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PREDICTION OF STRUCTURAL RESPONSE IN REINFORCED CONCRETE FRAMES SUBJECTED TO EARTHQUAKE GROUND MOTIONS

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ABSTRACT

This paper deals with the prediction of both force-based response measures and displacement-based response measures of different configurations of four story reinforced concrete frame buildings, with and without infill walls, and designed with different strength characteristics. The prediction is performed via statistical relationships between ground-motion intensity measures (IMs) and various engineering demand parameters (EDPs). The relationship is built on data obtained from nonlinear dynamic analyses of the frames subjected to one hundred strong motion records. The EDPs considered are maximum base shear, maximum story shear, maximum overturning moment, peak (over time) inter-story drift ratio, maximum (over all stories) peak inter-story drift ratio and roof drift ratio.

Introduction

Performance-based earthquake engineering (PBEE) is widely used in the design and assessment of structures in earthquake-prone regions. In the assessment phase, PBEE is primarily used to guide the retrofitting old buildings that do not meet current design codes and safety standards. PBEE is also used during building design when structures that do not conform to code prescriptions need to be shown to perform similarly or better than code-based buildings. For both assessment and design applications, the expected structural responses need be estimated with a high level of accuracy using the available computational resources. When PBEE is applied, the structural response is usually computed via nonlinear dynamic time-history analysis of sophisticated 2-Dimensional or 3-Dimensional computer models subject to real or synthetic

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ground motions that are "consistent" with the hazard at the building site. The accuracy in response prediction is accomplished by relying on relationships between ground-motion intensity measures (IMs) and measures of structural responses, which in this framework are often called Engineering Demand Parameters (EDPs). The higher the predictive power of the selected IMs, the lower is the uncertainty in the estimated EDP and, therefore, the lower the number of computer runs necessary to achieve the desired level of statistical accuracy.

Most of the studies available in the literature deal with response assessment of steel moment resisting frame buildings (Luco et al. 2005). In this case, the EDP chosen for structural damage prediction is typically the maximum inter-story drift ratio and the EDP chosen for content damage prediction is the absolute floor acceleration. Much less attention has been devoted to other types of buildings such as reinforced concrete frames (Haselton et al. 2008) and to the use of PBEE during design of new buildings rather than in the assessment of existing ones. During design, force-based measures rather than deformation-based measures often control. Little attention has been devoted so far in establishing the IMs that best predict such response quantities. In this study we investigate the response of four story reinforced concrete frame buildings of different vintages and infill walls configurations. The response is gauged in terms of both force-based EDPs, such as maximum base shear, maximum story shear, and overturning moment, and displacement-based EDPs, such as peak inter-story drift ratio, and maximum peak inter-story drift ratio over all stories. We have considered both brittle such as those commonly designed in the 1960s and more modern ductile buildings. Multiple IMs were used and their power of predicting different force-based and deformation-based EDPs will be contrasted and compared.

Intensity Measures and Engineering Demand Parameters

In order to predict the damage resulting from earthquake ground motions, it is first necessary to identify suitable ground-motion parameters that are well correlated with structural response and, in turn, with damage. The broad objective of this research work is to establish correlations between intensity measures (IMs) and engineering demand parameters (EDPs) that describe the performance of a structure, using a comprehensive set of ground-motion time histories for a variety of structures. In this paper the IMs considered range from peak values to spectral quantities, and include duration and energy-based quantities in order to account for the effect of different levels of ground motion intensity, duration, and frequency content. The intensity measures (IMs) considered here are: peak ground acceleration, PGA; peak ground velocity, PGV; peak ground displacement, PGD; Incremental Velocity, IV, which is the area under the maximum acceleration pulse (Bertero et al. 1976); Incremental Displacement, ID, which is the area under the maximum velocity pulse; effective duration, t_D, (Trifunac & Brady 1975); Housner Intensity, I_H (Housner 1952), given by:

$$I_H = \int_{0.1}^{2.5} S_{PV}\left(\xi = 5\%, T\right) dT \tag{1}$$

where S_{pv} is the pseudo-spectral velocity; T is the natural period; ξ is the damping ratio; Arias Intensity, I_A (Arias 1969), given by:

$$I_A = \frac{\pi}{2g}\int_{0}^{t_d} \ddot{a}_g^2(t)dt \tag{2}$$

where $\ddot{a}_g$ is the ground acceleration, T is the natural period; ξ is the damping ratio; spectral pseudo-acceleration at specified vibration periods of the structure: $S_a(T_1)$, at the fundamental period T_1 of the structure; $S_a(T_2)$ and $S_a(T_3)$ at shorter modal period; $S_a(2T_1)$ at a longer period (two time the fundamental period of the structure) to take into account the elongation of period due to the non-linear structural behavior caused by the inelastic response; spectral pseudo-relative velocity response S_{pv} at T_1, T_2, T_3, and $2T_1$; absolute, $E_{Ia}(T_1)$, and relative, $E_{Ir}(T_1)$, Input Energy, as the maximum value of the energy input into the system during ground shaking, at T_1, T_2, T_3, and $2T_1$. The absolute input energy is the work done by the total force applied to the base of the structure. The relative input energy is the work done by an equivalent lateral force on a fixed base system, and neglects the effects of rigid body translation (Uang and Bertero, 1990). Energy serves as an alternative to response quantities such as forces and displacements, and also includes the effect of duration.

The deformation-based EDPs considered here are:

- peak (over time) inter-story drift ratio, as the largest difference between the lateral displacements of two adjacent floors, divided by the height of the story (denoted as IDR_i for story i);
- maximum (over all stories) peak interstory drift ratio (e.g., Luco et al. 2005) (MIDR);
- ratio of the peak lateral roof displacement to the building height; denoted as RDR;
- average (over all stories) peak interstory drift ratio; denoted as AIDR;

IDR_i has been shown to be well correlated to both structural and non-structural damage experienced at the i^{th} story during an earthquake. MIDR can be used to estimate local instability story collapse. RDR can be considered as a measure of the global seismic response of the structure, and is also related to the global stability of the moment-resisting frame. AIDR can be related to overall damage in a structure and to instability of the structure as a whole.
Finally, the force-based EDPs used in this study are: maximum story shear (called V_i for story i); overturning moment (denoted as M)

Description of the reinforced concrete frames

The Italian reinforced concrete frame building stock includes a variety of configurations, with both regular and irregular plans and stiffness and/or mass distributions along the height.
The dwellings inventory recently set up by ISTAT (Italian National Institute of Statistics) was used as a reference to identify the most common configurations of reinforced concrete buildings existing in Italy, and to select those to be considered in this study. A preliminary estimate of the number of buildings subdivided in classes of mean floor area was taken from Bramerini and Di Pasquale (2006). This study indicates that 5-story or lower buildings constitute 95% of the entire inventory measured in terms of volume built. Buildings of smaller size and fewer stories prevail if one uses the number of buildings rather than the volume built as reference parameter. Moreover, the same study indicates that 90% of single-story buildings has an average floor area between 50 m^2 and 200 m^2. We selected the plans in Figure 1 by combining number of floors data with average floor area data. The first two plans (100 m^2 and 150 m^2) refer to single-story buildings (without stairs); the 75 m^2 plan can be assumed for buildings with 2, 3 and 4 stories; the 185 m^2 plan is typical of buildings with number of stories ranging between 3 and 6; finally, the 290 m^2 can be associated with 4- to 8-story buildings. Of course, the floor plans selected do

not encompass all possible ones, especially for average floor areas exceeding 200 m^2. However, the selected configurations are by far the most common.

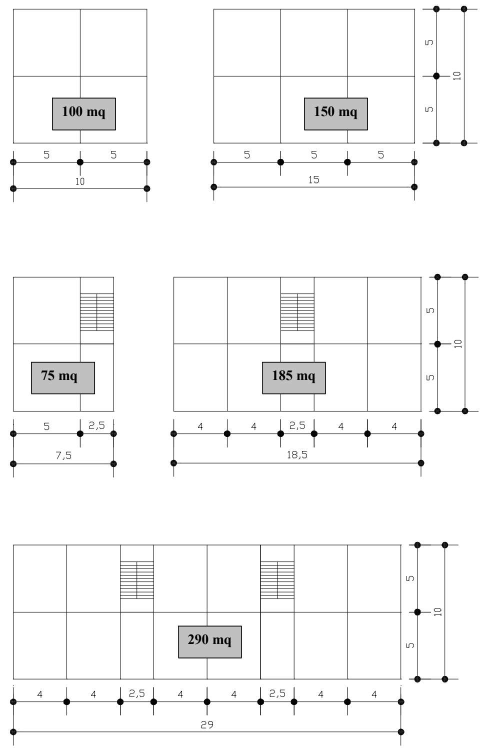

Figure 1. Plan configurations representative of the existing Italian reinforced concrete building inventory considered in this study

Figure 2 shows the 28 different two-dimensional frames obtained by extracting the frames along the two principal directions of each plan typology, including also those containing the stair shaft. 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, to limit the number of computer analyses in this study the 28 frames were reduced to 21 by excluding those with stairs shaft.

In the end, to give more breadth to this research study we evaluated frames having 2, 4, 6 and 10 stories with regular stiffness and mass distribution in plan and elevation, although, as discussed above, the taller frames are quite infrequent. In this paper, however, we only present the results from the 4-story frames. 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.

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.

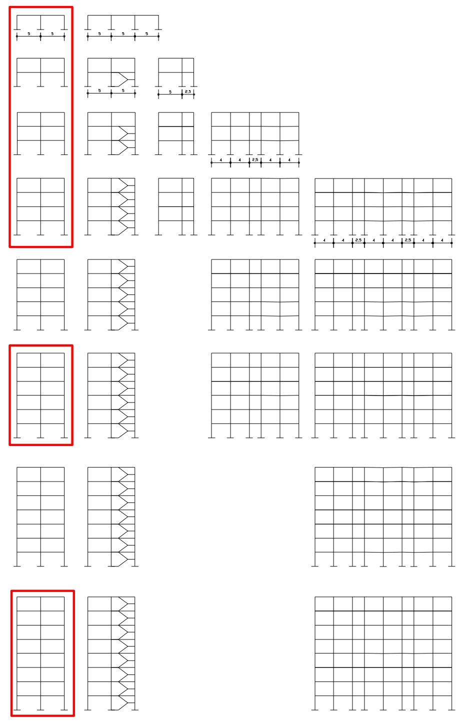

Figure 2. Typical frames found in existing Italian reinforced concrete buildings.

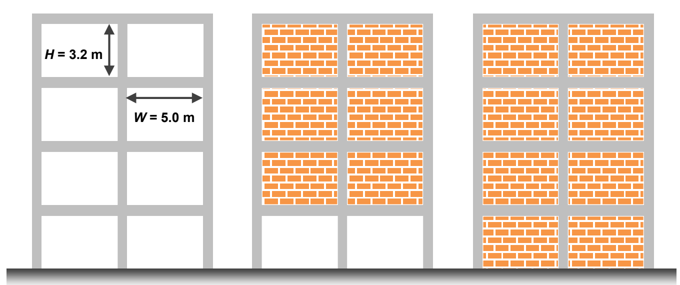

Figure 3. R.C. frame typologies: Bare (B), Infilled (T), Pilotis (P)

Both models take into account the interaction between axial and flexural stresses. An uniaxial Kent-Scott-Park concrete material object with degraded linear unloading/reloading stiffness and no tensile strength and a bilinear law with kinematic hardening were adopted to model the concrete and steel responses, respectively. Models that account for shear behavior were also built via a *section aggregator* object, which couples the axial/flexural response (already described by the fiber section) and the shear response (represented by an uniaxial constitutive law) at the integration points of the column elements. The consideration of shear fracture is limited to cases of low values of C_y, namely equal to 0.10 and 0.15, representative of buildings designed for gravity loads only or for very low seismicity levels. These frames are characterized by poor quantities of transverse steel, widely spaced along the element. Masonry infill walls were modeled through equivalent diagonal trusses element, with no tensile stress and inelastic in compression.

Strong motion records

The 98 ground motions used in this study were selected in such a way that at least some of them would drive even the more modern of the considered building frames into the severe nonlinear response range A wide range of ground motions characteristics is needed to permit an in-depth exploration of the influence of IMs measures on their correlation to EDPs. Magnitude, M_w, ranges between 5.0 and 7.6, while closest distance from the causative fault (D_f) ranges from 0.7 to 21 km. Several records are near-fault, including both forward and backward directivity regions. Recording stations are located on C-D NEHRP soil type. Finally peak ground acceleration (PGA) varies between 150 and 1200 cm/s^2, and peak ground velocity (PGV) ranges from 10 to 170 cm/s. The distribution of motions selected (in PGA-M_w and PGA.D_f space) is shown in Figure 4.

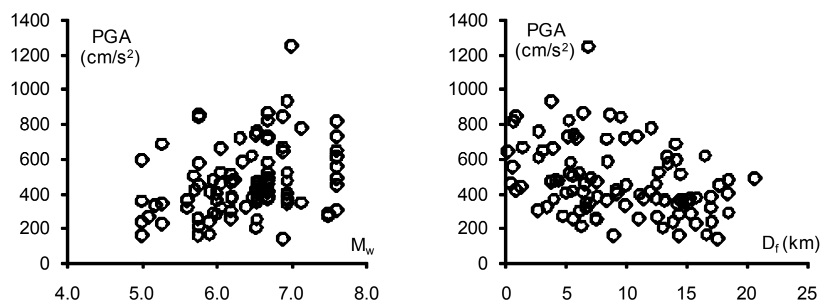

Figure 4. Distribution of PGA (cm/s^2) values as a function of Magnitude (M_w) and closest distance from the fault (D_f, km).

Regression Model

Statistical regression techniques were used here to identify the IM that best predicts each EDP of interest. The best predictor is the IM that provides the 'best regression fit' for the EDP. In order to develop a relationship between an EDP and an IM, it is first necessary to identify the functional form of the relationship that best fits the data. By inspecting the scatter plots between several EDPs and IMs, it was decided that among the simple models the one that fitted better in all cases had the following equation:

$$EDP = aIM^b \tag{3}$$

where a and b are regression parameters. Incidentally, this relationship was also considered by other researchers in the past (e.g., Cornell et al. 2002, Jalayer 2003). Of course, this equation can be rewritten as

$$\ln(EDP) = \ln(a) + b\ \ln(IM) \tag{4}$$

and this format shows a linear relationship between the logarithm of the EDP and the logarithm of the IM. The coefficients ln(a) and b can thus be obtained using linear regression between ln(EDP) and ln(IM). For instance, Figure 5 shows the scatter plots and the regression relationships between MIDR and IV and MIDR and $S_a(T_1)$ for the 4-story frame with infill walls designed for a base shear coefficient of 0.35. The fitted models are

$$\ln(MIDR) = -5.5480 + 1.3389 \ln(IV) \tag{5a}$$

$$\ln(MIDR) = 0.0456 + 1.3755 \ln(S_a(T_1)) \tag{5b}$$

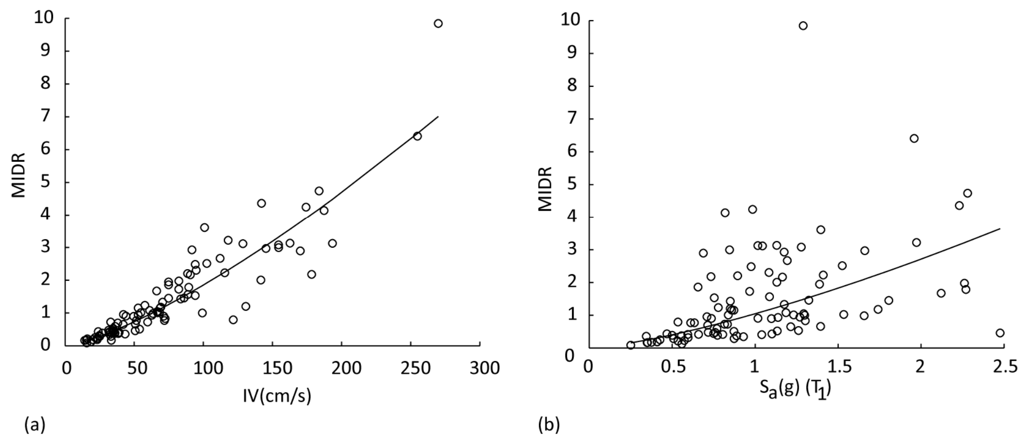

Figure 5. Frame with infill walls: Fitted models for MIDR vs. (a) IV (cm/s), and (b) $S_a(T_1)$ (g).

The IM that best predicts the EDP is then the one that provides the largest value of the coefficient of determination, R^2, among those considered. R^2 is the proportion of variability in a data set that is accounted for by the statistical model

For instance, the relationship between MIDR and IV intensity has a R^2 value of 0.8832, while the relationship between MIDR and $S_a(T_1)$ has a R^2 value of 0.4533. Both the R^2 values and the scatter plots show that MIDR is better predicted by IV rather than $S_a(T_1)$ for this frame.

Results and discussion

The procedure described above is used to guide the selection of the most efficient predictors for all the EDPs of interest for all the frames considered. Tables 1 to 3 show the "best" predictors identified for all the cases considered.

Table 1: Best predictors for EDPs computed for the bare frames

EDP\IM	C_y=0.1	C_y=0.15	C_y=0.25	C_y = 0.35	C_y=0.1,S	C_y=0.15,S
MIDR	I_H	I_H	I_H	I_H	I_H	I_H
RDR	ID	I_H	I_H	I_H	ID	I_H
AIDR	I_H	I_H	I_H	I_H	I_H	I_H
IDR_1	ID	I_H	I_H	I_H	ID	I_H
IDR_2	I_H	I_H	I_H	I_H	I_H	I_H
IDR_3	I_H	I_H	I_H	I_H	I_H	I_H
IDR_4	I_H	I_H	I_H	I_H	I_H	I_H
V_1	$S_a(T_1)$	$S_a(T_1)$	I_H	I_H	$S_a(T_1)$	$S_a(T_1)$
V_2	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	I_H	$S_a(T_1)$	$S_a(T_1)$
V_3	I_A	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	I_H	$S_a(T_1)$
V_4	$E_{Ia}(T_2)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$
M	I_H	I_H	I_H	I_H	I_H	I_H

Note that, strictly speaking, the IMs listed in the tables are not always those associated with the largest value of R^2. Given their strong correlation, multiple IMs provide values of R^2 that are very close to one another but some of these IMs are much easier to be predicted for a given

earthquake scenario than others. For example, more ground motion prediction equations (GMPE) exist for, say, S_a than for I_A. In these cases, we favored the IM with more established GMPEs. In addition, to give some sense of uniformity to these tables, if the values of R^2 were very close for two IMs, we selected the IM that was more represented for other frames or for other EDPs. It is emphasized, however, that in this somewhat heuristic search for the best predictor we limited our selection to one among the five top ranked IMs for each case, which is a legitimate strategy given the very similar values of R^2 associated with them.
It can be seen from the above tables that the best predictor for the displacement-based EDPs is the Housner Intensity for the bare and the pilotis frames, and the incremental velocity for the frame with infill walls. For instance, as discussed earlier Fig. 5 illustrates that the MIDR is better predicted by IV than by the traditionally-used $S_a(T_1)$ for the frame with infill walls. Similarly, Fig. 6 shows that the MIDR is better predicted by I_H than by $S_a(T_1)$ for the bare frame designed for a C_y of 0.35.

Table 2: Best predictors for EDPs computed for frames with infill walls

EDP\IM	C_y=0.1	C_y=0.15	C_y=0.25	C_y = 0.35	C_y=0.1,S	C_y=0.15,S
MIDR	IV	IV	IV	IV	IV	IV
RDR	IV	IV	IV	IV	IV	IV
AIDR	IV	IV	IV	IV	IV	IV
IDR_1	IV	IV	IV	IV	IV	IV
IDR_2	I_H	$S_a(2T_1)$	IV	IV	IV	$S_a(2T_1)$
IDR_3	IV	$S_a(2T_1)$	$S_a(2T_1)$	IV	IV	$S_a(2T_1)$
IDR_4	IV	IV	IV	IV	IV	IV
V_1	$S_a(2T_1)$	$S_a(2T_1)$	$S_a(2T_1)$	IV	$S_a(2T_1)$	$S_a(2T_1)$
V_2	$S_a(2T_1)$	$S_a(2T_1)$	$S_a(2T_1)$	IV	$S_a(2T_1)$	$S_a(2T_1)$
V_3	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	IV	$S_a(T_2)$	$S_a(T_1)$
V_4	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_2)$	$S_a(T_1)$
M	IV	IV	IV	IV	$S_a(2T_1)$	IV

Table 3: Best predictors for EDPs computed for pilotis frames (soft-story at the first story)

EDP\IM	C_y=0.1	C_y=0.15	C_y=0.25	C_y = 0.35	C_y=0.1,S	C_y=0.15,S
MIDR	I_H	I_H	I_H	I_H	I_H	I_H
RDR	I_H	I_H	I_H	I_H	I_H	I_H
AIDR	I_H	I_H	I_H	I_H	I_H	I_H
IDR_1	I_H	I_H	I_H	I_H	I_H	I_H
IDR_2	I_H	I_H	I_H	I_H	I_H	I_H
IDR_3	I_H	I_H	I_H	I_H	I_H	$S_a(T_1)$
IDR_4	I_H	I_H	I_H	I_H	I_H	I_H
V_1	$S_a(T_1)$	$S_a(T_1)$	I_H	I_H	$S_a(T_1)$	$S_a(T_1)$
V_2	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	I_H	$S_a(T_1)$	$S_a(T_1)$
V_3	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	I_H	$S_a(T_2)$	$S_a(T_1)$
V_4	$S_a(2T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_1)$	$S_a(T_2)$	$S_a(T_1)$
M	$S_a(T_1)$	$S_a(T_1)$	I_H	I_H	I_H	$S_a(T_1)$

The story shears are harder to predict (i.e., lower R^2 values) than deformation-based measures, and the best predictor for story shears appear to be changing across frame types. This behavior could be caused by the somewhat limited ground motion sample size. $S_a(T_1)$ is the intensity measure that predicts the story shear reasonably well in the majority of the cases. Example predictions of the fourth story shear, V_4, using $S_a(T_1)$ for the pilotis frames and the frames with infill walls designed for a C_y of 0.35 are shown in Figure 7. The overturning moment, *M*, is predicted well by incremental velocity for pilotis frames, and by the Housner intensity for the bare frames and for the frames with infill walls (Figure 8). The prediction errors for the overturning moment are comparable to those for the displacement-based EDPs.

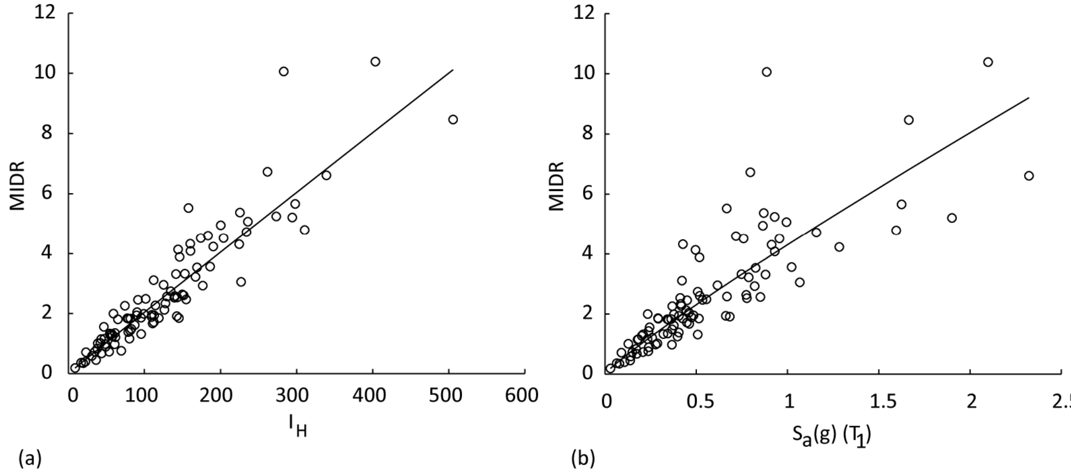

Figure 6. Bare frame: Fitted model for MIDR vs. (a) I_H (cm), and (b) $S_a(T_1)$ (g).

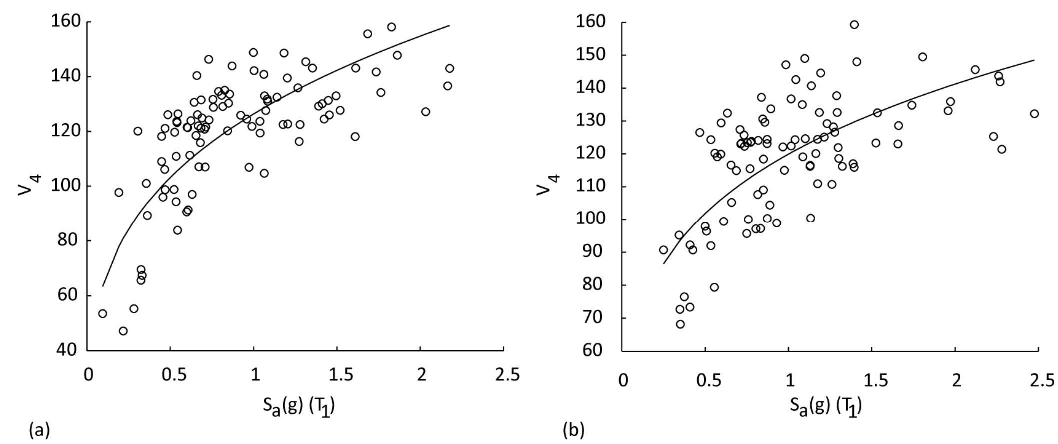

Figure 7. Fitted model for V_4 vs. $S_a(T_1)$ (g) for (a) pilotis frame and (b) frame with infill walls.

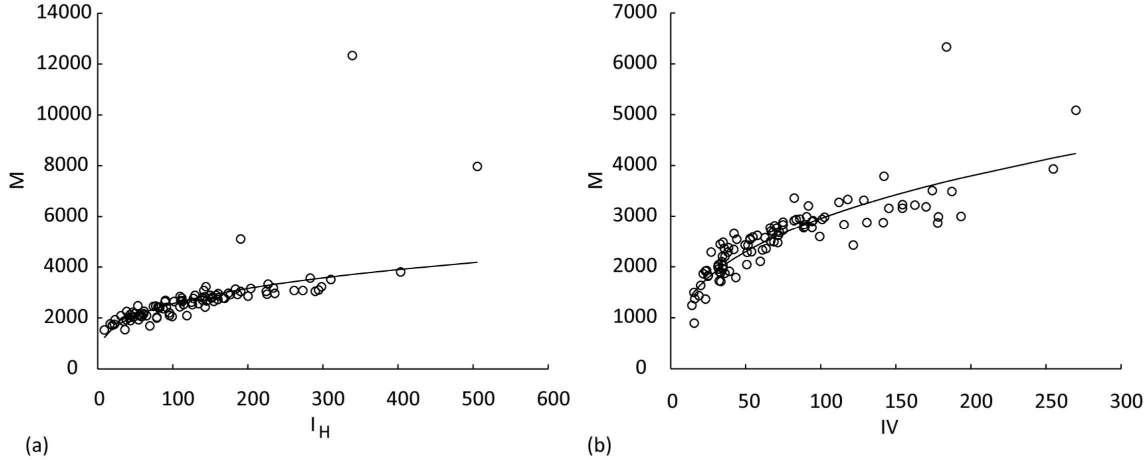

Figure 8. Fitted models for (a) Frame with infills: M vs. I_H (cm) (b) Pilotis frame: M vs. IV (cm/s).

Conclusions

This paper presented the preliminary results of a larger and more ambitious study aiming at identifying the ground motion parameters that are better correlated with both deformation-based and force-based structural response parameters. For this purpose we considered a large ensemble of reinforced concrete frames typical of the Italian inventory of different vintages existing in different seismic regions. This paper only shows some results obtained for 4-story frames with different infill wall configurations. In general, deformation-based parameters, such as maximum interstory drifts, and overturning moment appear to be better correlated with ground motion intensity parameters than base shear. Among all the intensity measures considered, the velocity-based ones, such as the Housner Intensity and the incremental velocity, seem to be better correlated with deformation-based parameters. This is not unexpected given that the first period of vibration of these frames is in the velocity-sensitive part of the spectrum. Spectral acceleration predicts better base shear.

Acknowledgments

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