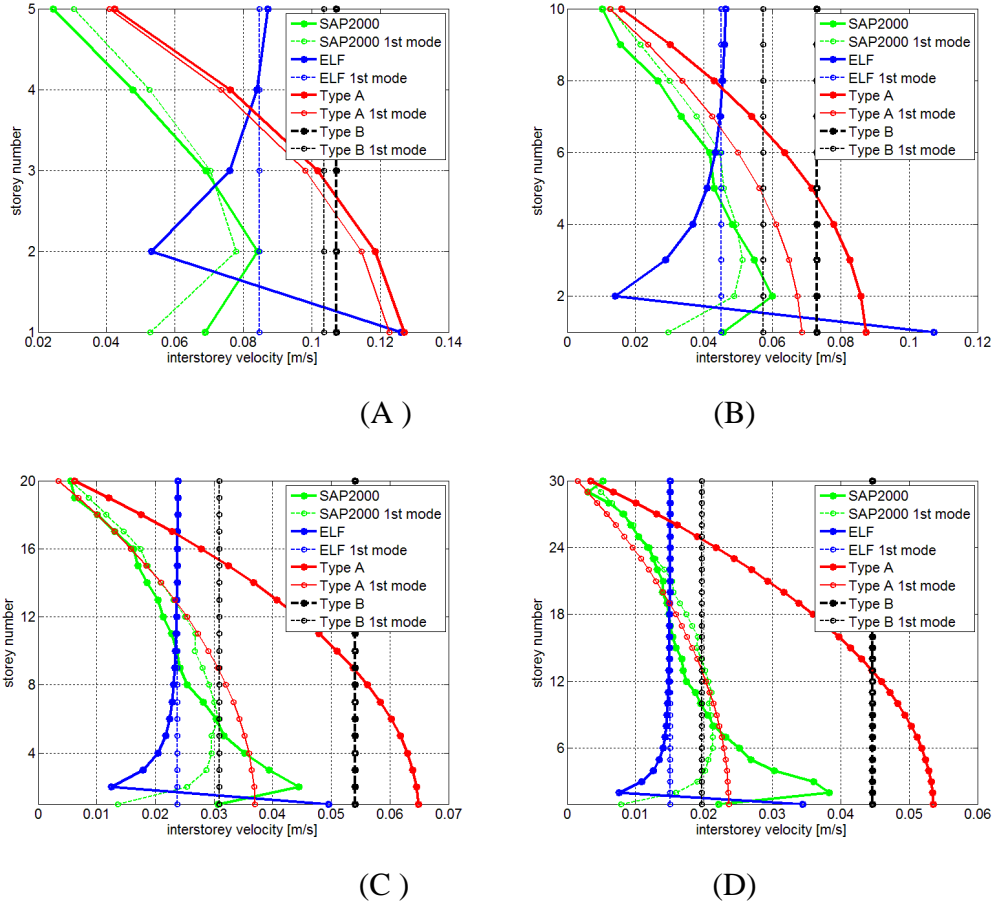
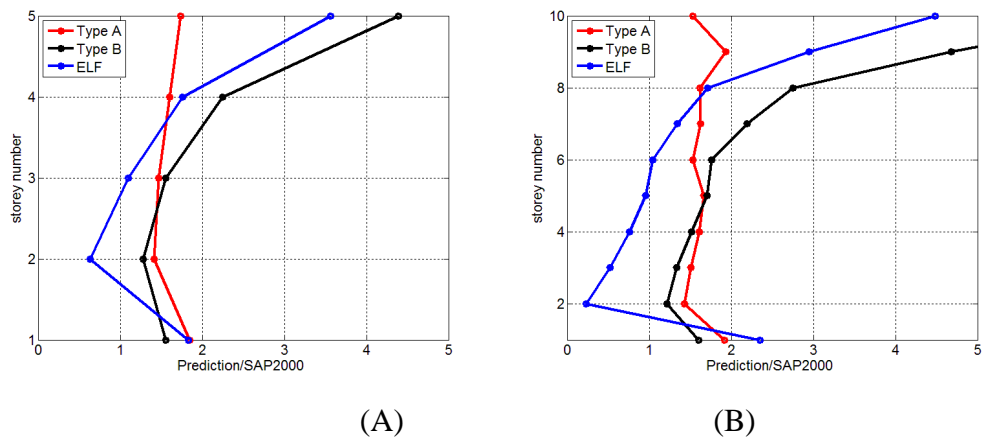


Figure 18: Inter-storey velocity profiles: (A) 5-storey frame; (B) 10-storey frame; (C) 20-storey frame; (D) 30-storey frame.



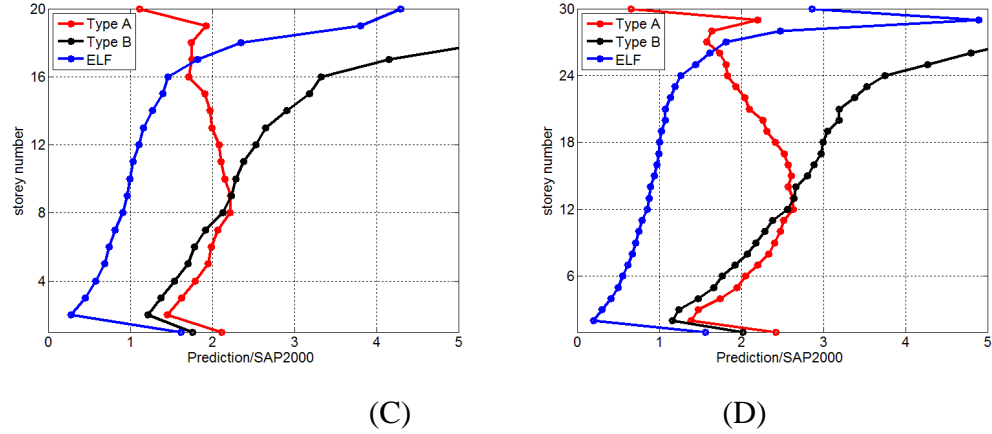


Figure 19: Prediction/SAP2000 ratios: (A) 5-storey frame; (B) 10-storey frame; (C) 20-storey frame; (D) 30-storey frame

Inspection of Figures 18 and 19 leads to the following observations:

- As expected, the total responses get more and more far apart from the first mode response as the total number of stories (and thus the fundamental period) increases. This is due to the increasing contribution of the higher modes.
- The ELF estimations are in general smaller than those given by Type B and Type A predictions (excluding the top stories). Indeed, the peculiar shape of the residual mode vector leads to a significant higher mode contribution at the lower stories only. On the contrary, at the upper stories, the ELF first mode and the ELF total responses are practically coincident. In more detail, at the bottom storey, the ELF predictions are generally conservative, while they become quite unconservative up to almost 2/3 of the building height. At the top stories, the ELF predictions become again conservative. At the roof level, the values of Prediction/SAP2000 ratios may become quite large (even larger than 4).
- In general, the Type A predictions (e.g. the ones obtained applying the same correction factor M at all stories) leads to conservative estimations of the peak inter-storey velocity along the whole building height. More in detail: (i) for the 5-storey and 10-storey frames, the actual total profiles as obtained from the numerical simulations (referred to as SAP2000) are closer to the Type A first mode estimations; (ii) for the 20-storey and 30-storey frames, the actual total response is in between the first mode and the total Type A

predictions. In general the values of Prediction/SAP2000 ratios are between 1 and 2,5.

- The Type B predictions (obtained multiplying the uniform inter-storey velocity profile as given by Eq. 4 by the same correction factor M at all stories), become more and more conservative going from the bottom to the top. At the top storey, the values of Prediction/SAP2000 ratios may become very large (even larger than 5).

9. CONCLUSIONS

The paper presents the main results of a study focused on the assessment of peak inter-storey drifts and peak inter-storey velocities developed in frame structures equipped with added viscous dampers during earthquakes. Simple analytical formulas for their predictions have been introduced.

The effectiveness of the predicted formulas, which are grounded on an assumed first mode shape, and targeted to professional engineers for the preliminary design phase, has been evaluated by performing a large number of time-history analyses and compared with other formulations available in the literature. The results of the study allow to make the following conclusions:

- The analytical predictions of the inter-storey drifts based on the analytical first mode shape are reasonably accurate and suitable to be used in a wide period range (up to 5 s) without any correction factor.
- On the contrary, the analytical predictions of the peak inter-storey velocities based on the analytical first mode response are reasonably accurate only for low-period structures. On the contrary, for frame structures with fundamental periods larger than 0.5 s, the estimations become unconservative especially at the bottom storeys, due to a significant contribution of the higher modes.
- For structures with fundamental periods larger than 0.5 s, the expression of the peak inter-storey velocity at the ground floor based on the analytical first mode shape have been corrected by introducing a correction factor M , whose values have been calibrated through the results of extensive time-history simulations. The values of the correction factor M exhibit a linear trend with respect to the fundamental period of the structure. On average,

the maximum values of M are around 2.5-3.0 for regular frame structures of about 5.0 s fundamental period.

- A first analysis aimed at assessing the sensitivity of the calibrated correction factors on (i) the average spectral properties of the earthquake ground motions, (ii) the along-the-height lateral stiffness distribution and dampers placements and (iii) the beam-to-column stiffness ratios revealed that such parameters leads to variations in the values of the correction factors which are in the order of 10-20%. Nonetheless, additional studies are necessary to better evaluate such effects.

ACKNOWLEDGEMENTS

Financial supports of Department of Civil Protection (DPC-Reluis 2014–2018 Grant – Research line 6: “Seismic isolation and dissipation”) is gratefully acknowledged.

REFERENCES

- Adachi F, Fujita K, Tsuji M, Takewaki I (2013) Importance of inter-storey velocity on optimal along-height allocation of viscous oil dampers in super high-rise buildings. *Engineering Structures*, 56: 489-500
- Akkaş S, Yazgan U, Gülkan P (2005). Drift estimates in frame buildings subjected to near-fault ground motions. *Journal of Structural Engineering*, 131(7): 1014-1024
- Bozorgnia Y, Bertero VV (2001) Improved shaking and damage parameters for post-earthquake applications Proc., SMIP01 Seminar on Utilization of Strong-Motion Data, California Division of Mines and Geology, Los Angeles, 1–22
- CSI (Computers and Structures Inc.) (2013). SAP2000 v16 Integrated Finite Element Analysis and Design of Structures. CSI, Berkeley,.
- Christopoulos C, Filiatrault A (2006) Principles of passive supplemental damping and seismic isolation. IUSS Press, Pavia
- Chopra AK (2001) Dynamics of structures. Theory and applications to earthquake engineering. Prentice-Hall, Upper Saddle River
- Chopra AK, Chintanapakdee C (2001). Drift spectrum versus modal analysis of structural response to near-fault ground motions. *Earthquake Spectra* 17(2), 221–234
- Hilber HM, Hughes TJ, Taylor, L. (1977). Improved numerical dissipation for time integration algorithms in structural dynamics. *Earthquake Engineering and Structural Dynamics*, 5(3): 283-292
- MATLAB 8.0 and Statistics Toolbox 8.1, The MathWorks, Inc., Natick, Massachusetts, United States.
- Moehle JP (1994) Seismic drift and its role in design. Proc., 5th U.S.–Japan Workshop on the Improvement of Building Structural Design and Construction Practices, San Diego, 65–78.
- Miranda E, Akkaş SD (2006). Generalized inter-storey drift spectrum. *Journal of Structural Engineering*, 132(6): 840-852.

- NTC (2008) Norme Tecniche per le Costruzioni, Italian building code, adopted with D.M. 14/01/2008, published on S.O. n. 30 G.U. n. 29 04/02/2008
- Palermo M, Silvestri S, Gasparini G, Trombetti T (2014). A statistical study on the peak ground parameters and amplification factors for an updated design displacement spectrum and a criterion for the selection of recorded ground motions. *Engineering Structures*, 76, 163-176.
- Palermo M, Silvestri S, Gasparini G, Trombetti T (2015). Seismic Modal Contribution Factors. *Bulletin of Earthquake Engineering*, DOI 10.1007/s10518-015-9757-7
- Palermo M, Silvestri S, Landi L, Gasparini G, Trombetti T (2016). Peak velocities estimation for a direct five-step design procedure of inter-storey viscous dampers. *Bulletin of Earthquake Engineering*, 14(2), 599-619.
- Pekcan G, Mander JB, Chen S (1999). Fundamental considerations for the design of non-linear viscous dampers. *Earthquake Engineering and Structural Dynamics*; 28: 1405-1425.
- Silvestri S, Gasparini G, Trombetti T (2010) A five-step procedure for the dimensioning of viscous dampers to be inserted in building structures. *J Earthq Eng* 14(3):417–447
- Sozen MA (1983) Lateral drift of reinforced concrete structures subjected to strong ground motion. *Bull. New Zealand Natl. Soc. Earthquake Engineering*, 162, 107–122.
- Vanmarcke EH, Cornell CA, Gasparini DA, Hou S (1990) SIMQKE-I: simulation of earthquake ground motions. T.F. Blake, Newbury Park, California, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, Modified