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Correlations between Energy and Displacement Demands for Performance-Based Seismic Engineering

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Abstract—The development of a scientific framework for performance-based seismic engineering requires, among other steps, the evaluation of ground motion intensity measures at a site and the characterization of their relationship with suitable engineering demand parameters (EDPs) which describe the performance of a structure. In order to be able to predict the damage resulting from earthquake ground motions in a structural system, it is first necessary to properly identify ground motion parameters that are well correlated with structural response and, in turn, with damage. Since structural damage during an earthquake ground motion may be due to excessive deformation or to cumulative cyclic damage, reliable methods for estimating displacement demands on structures are needed. Even though the seismic performance is directly related to the global and local deformations of the structure, energy-based methodologies appear more helpful in concept, as they permit a rational assessment of the energy absorption and dissipation mechanisms that can be effectively accomplished to balance the energy imparted to the structure. Moreover, energy-based parameters are directly related to cycles of response of the structure and, therefore, they can implicitly capture the effect of ground motion duration, which is ignored by conventional spectral parameters. Therefore, the identification of reliable relationships between energy and displacement demands represents a fundamental issue in both the development of more reliable seismic code provisions and the evaluation of seismic vulnerability aimed at the upgrading of existing hazardous facilities. As these two aspects could become consistently integrated within a performance-based seismic design methodology, understanding how input and dissipated energy are correlated with displacement demands emerges as a decisive prerequisite. The aim of the present study is the establishment of functional relationships between input and dissipated energy (that can be considered as parameters representative of the amplitude, frequency content and duration of earthquake ground motions) and displacement-based response measures that are well correlated to structural and non-structural damage. For the purpose of quantifying the EDPs to be related to the energy measures, for comprehensive range of ground motion and structural characteristics, both simplified and more accurate numerical models will be used in this study for the estimation of local and global displacement and energy demands. Parametric linear and nonlinear time-history analyses will be performed on elastic and inelastic SDOF and MDOF systems, in order to assume information on the seismic response of a wide range of current structures. Hysteretic models typical of frame force/displacement behavior will be assumed for the local inelastic cyclic response of the systems. A wide range of vibration periods will be taken into account so as to define displacement, interstory drift and energy spectra for MDOF systems. Various scalar measures related to the deformation demand will be used in this research. These include the spectral displacements, the peak roof drift ratio, and the peak interstory drift ratio. A total of about 900 recorded ground motions covering a broad variety of condition in terms of frequency content, duration and amplitude will be used as input in the dynamic analyses. The records are obtained from 40 earthquakes and grouped as a function of magnitude of the event, source-to-site condition and site soil condition. In addition, in the data-set of records a considerable number of near-fault signals is included, in recognition of the particular significance of pulse-like time histories in causing large seismic demands to the structures.

1. Introduction

Research on performance-based seismic engineering poses many challenges, among them being the need for a reliable procedure to predict structural damage and collapse as a function of the earthquake ground motion intensity. A large source of variability in seismic performance assessment arises from simplification in defining earthquake intensity relative to the proper damaging consequences of ground motions on structures.

Usually, the intensity of ground shaking and the demand on structures have been characterized using parameters such as peak ground acceleration as well as strength-based parameters such as response spectrum ordinates (e.g., pseudo-spectral acceleration),

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that represent the maximum amplitude of shaking for structures with specified natural periods and damping ratios.

Various studies suggested that improved performance parameters, such as deformation and energy demands, could be considered explicitly during seismic design (Fajfar and Gaspersic, 1996; Priestley and Calvi, 1997; Teran-Gilmore, 1996, 1998; Krawinkler *et al.*, 1999; Leelataviwat *et al.*, 1999; Bertero and Bertero, 2002).

Actually, having been considered in recent design guidelines, the displacement-based approach is more familiar (Whittaker *et al.*, 1998; Bommer and Elnashai, 1999; Gupta and Krawinkler, 2000; Miranda, 2000, 2001; Borzi *et al.*, 2001; Miranda and Aslani, 2002; Decanini *et al.*, 2003; Krawinkler *et al.*, 2003); on the contrary there is a considerably lesser amount of research on the advancement of an energy-based methodology (Bertero and Uang, 1992; Fajfar and Vidic, 1994; Teran-Gilmore, 1998; Leelataviwat *et al.*, 1999; Chai and Fajfar, 2000; Bertero and Bertero, 2002; Akbas and Shen, 2002; Leelataviwat *et al.*, 2009), that is mainly focused on satisfying the energy balance equation using a monotonically increasing deformation approach.

However, it has long been recognized that to understand the demands placed on structures during earthquakes one might also employ an energy-based approach, especially in assessing the damage potential of ground motions (Akiyama, 1985; Decanini and Mollaioli, 1998, 2001; Manfredi, 2001; Kalkan and Kunnath, 2007, 2008).

Energy-based methodologies, beyond the potentiality of designing earthquake-resistant structures by balancing energy demands and supplies, allow one to characterize the different types of time histories (impulsive, periodic, with long-duration pulses, etc.) which may correspond to an earthquake ground shaking properly, considering the dynamic response of a structure simultaneously (Mollaioli *et al.*, 2006).

Since energy is a cumulative measure of ground shaking, it also captures duration effects. In fact, it is well known that a certain amount of seismic damage can be due not only by the maximum response such as force or lateral displacement, but also by inelastic excursions below the maximum response. Therefore, energy demand parameters can be considered as reliable tools to use in seismic hazard analysis, for selecting earthquake scenarios, and designing a simulated earthquake for many types of engineering analyses (Chapman, 1999; Chou and Uang, 2000; Ghosh and Collins, 2002; Hao 2002; Hori and Inoue, 2002; Reinoso *et al.*, 2002; Riddell and Garcia, 2002; Ordaz *et al.*, 2003; Trifunac, 2008). Finally, using the energy concept and the energy balance equation allows one to optimize the design and detailing and to select strategies and techniques for innovative control or protective systems, such as base isolation and passive energy dissipation devices, in the earthquake-resistant design of new structures or in the seismic retrofitting of existing buildings.

Energy-based demand can be also considered as a useful link between the characterization of the seismic hazard based on available databases, containing data collected by existing networks, with that based on modelling techniques developed from the knowledge of the seismic source process and of the propagation of seismic waves, which can realistically simulate the ground motion associated with a given earthquake scenario (Decanini *et al.*, 1999; Panza et al., 2000, 2001, 2003a, b.

In order to characterize the damage potential of earthquake ground motion it is, therefore, necessary to properly evaluate the degree of correlation between energy demand parameters, herein considered as indexes of the seismic hazard at a given site, with structural response parameters, such as displacement and drift demands, that are representative of damage suffered by structures.

The fundamental steps of an energy-based methodology approach to the seismic demand evaluation for structures can be summarized in Fig. 1. Ground motion time histories, which may include both ground motion records (comprising accelerograms recorded in both ordinary and special site conditions such as on soft soil, in the near field, with long-duration pulses), and broad-band synthetic accelerograms, all characterized by an appropriate set of ground motion parameters, are used to perform dynamic analyses on SDOF and/or MDOF systems. Systems response to the energy and deformation demands imposed by the dynamic excitation and dependent on the hysteretic behavior, involving

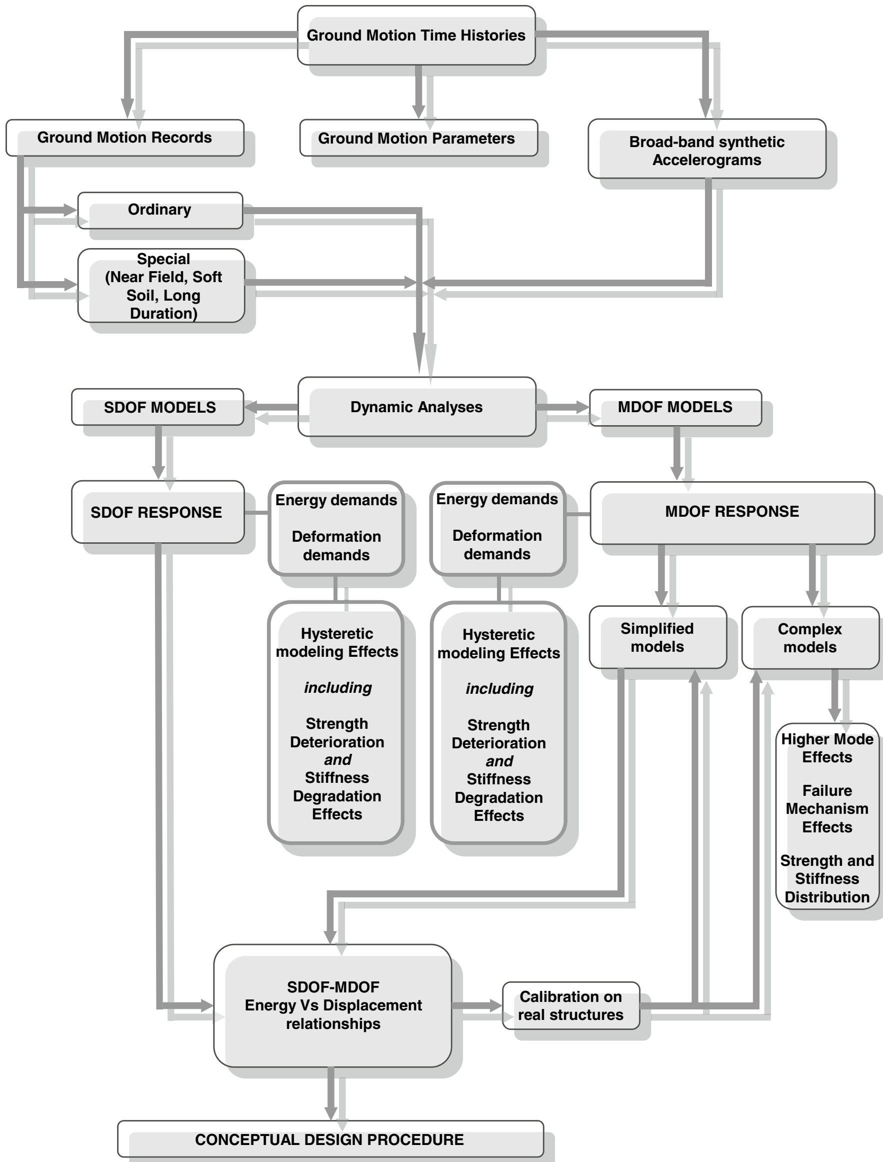

Figure 1
Seismic energy demand evaluation

strength deterioration and stiffness degradation effects, is therefore analyzed in order to relate appropriate energy quantities to significant kinematic and deformation parameters. From the comparison of the energy/displacement relationships derived for SDOF and MDOF systems, and suitably calibrated on real structures, simplified models validation and/or refinement ensue; furthermore complex models, envisaging a more accurate description of higher-mode effects, failure mechanism effects and strength/stiffness distribution, can be defined implemented. Final objective of such an energy-based approach to seismic analysis is the development of design procedures which, from the knowledge of the correlation between energy and displacement amount for SDOF and/or MDOF systems allows one either to define global and local displacement demands on the basis of energy design spectra, or conversely to establish energy demands on the basis of design displacement demands.

In the present paper, the characterization of the correlations between energy and displacement demands of linear and nonlinear single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems, representative of actual framed structures, is investigated. A simplified MDOF model is used so as to provide concise results on the seismic behaviour of the different structural systems subjected to such a large number of strong motion records. A stiffness-degrading hysteretic model is used to represent the cyclic behavior of the frames in each story.

In such analyses, the influence of source-to-site-distance, magnitude, soil class, and ground motion features (severity, duration, amplitude and frequency content) on earthquake input and hysteretic energy as well as their distribution in building systems, is examined.

Dynamic analyses are performed by subjecting the SDOF and MDOF systems to a large set of earthquake records, representative of different scenario earthquakes, including near-fault ground motions.

2. *Simplified Modeling of MDOF Systems*

Several research papers have assessed the energy and displacement demands in single-degree-of-freedom (SDOF) systems. Quite the reverse, only little attention was devoted to energy analyses for multi-story building structures (Chou and Uang, 2003; Decanini *et al.*, 2000; Estes and Anderson, 2002; Shen and Akbas, 1999; Leelataviwat, *et al.*, 2002); therefore, a broad extension of the evaluation of such demands to multi-degree-of-freedom (MDOF) systems is essentially needed.

As a matter of fact, there is a pressing requirement to carry out analytical and experimental studies on the possibility to consider the results obtained from analysis of SDOF systems also applicable to MDOF systems. Therefore, since real structures, and particularly frame building structures, can effectively be represented as MDOF systems, it is necessary to develop reliable methods for (1) evaluating how SDOF spectra for a given parameter can be affected by the MDOF response of real structures and (2) modifying the spectra obtained for SDOF systems so that the effects of MDOF systems behaviour are taken into account.

It must be stressed that nonlinear dynamic analyses of three-dimensional MDOF systems, besides requiring computational effort and time consumption, are founded on many assumptions involving many uncertainties, the reliability of the predicted performance is questionable and very difficult to ascertain. The main uncertainty factors include the reliability of the model adopted and its dynamic characteristics, particularly in its nonlinear range of behaviour, and the dynamic characteristics of the ground motion. Nevertheless, simplified MDOF models, such as stick models with one degree of freedom per floor, or even equivalent SDOF models, can successfully be adopted to represent low, medium and high-rise frame structures having sufficiently regular configuration and structural layout, with evident advantages in the accomplishment of a considerable number of dynamic analyses and of a wide-ranging parametric studies in reasonably extended lapses of time. Simplified models usually consider two-dimensional shear elements with only one translational degree of freedom per floor. Formulation of such simplified models requires the knowledge of the structure's reactive mass, lateral stiffness, vibration mode shape and frequency, and the lateral strength-displacement constitutive relation for each story. In the case of low-rise structures with regular configuration and structural layout in both plan and height, equivalent SDOF models may even be adopted.

Since the objective of the present research is to carry out the assessment of both energy and displacement demands in multi-degree-of-freedom (MDOF) systems, for the need of generality of the results and hence of the conclusions, the MDOF systems selected as representative of structural frame models are not intended to correspond to specific structures. Instead, a simplified model was used in this study for the estimation of the dynamic responses to different ground motions, so that the sensitivity of results to different mechanical properties can be readily evaluated. Dynamic analyses were performed using an equivalent discrete shear-type model (ESTM model), which allowed a relatively simple procedure in the integration of the equation of the motion

(Decanini *et al.*, 2002, 2004). The effectiveness of the ESTM model was also validated by comparing the results with those derived from more detailed non-linear time histories analyses (Mura, 2003; Decanini *et al.*, 2004). The implementation of a simplified modeling permitted us to achieve a spectral representation of energy and displacement demands at both global and local level and to perform an extensive parametric investigation on different structures.

In this paper, ten multi-story structural systems are modeled by using two-dimensional, two-bay generic frames with constant story height and beam spans (Fig. 2). The number N of stories considered in the analysis varies from 2 to 24 in order to simulate a significant range of typologies of reinforced concrete buildings. For the characterization of the inelastic response of multi-story frames subjected to ground motions, a further approximation was introduced in order to describe the hysteretic behavior of the system by means of simple rules. In this case it was necessary to define approximately a yielding resistance for each story.

The main aspects of the structural response of multi-story frame systems subjected to strong ground motions are connected with the mechanism of development of plastic deformations within the structure. In a well-designed frame, possibly according to the capacity design rules, the formation of soft-stories should be avoided taking care that the inelastic deformation demands are uniformly spread throughout the structure. This can be achieved provided that a well defined global mechanism, the so-called beam-sway mechanism, is ensured. The development of such a mechanism implies that plastic hinges are formed in all beams at all stories, while the strength capacity of columns and joints is preserved.

To this purpose, in the adopted simplified modeling, it was necessary to manage accurately the local inelastic behavior in the ESTM model that operates at story level.

By means of comparison with other research (Nassar and Krawinkler, 1991; Seneviratna and Krawinkler, 1997) and ad hoc analyses, the strain hardening was identified as a simple tool capable to ensure a beam-sway mechanism. Such assertion is based prevalently on the fact that, though significant, the stiffness reduction caused by the concentration of plastic deformations in the beams of a given story is less rapid than in occurrence of a story mechanism. For this reason the strain hardening ratio, p, was tuned to a value $p = 10\%$ for properly designed RC frames.

Another important issue is constituted by the choice of a data-set of accelerometric signals adequate

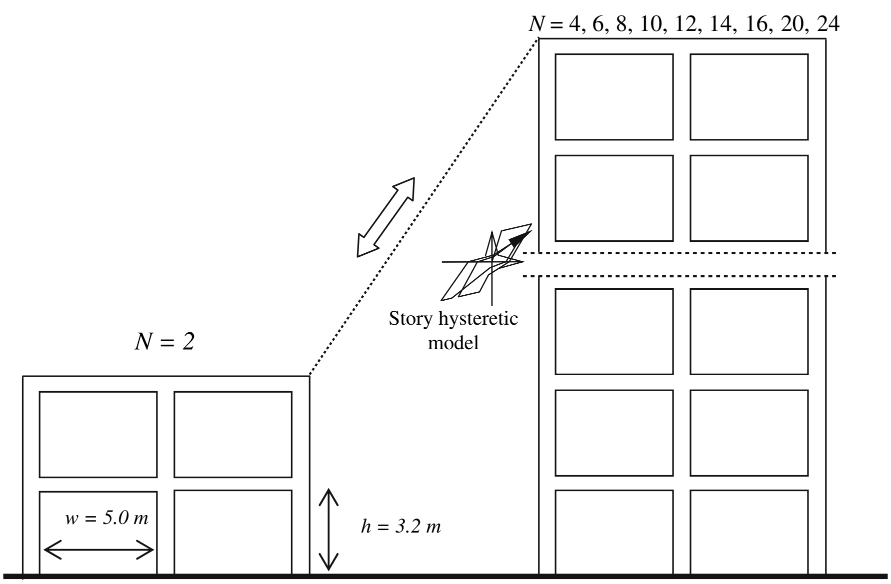

Figure 2
Structural layout of the analyzed frames

to represent the major characteristics of the seismic demand on the analyzed structures.

Recorded ground motions covering a broad variety of condition in terms of frequency content, duration and amplitude were appositely selected. In addition, from the recognition of the particular significance of pulse-like time histories in causing large energy and displacement demands to the structures, a considerable number of near-fault signals was included in the data-set of records, taking into account the presence of both forward and backward directivity effects.

3. *Correlation of Damage Measures to ground motion parameters*

Estimates of the seismic performance of structures has conventionally been based on taking the pseudo-spectral acceleration into account, among a variety of ground motion parameters. However, it is now widely recognized that performance predictions traditionally based on pseudo-spectral acceleration are subject of a great deal of variability, which in turn requires considerable effort to obtain results with a satisfactory level of confidence. This is the reason why, in the present study, the capability of various ground motion Intensity Measures in predicting deformation levels for a structure has preliminarily been investigated, in order to identify parameters providing the best levels of correlation to structural damage measures. Further developments may involve improving vector hazard capabilities, since the nonlinear response of many structures is described by multiple Intensity Measures (Luco *et al.*, 2005).

A series of MDOF systems, described through the ESTM models introduced in the previous section and representative of frame buildings of various heights and ductilities, were subjected to a large number (about 900) of strong ground motions recorded during 40 past and recent earthquakes (Table 1). Such records represent a significant variety of seismic conditions in terms of magnitude, source-to-site distance, fault mechanism. In particular, strong ground motions include near-source and ordinary ground motions, recorded at distances, D_f, less than and greater than 5 km from the causative fault, respectively; besides, they are grouped according to the EC8 four soil classes, A, B, C, D (with stiffness decreasing from A, rock or firm soil, to D, soft soil), and to two magnitude intervals, $5.4 \leq M_w \leq 6.2$ and $6.5 \leq M_w \leq 7.1$. Several ground motion parameters or intensity measures such as, for instance, peak ground acceleration, PGA, peak ground velocity, PGV, arias intensity, I_A, Housner intensity, I_H, spectral acceleration, S_a, input energy-equivalent velocity, defined as $V_{E_I} = \sqrt{2E_i/m}$ (where E_i is the input energy and m the system mass) were preliminarily been evaluated for each record. Then, ground motion parameters were related to the maximum interstory drift, $IDI_{\max}$, chosen as representative structural damage index, measured through the nonlinear dynamic analyses of the MDOF systems.

The extent of the correlation between the structural response index and ground motion intensity measures can be appreciated observing the relationships between these variables graphically (Figs. 3, 4). Data points shown on each plot result from nonlinear dynamic analyses with the selected ground motion

Table 1

Earthquakes list

Earthquake	Year	M_w	Earthquake	Year	M_w
Livermore	1980	5.4	Dinar	1995	6.2
Friuli 11-09-1976 16.35.01	1976	5.5	Managua, Nicaragua	1972	6.2
Lazio-Abruzzo 1984	1984	5.5	Morgan Hill	1984	6.2
Sicilia Sud Orientale 1990	1990	5.6	Friuli, Italy	1976	6.5
Umbria 1984	1984	5.6	Imperial Valley	1979	6.5
Coyote Lake	1979	5.7	Kozani	1995	6.5
Mammoth Lakes	1980	5.7	Borrego Mtn	1968	6.6
NE of Banja Luka (Bosnia)	1981	5.7	San Fernando	1971	6.6
Umbria-Marche	1997	5.7	Northridge	1994	6.7
Norcia (Valnerina)	1979	5.8	Superstition Hills (B)	1987	6.7
San Salvador	1986	5.8	Gazli, USSR	1976	6.8
Westmorland	1981	5.8	Nahanni, Canada	1985	6.8
Athens (Greece)	1999	5.9	Spitak, Armenia	1988	6.8
Kalamata (Greece)	1986	5.9	Erzincan, Turkey	1992	6.9
Basso Tirreno 1978	1978	6	Irpinia, Italy	1980	6.9
Friuli 15-09-1976 09.21.18	1976	6	Kobe	1995	6.9
N. Palm Springs	1986	6	Loma Prieta	1989	6.9
Umbria-Marche	1997	6	Montenegro (YU)	1979	6.9
Whittier Narrows	1987	6	Cape Mendocino	1992	7
Parkfield	1966	6.1	Imperial Valley	1940	7

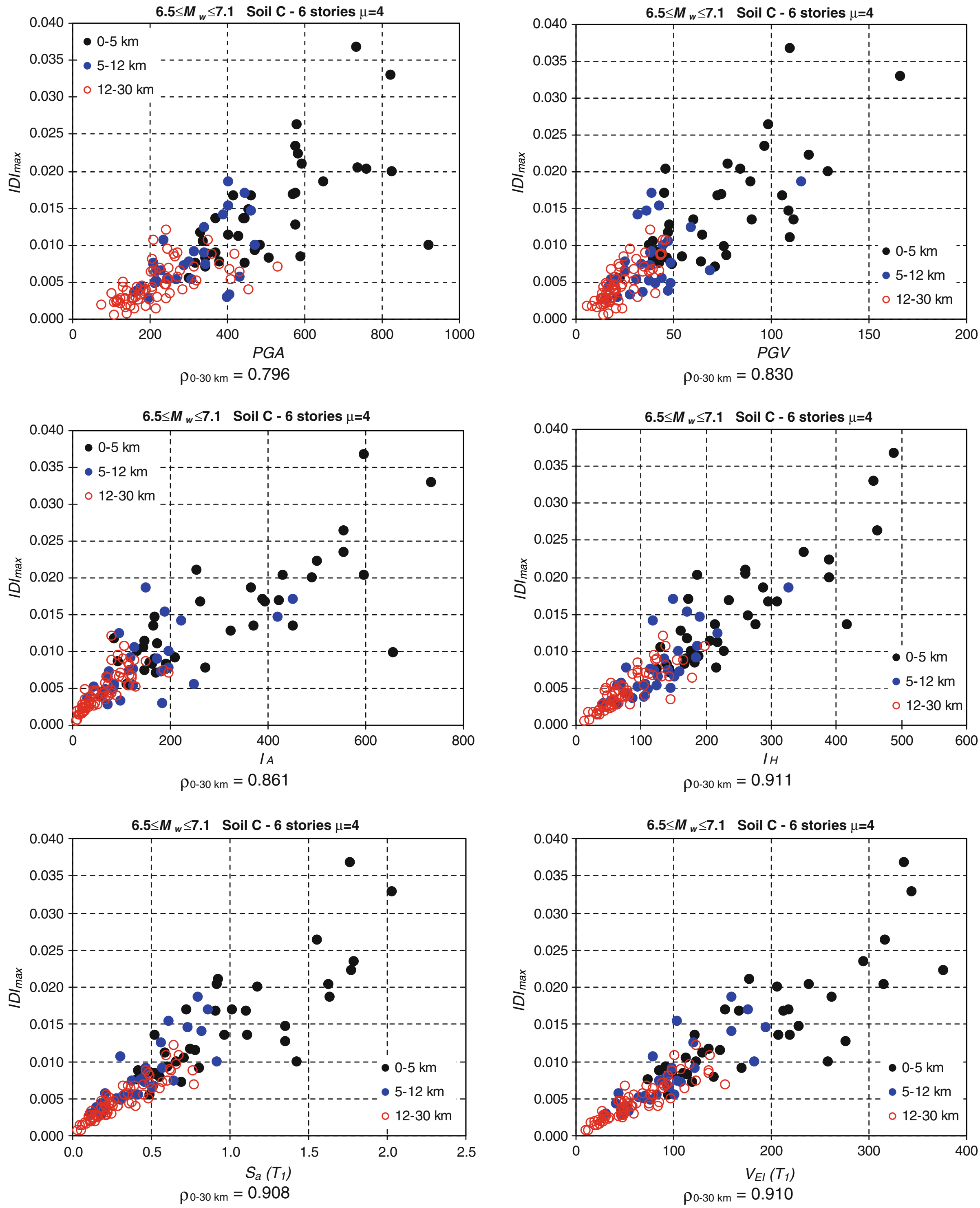

Figure 3
Correlation of various intensity measures with the maximum interstory drift for MDOF systems

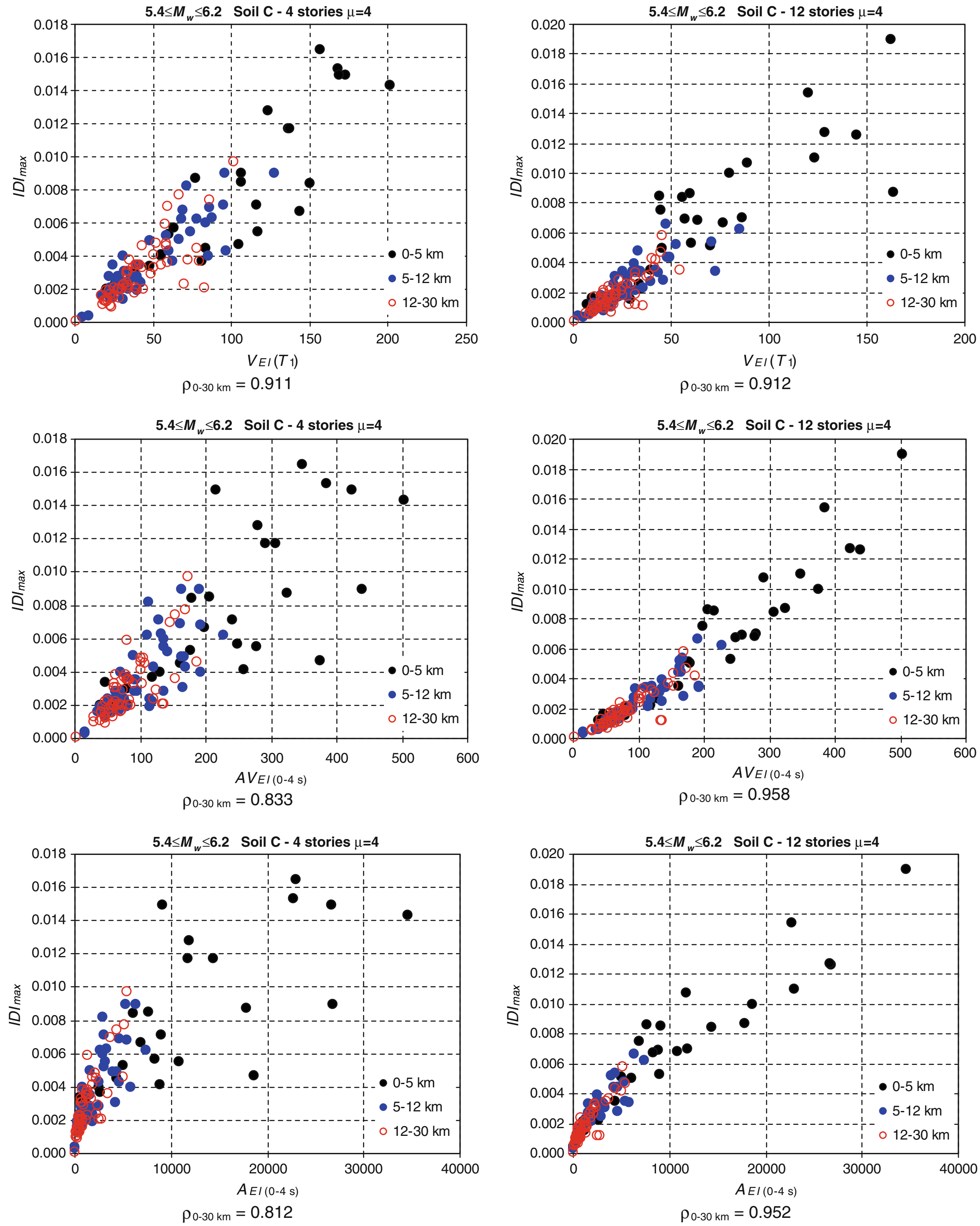

Figure 4

Correlation of energy-based intensity measures with the maximum interstory drift for low and medium/high-rise frames records. Diagrams in Fig. 3 refer, for instance, to medium-rise structures (6 story-buildings) with ductility ratio $\mu = 4$, on intermediate soil, type C. Relevant analyses were performed with ground motions recorded from events with magnitudes $6.5 \leq M_w \leq 7.1$, and at various intervals of distance from the causative fault; the correlation coefficient $\rho_{0\text{–}30\ \mathrm{km}}$, also indicated, refers to the entire interval of considered distances. Conventional intensity measures, such as PGA and PGV, do not appear to exhibit satisfactory correlation with the chosen damage measure.

The correlation level increases slightly in case the Arias intensity, I_A, is taken into account; considerable scatter persists for records obtained in near-fault conditions. Better results are instead obtained as long as Housner Intensity, I_H, spectral acceleration corresponding to the fundamental period T_1, $S_a(T_1)$, and input energy-equivalent velocity at T_1, $V_{E_I}(T_1)$, are considered. Pseudo-spectral acceleration, $S_a(T_1)$, one of the most widely adopted spectral quantitities, appears to constitute in this case an effective intensity measures. However, being a displacement-based parameter, it only reflects the effects of amplitude and does not allow one to take the influence of the ground motion duration into account. Therefore, its strong correlation with structural response is limited to cases of moderately inelastic systems, whose behaviour is governed by the fundamental mode of vibration; the contributions to the response from higher modes of vibration can be significant under such circumstances (Bazzurro and Cornell, 2002).

Effects of inelasticity and ground motion duration that are overlooked by the more conventional spectral parameters are instead implicitly captured by I_H and $V_{E_I}(T_1)$. In particular, the input energy-equivalent velocity, being an energy-based ground motion parameter, is directly related to the number and amplitudes of the cycles of oscillator response: hence, it conveys information on the ground motion duration, reflecting cumulative effects by virtue of the integration over time that is involved in their computation.

Figure 4 shows the correlation of some significant energy-based ground motion parameters, namely $V_{E_I}(T_1)$, already introduced, the area of the V_{E_I} spectrum in the range of periods between 0 and 4.0 s, $AV_{E_I 0-4s}$, and the area of the E_i spectrum in the same range of periods, AE_{I0-4s}. Plots refer to 4-story (left side) and 12-story frames (right side), moderately inelastic ($\mu = 4$), on soil of Class C. A marked improvement in the correlation strength can be observed for high-rise buildings in comparison with low-rise buildings; at the same time, the considerable dispersion affecting the results obtained with near-source ground motions appears significantly reduced for buildings of the former typology.

4. Relationships between Energy and Displacement Demand

The identification of reliable relationships between energy and displacement demands represents a fundamental issue in both the development of more reliable seismic code provisions and the evaluation of seismic vulnerability aimed to the upgrading of existing hazardous facilities.

Even though the seismic performance of a structure is directly related to the global and local deformations of the structure, the energy balance formulation appears much more effective in concept, as it permits a rational assessment of the energy absorption and dissipation mechanisms that can be effectively accomplished to balance the energy imparted to the structure. As the two aspects could become consistently integrated within a performance-based seismic design methodology, understanding how input and dissipated energy are correlated with displacement demands emerges as a decisive prerequisite.

For SDOF systems, Fajfar (1992), and Fajfar and Vidic (1994) proposed a non-dimensional parameter, γ, defined by the following formula:

$$\gamma = \frac{\sqrt{E_H/m}}{\omega\delta} \tag{1}$$

where E_H is the dissipated hysteretic energy, m is the mass of the system, ω is the natural circular frequency and δ is the maximum displacement of the system. This expression, which represents a normalization of the dissipated hysteretic energy, can be read as the ratio between two equivalent velocity amounts. The above relationship was introduced by

FAJFAR and GASPERSIC (1996) in the so-called *N2 method*, i.e. a nonlinear methodology for the two-dimensional seismic analysis of reinforced concrete buildings vibrating predominantly in the first mode, both in the elastic and in inelastic range. As also shown by DECANINI *et al.* (2000), there exists a stable, approximately parabolic relationship between global displacement (herein named *roof displacement*, δ_{roof}) demand and hysteretic energy, E_H, demand for equivalent single-degree-of-freedom (ESDOF) systems. Such ESDOF systems were modeled by assuming an assigned time-independent global deflection shape for the lateral displacements of the structure, thus restricting its dynamic response to the first vibration mode only.

More stable quantities than that obtained with the γ parameter can be yielded by setting a relation between the square root of the input energy and the displacement. As a matter of fact input energy, E_I, represents an effective tool in the characterization of the seismic demands; moreover, it constitutes a very stable parameter, scarcely depending on the hysteretic properties of the structure. A new parameter, ζ, was then defined as follows (TERAN-GILMORE, 1996, 1998):

$$\zeta = \frac{\sqrt{E_I/m}}{\omega\delta} \tag{2}$$

where E_I is the input energy, m is the mass of the system, ω is the natural circular frequency and δ is the maximum displacement of the system. In particular, it was found that the quantity above is more stable than that expressed by γ (DECANINI and Mollaioli 2001; DECANINI *et al.*, 2000).

As a general rule, methods based on the analysis of the dynamic response of SDOF or ESDOF systems permit one to obtain a quite good evaluation of global displacement and energy demands. However, as they neglect significant effects on the seismic response of multistory frames due to higher modes, they cannot be used for the estimation of local energy and displacement demands; besides, it should be considered that the local seismic behavior is also strongly affected by the particular characteristics of the strong ground motions. Therefore, in order to extend the characterization of the relationships between energy and displacement demands to multi-degree-of-freedom (MDOF) systems, in the present work two further pairs of parameters are defined, one that relates input or hysteretic energy with global displacements, the other that correlates energy quantities with interstory drifts.

The first two parameters, ζ' and γ', respectively depending on the input energy, E_I and on the dissipated hysteretic energy, E_H, can be directly derived by utilizing modified expressions of formulae derived for the SDOF systems, Eqs. 2 and 1 in that order, as

$$\zeta' = \frac{\sqrt{E_I/m}}{\omega_1\delta_{\mathrm{roof}}} \tag{3}$$

$$\gamma' = \frac{\sqrt{E_H/m}}{\omega_1\delta_{\mathrm{roof}}} \tag{4}$$

where m is the total mass of the system, ω_1 is the fundamental frequency, and δ_{roof} is the top displacement. The definition of the second two parameters, γ'' and ζ'', required instead further modifications of Eqs. 2 and 1 respectively. Substitution of the global displacement, δ, with the inter-story drift, $\Delta_{\max}$, in Eqs. 2 and 1 leads to the following expressions:

$$\zeta'' = \frac{\sqrt{E_I/m}}{\omega_1 \cdot N \cdot \Delta_{\max}} = \frac{\sqrt{E_I/m}}{\omega_1 \cdot N \cdot h \cdot IDI_{\max}} \tag{5}$$

$$\gamma'' = \frac{\sqrt{E_H/m}}{\omega_1 \cdot N \cdot \Delta_{\max}} = \frac{\sqrt{E_H/m}}{\omega_1 \cdot N \cdot h \cdot IDI_{\max}} \tag{6}$$

where N is the number of stories, h is the story height and $IDI_{\max} = \Delta_{\max}/h$ is the maximum interstory drift index. The quantities above have proved (MURA, 2003) to be capable of yielding the best results in terms of statistical stability with regard to the characteristics of the ground motion records considered.

Introducing the coefficient of distortion, COD, defined as the ratio of the maximum interstory drift index, $IDI_{\max}$, to the average interstory drift index, IDI_{avg} (TERAN-GILMORE, 1998), in Eqs. 5 and 6 and considering frame structures having the same height in all stories, so that the total height of the frame and the maximum interstory drift index are expressed by

$$H = Nh \tag{7a}$$

$$IDI_{\max} = \mathrm{COD} \cdot IDI_{\mathrm{avg}} = \mathrm{COD}\frac{\delta_{\mathrm{roof}}}{H} \tag{7b}$$

which implies

$$N \cdot h \cdot IDI_{\max} = H \cdot IDI_{\max} = \mathrm{COD} \cdot \delta_{\mathrm{roof}} \tag{8}$$

lead to the following relationships:

$$\zeta'' = \frac{\zeta'}{\mathrm{COD}} \tag{9}$$

$$\gamma'' = \frac{\gamma'}{\mathrm{COD}} \tag{10}$$

From an operational point of view, once the correlations between these parameters and structural quantities, such as the fundamental period, are known (for instance, through regression analysis), seismic performance indexes (i.e., the maximum interstory drift and the top displacement) can directly be derived (Fig. 5). In cases for which it is reasonable to resort to SDOF systems in the simplified modelling of low-rise buildings, in particular, the top displacement can be approximated with the maximum oscillator displacement.

5. *Influence of Ground Motion and Structure Characteristics on Energy-Based Parameters*

To the purpose of assessing the influence of structural and nonstructural (i.e., peculiar to the ground motion) features on the seismic behavior, a series of SDOF systems were subjected to a total of 45 records chosen from the whole data set in order to represent different seismic conditions in terms of magnitude, source-to-site distance, fault mechanism. In particular, this subset of strong ground motions includes near-source and ordinary ground motions, recorded at distances, D_f, less than and greater than 5 km from the causative fault, respectively. In Table 2 the selected time histories are listed, while in Fig. 6 the distribution of the records according to magnitude, M_w, and the closest distance from the surface projection of the fault rupture, D_f, is shown.

Generally the shapes of the spectra of the inter-story drift index, *IDI*, defined as the inter-story displacement normalized by the story height, are more influenced by the signal energy content than the displacement spectra. This circumstance emerges from the comparison between energy and drift spectra, which indicate both spectral coincidence of periods where the maximum values are attained and agreement in the general trend. In contrast, similar correspondences are not recognizable in the displacement spectra shapes. This should mean that a strong energy demand imposed to a structural system does not necessarily cause a strong top displacement,

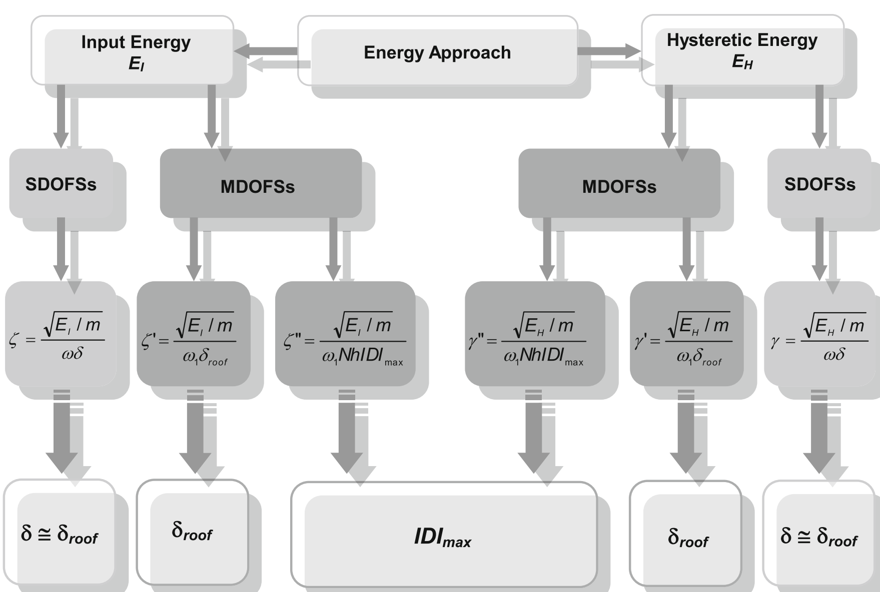

Figure 5
Energy parameters for the evaluation of seismic performance

Table 2

Strong motions employed for SDOF systems analysis

Earthquake	Year	M_w	Recording Station	*PGA* (cm/s^2)	Earthquake	Year	M_w	Recording Station	*PGA* (cm/s^2)
Imperial Valley	1940	7.0	El Centro Array #9	342	Loma Prieta	1989	6.9	Capitola	463
Tokachi-Oki	1968	7.9	HachinNS	312	Landers	1992	7.3	Lucerne #	713
Tokachi-Oki	1968	7.9	HachinEW	206	Landers	1992	7.3	Joshua Tree #	278
Friuli	1976	6.5	Tolmezzo	348	Erzincan	1992	6.7	Erzincan	505
Romania	1977	7.5	Bucharest-BRI	202	Northridge	1994	6.7	Rinaldi Receiving Sta #	822
Tabas, Iran	1978	7.4	Tabas	835	Northridge	1994	6.7	Sylmar - Olive View FF	827
Montenegro	1979	6.9	Bar-Skupstina Opstine	356	Northridge	1994	6.7	Newhall - Fire Sta	579
Montenegro	1979	6.9	Petrovac-Hotel Oliva	446	Northridge	1994	6.7	Santa Monica City Hall	866
Montenegro	1979	6.9	Ulcinj-Hotel Olimpic	236	Kobe	1995	6.9	Kobe University	285
Imperial Valley	1979	6.5	El Centro Array #7	454	Kobe	1995	6.9	KJMA	806
Imperial Valley	1979	6.5	Agrarias	217	Kobe	1995	6.9	Takatori	599
Imperial Valley	1979	6.5	Bonds Corner	760	Kobe	1995	6.9	Port Island (0 m)	309
Imperial Valley	1979	6.5	Calexico Fire Station	270	Dinar	1995	6.4	Dinar	345
Imperial Valley	1979	6.5	EC Meloland Overpass FF	291	Kocaeli	1999	7.4	Sakarya	369
Imperial Valley	1979	6.5	El Centro Array #6	431	Kocaeli	1999	7.4	Yarimca	342
Irpinia	1980	6.8	Calitri	153	Kocaeli	1999	7.4	Duzce-Met.	351
Cile	1985	7.8	Llolleo	698	Duzce	1999	7.1	Duzce-Met.	525
Messico	1985	8.1	Secretaria com. & tran.	168	Duzce	1999	7.1	Bolu	807
Nahanni	1985	6.8	Site 1	1,075	ChiChi	1999	7.6	TCU129	991
San Salvador	1986	5.8	Geotech Investig Center	681	ChiChi	1999	7.6	TCU052	341
Superstitn Hills	1987	6.5	Parachute Test site	446	ChiChi	1999	7.6	TCU068	555
Loma Prieta	1989	6.9	Los Gatos Pres. Center	552	ChiChi	1999	7.6	TCU065	799
Loma Prieta	1989	6.9	Corralitos	617					

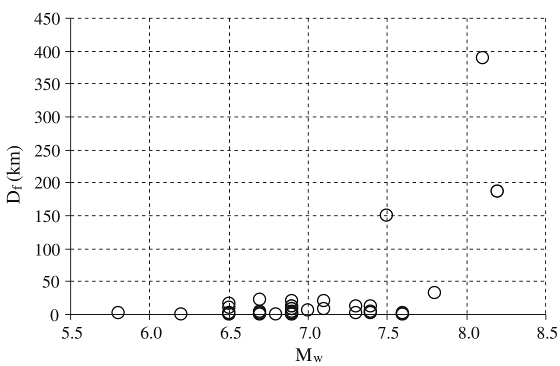

Figure 6
Magnitude versus minimum distance to the causative fault

while a strong local deformation demand is quite likely to occur.

The trend to assume the same shape of the input energy spectra is confirmed by the frequent coincidence of periods between energy and drift peaks and valleys. In Fig. 7 the spectral shapes of the maximum interstory drift, $IDI_{\max}$, estimated in this study, are compared to the spectral shapes, relevant to SDOF systems, of the input energy, E_I and the acceleration, S_a. The spectra were computed for ground motions of very different characteristics. More specifically Erzincan (Fig. 7a) represents a typical forward directivity pulse-like time history, while Hachinoe (Fig. 7b) stands for a far-field long duration record.

However, the maximum drift is greatly influenced by both the amount of energy input and the way the energy is imparted to the structure. A greater rate of input in the energy demand time history influences strongly the drift demand for pulse-like time histories in comparison with long duration motions, which of course could impart the same amount of energy to the structure, but in a longer time. As a matter of fact, near-fault records could induce a lower number of cycles in the structure than far-field ones, but with a higher energy content. This circumstance can account for a larger concentration of the drift demand.

5.1. *Influence of the Stiffness Distribution Pattern*

One aspect of the dependency of the selected parameters for the quantification of the seismic demands in terms of energy and displacement on structural properties of the considered frames is

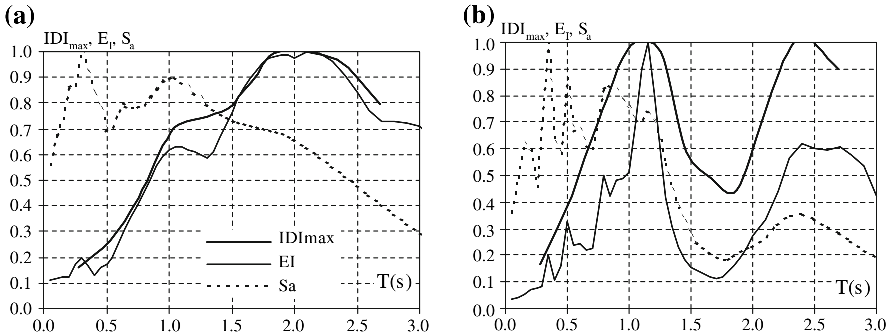

Figure 7
Comparison between normalized elastic spectra of $IDI_{\max}$, E_I and S_a. Erzincan (**a**), Hachinoe (**b**)

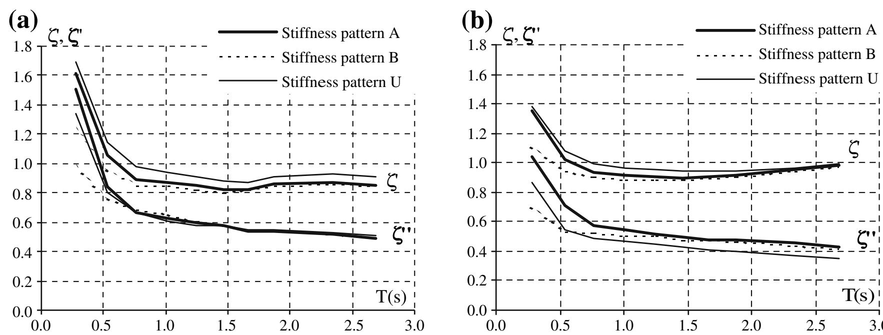

Figure 8
ζ' and ζ'' spectra. Comparison between stiffness distribution patterns. Median all records. **a** Elastic; **b** $\mu = 4$ (hysteretic model DGR3)

constituted by the influence of the stiffness distribution patterns through the height of the multi-story frames.

For this reason each selected frame was designed according to three different stiffness patterns, namely (1) a realistic approximately parabolic stiffness distribution with full constraint for the joints at the base of the columns (stiff foundation structures), denoted as *stiffness pattern A*; (2) the same stiffness distribution as pattern *A* but in presence of a flexible foundation, denoted as *stiffness pattern B*; (3) a uniform distribution of stiffness along the height, defined by calibrating the stiffness so as to obtain the same vibration periods as the stiffness distribution *A*, denoted as *stiffness pattern U*.

Generally, the parameters ζ' and ζ'' are not significantly influenced by the stiffness distribution along the height of the frames, either in the elastic and in the inelastic range (Fig. 8a, b, respectively).

The lowest values of the parameter ζ'' can be easily explained by the fact that it is derived from ζ' by division by the coefficient of distortion COD, which is a quantity always greater than unity. The behaviour exhibited by scheme *B* differs from that of scheme *A* only in the low-period range, due to the larger influence of the stiffness reduction at the first story, while assuming the uniform stiffness distribution *U* appears to produce considerable divergences from spectra computed according to the other two patterns. Analogous consideration can be made for the parameters γ' and γ''; however, divergences of spectra relevant to the stiffness distribution pattern *U* from those relevant to *A* and *B* schemes appear more marked than those observed for ζ' and ζ'' (Fig. 9a, b).

Although the ground motions utilized in this study differ noticeably in terms of frequency content, duration and amplitude, it was possible to make some consideration on statistical grounds. First of all, it was noted that the value of the coefficient of variation, COV, ranges between 0.15 and 0.40 for the four parameters ζ', ζ'', γ', γ'' and the whole dataset of records, both in the elastic and inelastic range. It was

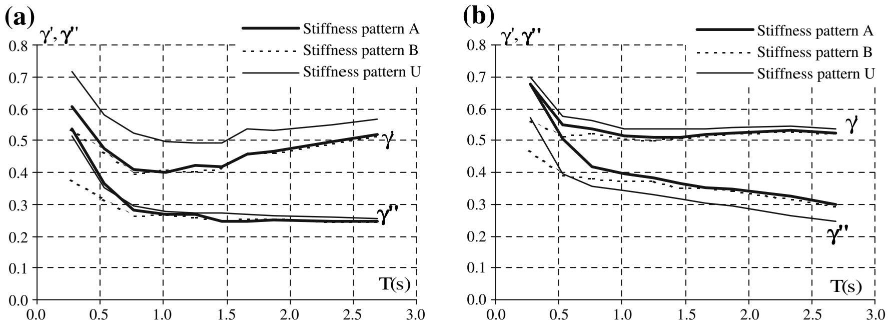

Figure 9
γ' and γ'' spectra. Comparison between stiffness distribution patterns. Median all records. **a** $\mu = 1.5$; **b** $\mu = 4$ (hysteretic model DGR3)

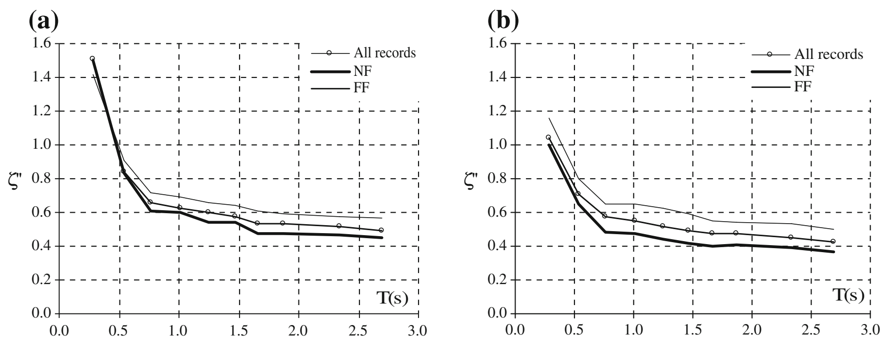

Figure 10
ζ'' spectra. Median all, Near-fault (NF), Far field (FF) records. Stiffness distribution patterns A. (**a**) Elastic; (**b**) $\mu = 4$ (hysteretic model DGR3)

also observed that ζ'' has lower COV values than ζ', particularly in the medium-long period range, indicating that the energy demand is better correlated to the interstory drift than to the roof displacement demand. A large dispersion was observed in the high-frequency region, principally for the two-story frames in the elastic range, consistently with the behaviour already detected for SDOF systems (Decanini *et al.*, 2000) Similar considerations can be made with reference to the inelastic range, with the exception of a significant decrease of the dispersion for the lowest frames.

5.2. *Influence of Source-to-Site Distance*

The influence of source-to-site distance, D_f, deserves further consideration. In Figs. 10 and 11 the median spectra of the parameters ζ'' and γ'', grouped as near-fault, NF, and far-field, FF, records, together with the median spectra of all records, are shown for the elastic case and ductility ratios μ equal to 1.5 and 4. It is possible to detect that while the spectral shapes are very similar, the NF spectral values are systematically lower than the other ones, indicating a largest deformation demand associated to a very limited number (one or two) of loading cycles. This effect is due to the presence of long duration pulses in near-fault time histories causing a sudden energy dissipation in a short time. Displacements of large amplitude takes place consequently, with corresponding relatively lower ratios between the square root of the energy demand and the displacement demand. On the contrary, the energy demand on a structural system subjected to a far-field motion tends to gradually increase during a longest time interval causing an incremental build up of deformations. Similar results were also found for the parameters ζ' and γ'.

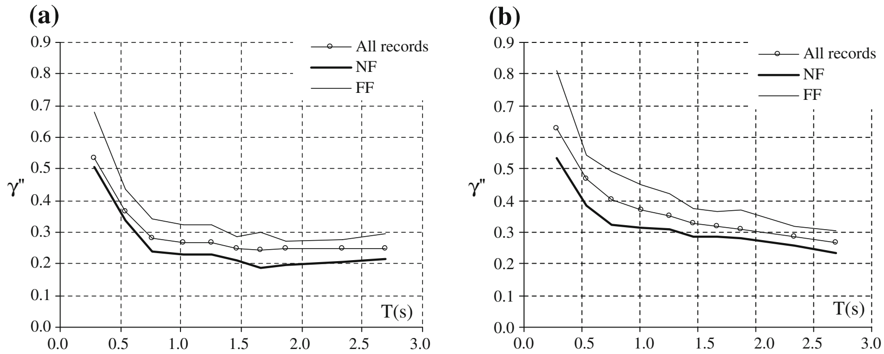

Figure 11
γ'' spectra. Median all, Near-fault (NF), Far field (FF) records. Stiffness distribution patterns A. (**a**) $\mu = 1.5$; (**b**) $\mu = 4$ (hysteretic model DGR3)

Table 3

Characteristic of the hysteretic models used in this study

EPP	
DGR1	Reloading stiffness degradation
DGR2	Reloading stiffness degradation, strain hardening ratio $p = 10\%$
DGR3	Reloading and unloading stiffness degradation, strain hardening ratio $p = 10\%$
DGR4	Reloading and unloading stiffness degradation, pinching effect, strain hardening ratio $p = 10\%$
DGR5	Reloading and unloading stiffness degradation, strength degradation, strain hardening ratio $p = 10\%$

5.3. Influence of the Hysteretic Model

A stiffness-degrading hysteretic model was then adopted to simulate the cyclic behavior of the frames in each story. Many hysteretic models were proposed to predict the nonlinear behavior of reinforced concrete structural systems. Such models are typically characterized by control parameters, which govern the shape of the generated loops, that have to be calibrated from observed experimental testing. Moreover, in highly nonlinear material like reinforced concrete the system characteristics are recurrently altered by either degradation of strength and stiffness or apparent pinching of loops resulting from repeated opening and closing of cracks. Consequently, most computational algorithms included in several codes generally utilize simplified piece-wise, linear hysteretic macro-models which relate force and deformation or moment and curvature in the inelastic range. Such macro-models have the advantage of capturing the global behavior in an equivalent sense without resorting to complex finite element discretization.

The force–deformation behavior in concrete structures is influenced by many factors such as the nature of the response and the level of the reinforcement detailing. In such cases, it is necessary to include more than one type of degrading behavior to model the expected response. Two extreme patterns of behavior can be considered: stable flexural response, characteristic of well-detailed, well-confined flexural elements, and severely degraded response, likely to be observed in shear-critical elements such as inadequately reinforced concrete joints and poorly confined short columns or coupling beams.

Thus, on the basis of the aforementioned consideration, six hysteretic models were selected, namely the elastic-perfectly plastic model, EPP, introduced as a term of comparison, and five degrading models, DGR1-5 (Table 3).

The hysteresis model used in this study, represented in Fig. 12, utilizes several control parameters that establish the rules for inelastic loading reversals. Stiffness degradation is modeled as a function of

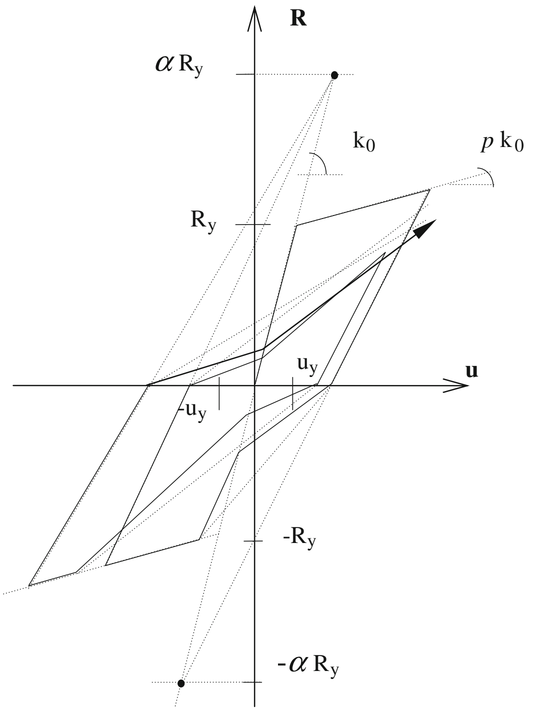

Figure 12
Tipical cyclic behavior of the four parameter hysteretic model

ductility, strength deterioration is modeled as a function of both dissipated energy and ductility and pinching is modeled by two independent parameters to control the degree of pinching.

The constitutive law is controlled by four parameters, namely p, α, β, γ. The parameter p, the strain hardening ratio introduced above, controls the post-yielding stiffness and is set equal to 0.1; α is related to the unloading stiffness; β controls the strength degradation; γ controls the pinching effect due to closing cracks during the reloading phase. If α tends to infinity there is no stiffness degradation during the unloading phase; the value $\alpha = 2$ represents a mean value. For $\beta = 0$ there is no strength degradation due to energy dissipation; the value $\beta = 0.1$ corresponds to a mean value for which this effect is quite reasonable, as for larger values it can lead to unrealistic results or produce numerical instability. For $\gamma = 1$ there is no pinching, while $\gamma = 0$ corresponds to the maximum pinching effect; $\gamma = 0.5$ can be considered a realistic values even though it leads to quite significant effect. It is also possible to note that, unless caused by secondary effects on more internal cycles, the DGR1 and DGR2 models correspond to the Clough model without and with strain hardening, respectively, while the DGR3 model matches the Takeda model.

The influence of the hysteretic behaviour is generally not significant, though with some exceptions. In Figs. 13 and 14 the median spectra of the parameters ζ', ζ'' and γ', γ'', respectively, are shown for a ductility ratio $\mu = 4$. The trend of the degrading models is substantially different from that of the EPP model, particularly for the parameters ζ'' and γ''. This can be attributed the fact that in a degrading hysteretic behaviour the maximum displacement is directly related to the amplitude of the instantaneous oscillation rather than to a residual cumulated displacement, as it usually occurs for the EPP model.

The spectra corresponding to the degrading models DGR3, DGR4 and DGR5 are very close in the whole period range and for the various ductility ratios.

As it was observed for the influence of the stiffness distribution pattern, in the high frequency range the parameters ζ', ζ'' and γ', γ'' exhibit the maximum spectral values, that tend to attenuate as the ductility increases. This effect is prevalent for the parameters ζ' and ζ''. For example, as far as ζ' is concerned, it could be mainly attributable to the way the absolute input energy is defined, and therefore can be explained in the light of behavior of SDOF systems. In fact, for SDOF systems in the elastic case the input energy tends to the value $\mathrm{PGV}^2/2$, where PGV is the peak ground velocity, for $T \to 0$. Consequently its square root moves to $\mathrm{PGV}/\sqrt{2}$, while the displacement δ tends quickly to 0 so as $\omega\delta = 2\pi\delta/T$ tends to 0. The analogy with the behavior of MDOF systems comes out as soon as one recognizes that the input energy and the displacement δ mentioned above correspond, apart from a participation factor raised to the first and to the second power, respectively, to the quantities δ_{roof} and E_I in Eq. 3. In the inelastic range for the lowest periods there is a slight energy modification while the displacement amplification is significant ($R_\delta \to \mu$ for $T \to 0$) and therefore the values of ζ' and ζ'' are lesser.

Finally, it was observed that ζ' and ζ'' spectra are not much influenced by the ductility ratio, while γ'

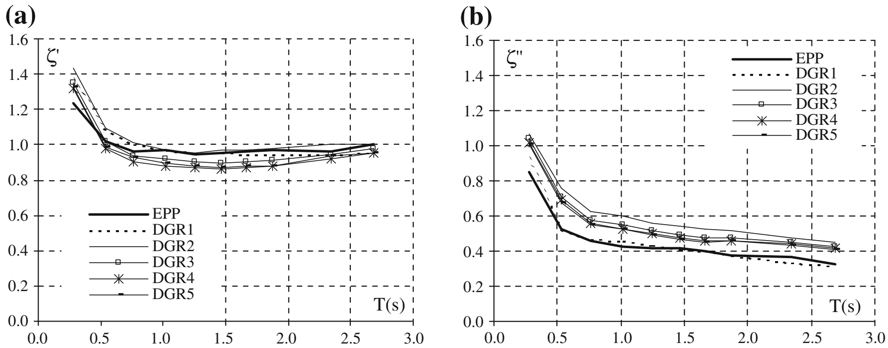

Figure 13
ζ' (**a**) and ζ'' (**b**) spectra. Comparison between hysteretic models. Median all records. Stiffness distribution pattern A. $\mu = 4$

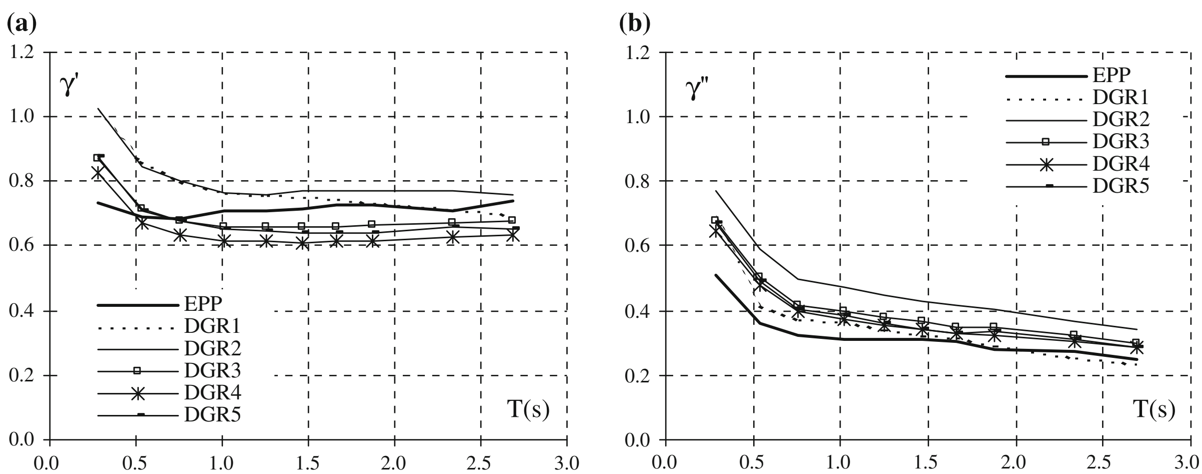

Figure 14
γ' (**a**) and γ'' (**b**) spectra. Comparison between hysteretic models. Median all records. Stiffness distribution pattern A. $\mu = 4$

and γ'', related to the dissipated hysteretic energy, are more affected by ductility, especially in the low ductility range.

6. Evaluation of Energy-Based Parameters for SDOF and MDOF Systems

The energy-based parameters introduced in the previous sections were evaluated subjecting both SDOF systems and MDOF systems, modelled using the equivalent shear-type model, ESTM, to the strong ground motion records of the entire database of accelerograms. In the light of the scarce significance of the hysteretic behaviour influence on such parameters, the nonlinear analyses were limited to models characterized by the constitutive law indicated as DGR3 (Table 3).

Figures 15 and 16 show diagrams of mean γ, γ' and γ'' values, in black, red and blue lines, respectively. The abscissa values must be intended as oscillator periods as long as γ curves, and hence SDOF systems, are concerned, while they represent the fundamental period of the structure (ranging from 0.281 s for two-story frames, to 2.689 s for 24-story frames), as long as γ' or γ'', and hence MDOF systems, are concerned. Figure 15 refers to an interval of magnitudes ranging from 6.5 to 7.1, to the four different soil classes and moderately inelastic systems, with $\mu = 4$. Curves in each plot are evaluated for different intervals of distance to the seismic source; thicker lines refer to mean values over the whole

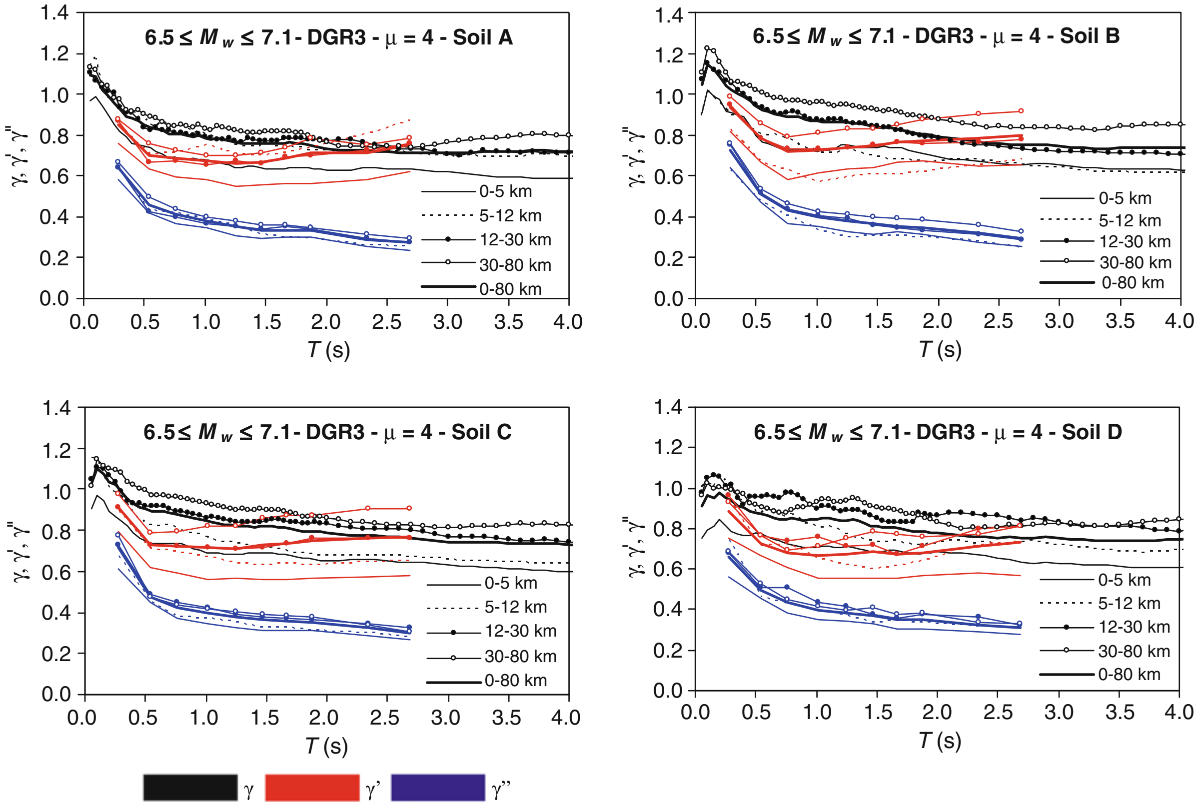

Figure 15
γ, γ' and γ'': Soils A, B, C, D; DGR3, $\mu = 4$, $6.5 \leq M_w \leq 7.1$

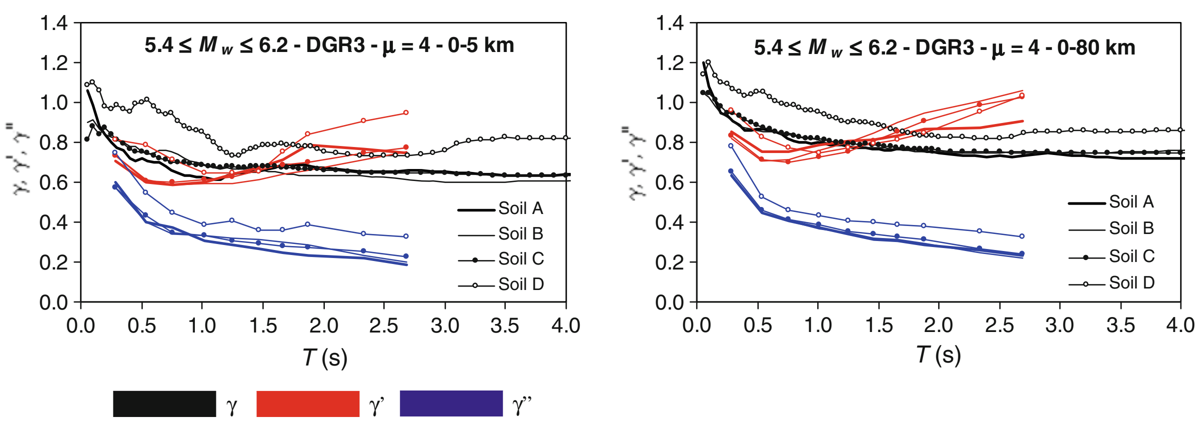

Figure 16
γ, γ' and γ'': near-fault and ordinary records; DGR3, $\mu = 4$, $5.4 \leq M_w \leq 6.2$

range of distances. Figure 16 refers to an interval of magnitudes ranging from 5.4 to 6.2; the left-side diagram shows values determined with near-fault records, while the right-side one refers to the whole data set.

It appears that the parameter γ may provide a satisfactory estimation for γ'. In the low to medium-period range, from the definition of γ and γ', Eqs. 1 and 4, respectively, values of δ_{roof} are slightly greater than δ; the reverse occurs as the fundamental period of the MDOF system increases. This can be explained in light of the fact that the deflection of a MDOF system representative of low-rise buildings is prevailingly governed by the first-mode shape, and of the reduced stiffness in comparison with that of a SDOF system with period, T, coincident with the MDOF system fundamental period, T_1. On the contrary, the dynamic behaviour of high-rise buildings is affected by higher modes, especially in the case of near-fault records, and maximum displacement may not necessarily be attained at the top-story level. Correspondingly, δ tends to exceed δ_{roof} in the medium to high-period range, providing a conservative estimation for the latter.

Curves relevant to the parameter γ'' always lie below those relevant to γ and γ'; while they originate in proximity of these curves, they tend to diverge rapidly as the fundamental period of the structure increases. Besides, all γ'' curves lie far below the line γ''=1, which can be interpreted, according to the definition given in Eq. 6, to the ratio of two velocity quantities. This indicates scarce correlation between dissipated energy, E_H, and IDI_{max}, perfect correlation being attained as the velocity value expressed as a function of the dissipated energy, numerator of Eq. 6, equals the velocity value expressed as a function of the displacement, denominator of Eq. 6.

Figures 17, 18, 19, 20 refer to the parameters ζ, ζ' and ζ''. The trend appears more regular in the elastic range (Figs. 17, 18). As far as the inelastic behaviour is concerned, a behaviour similar to that observed for γ and γ' can be observed for ζ and ζ', respectively. However, differently from the case of γ'', curves relevant to the parameter ζ'', though decreasing beyond the low-period range, appear to assume a stable trend as the fundamental period of the structure increases, indicating that, according to Eq. 5, input energy is better correlated to the damage measure represented by maximum interstory drift than dissipated energy.

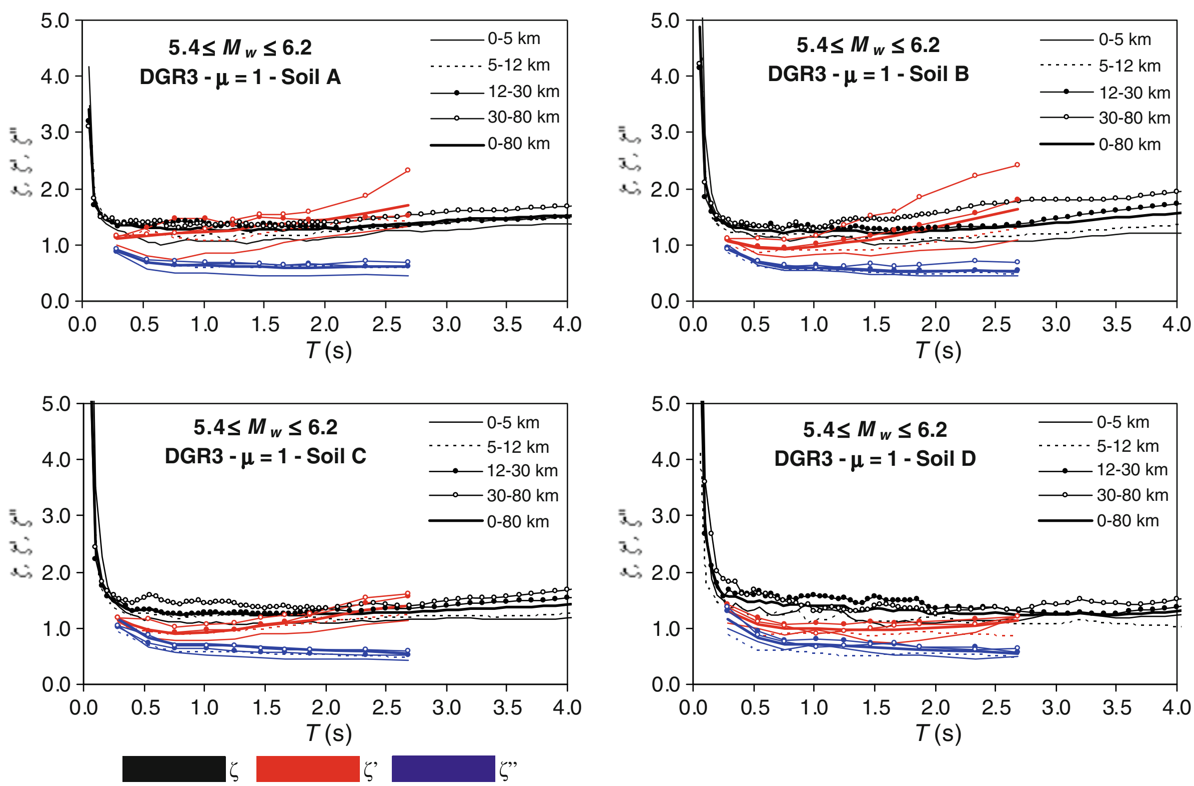

Figure 17
ζ, ζ' and ζ'': Soils A, B, C, D; DGR3, $\mu = 1$, $5.4 \leq M_w \leq 6.2$

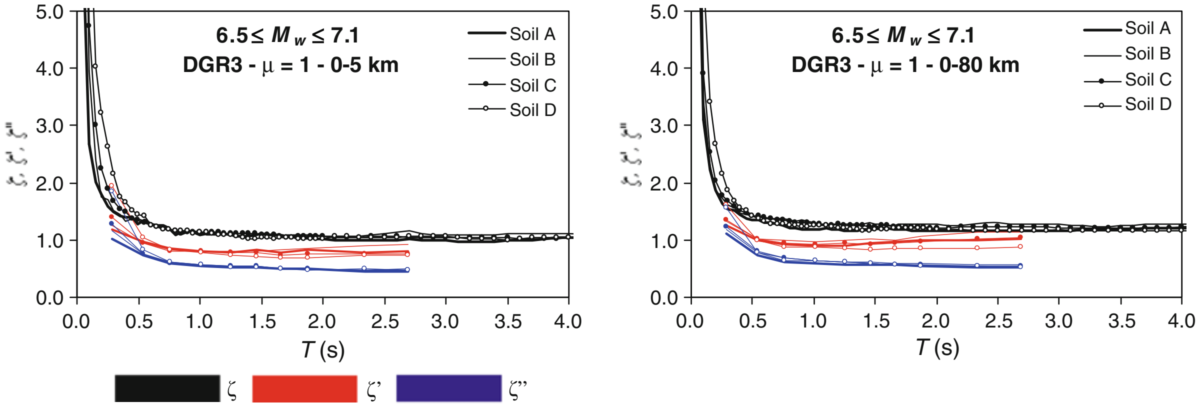

Figure 18
ζ, ζ' and ζ'': near-fault and ordinary records; DGR3, $\mu = 4$, $6.5 \leq M_w \leq 7.1$

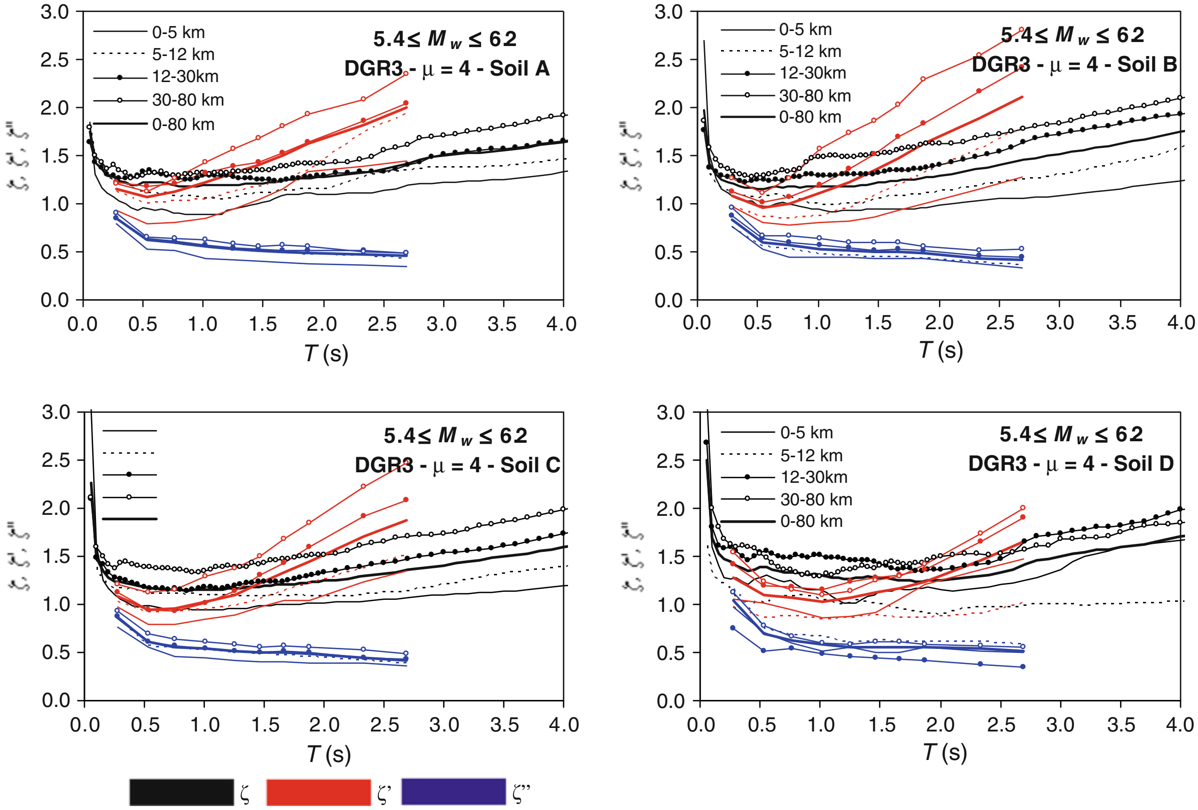

Figure 19
ζ, ζ' and ζ'': Soils A, B, C, D; DGR3, $\mu = 4$, $5.4 \leq M_w \leq 6.2$

7. Conclusions

In this paper reliable relationships between energy and displacement demands for SDOF and MDOF systems were analyzed. A simplified equivalent shear-type model (ESTM), able to provide a reasonable estimation of the seismic response of MDOF systems at global and local level was adopted. To this purpose several parametric analyses were performed on a wide range of frame structures, subjected to

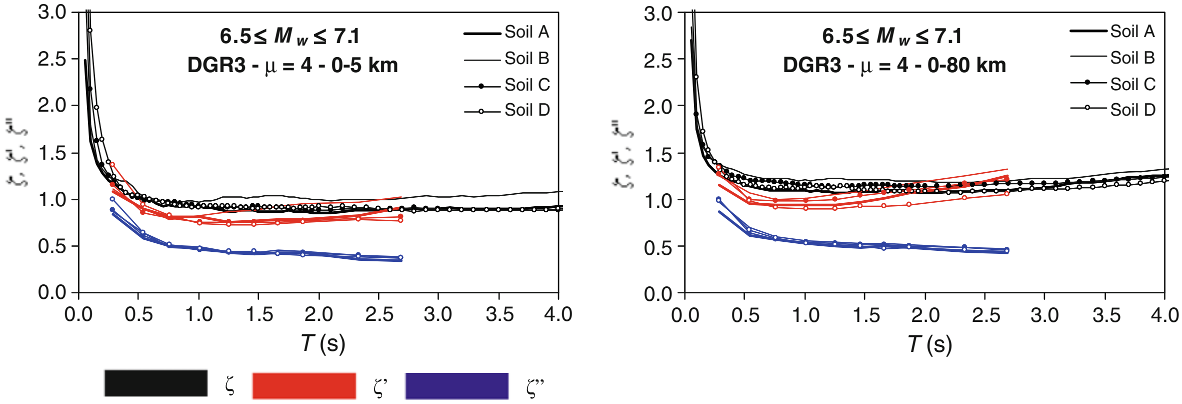

Figure 20
ζ, ζ' and ζ'': near-fault and ordinary records; DGR3, $\mu = 4$, $6.5 \leq M_w \leq 7.1$

recorded ground motions of different characteristics in order to provide a spectral representation of the seismic demands.

Preliminary correlation analysis were carried out to assess the higher degree of correlation of energy-based parameters to appropriate damage measures, in comparison with conventional spectral parameters. Subsequently, the extension of the characterization of the relations between energy and displacement demands from SDOF systems to MDOF systems was achieved by introducing two pairs of indices that relate input energy and dissipated hysteretic energy to both roof displacement and inter-story drift. Such expressions could be interpreted as the ratios between equivalent velocities.

On the basis of the above mentioned simplified methodology, it was possible to assess the effects of the stiffness distribution patterns along the height of the structures and the hysteretic models characterizing the cyclic behavior of the simplified MDOF models on the examined parameters. As far as the hysteretic behaviour is concerned, it was found that its influence is generally not important for the degrading models, in the whole period range and for various ductility ratios; on the contrary, the EPP model exhibits a trend significantly divergent from the former ones. The influence of the stiffness distributions through the height of the multi-story frames is not particularly significant. However, it was possible to detect a certain deviation of the uniform stiffness pattern in comparison to theparabolic stiffness distribution schemes.

The fact that the results obtained do not appreciably depend either on the hysteretic model for degrading systems, or on the usually employed stiffness distribution schemes, indicates the possibility to reduce the complexity of the problem when extending the methodology to a broader set of structural systems and strong motions records.

Finally, nonlinear analyses performed subjecting SDOF and MDOF systems to a considerable number of appropriately selected recorded ground motions, allowed us to point out the degree of correlation of energy parameters based on the dissipated energy and on the input energy to the chosen damage measures. As far as energy parameters incorporating SDOF systems maximum displacement and MDOF systems top-story max displacement are concerned, results show that modelling low-rise buildings resorting to SDOF systems gives satisfactory estimation of the maximum displacement demand. For medium to high-rise structures. Conversely, results obtained for SDOF and MDOF systems diverge significantly, also appearing markedly affected by factors such as soil stiffness and distance to the causative fault. As far as energy parameters involving the maximum interstory drift ratio as damage measure are concerned, they appear much less affected by soil-site conditions. Besides, the degree of correlation between energy and displacement quantities is noticeably more stable in the where case input energy rather than energy dissipated by hysteresis is considered, in comparison with the simplified, in particular, the top displacement can be approximated with the maximum oscillator displacement. Further developments may involve, provided that the correlations between these parameters and structural quantities are formally obtained via appropriate regression analyses, the establishment of methodologies allowing one to predict the seismic performance of structures through damage indexes (such as the maximum interstory drift and the top displacement) directly by the so-obtained correlation equations.

Acknowledgments

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