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Dimensional response analysis of bilinear systems subjected to non-pulse-like earthquake ground motions

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Abstract

The maximum inelastic response of bilinear single-degree-of-freedom systems when subjected to ground motions without distinguishable pulses is revisited with dimensional analysis by identifying time scales and length scales in the time histories of recorded ground motions. The characteristic length scale is used to normalize the peak inelastic displacement of the bilinear system.

The paper adopts the mean period of the Fourier transform of the ground motion as an appropriate time scale and examines two different length scales which result from the peak ground acceleration and the peak ground velocity. When the normalized peak inelastic displacement is presented as a function of the normalized strength and normalized yield displacement, the response becomes self similar and a clear pattern emerges.

Accordingly, the paper proposes two alternative predictive master curves for the response which involve solely the strength and yield displacement of the bilinear SDOF system in association with either the peak ground acceleration or the peak ground velocity, together with the mean period of the Fourier transform of the ground motion. The regression coefficients that control the shape of the predictive master curves are based on 484 ground motions recorded at rock and stiff soil sites and are applicable to bilinear SDOF systems with post-yield stiffness ratio equal to 2% and inherent viscous damping ratio equal to 5%.

KEY-WORDS: Dimensional analysis, Self similarity, Inelastic displacement, Peak ground acceleration, Peak ground velocity, Mean period

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INTRODUCTION

Currently, the two most widely used approaches in estimating the peak inelastic response of inelastic single-degree-of-freedom (SDOF) systems, which are known as the "displacement modification" method and the "equivalent linearization" method (FEMA 2004).

The displacement modification method is based on the statistical analysis of the results of time histories of inelastic and corresponding elastic SDOF systems and derives from the early ideas of Veletsos and Newmark (1960) and Veletsos et al. (1965). The method aims on the generation of a multiplication coefficient, the so-called inelastic displacement ratio, which is used to modify the maximum response of the elastic SDOF system (Ruiz-Garcia and Miranda 2003; Chopra and Chintanapakdee 2004; and references reported therein) in an effort to estimate the response of the inelastic system with some pre-yielding period.

In the equivalent linearization method (Iwan 1980; ATC 1996), the maximum displacement of the inelastic SDOF system is approximated by the maximum displacement of an elastic SDOF system with equivalent stiffness and equivalent damping aiming to represent the period shift and the hysteretic energy loss of the inelastic SDOF system, respectively. The ATC-55 (FEMA 2004) project presented improved equivalent linearization procedures by introducing new equations to derive the equivalent stiffness and damping as functions of the ductility demands.

Despite their fundamental differences, the displacement modification and the equivalent linearization both use a substitute elastic structural system for approximating the response of the real inelastic system. On the other hand, a handful of recent studies showed alternative and promising approaches for predicting the maximum inelastic response without the need to use the substitute elastic system.

The unique advantages of normalizing the response with a time scale and a length scale of the excitation was first proposed by Makris & co-workers (Makris and Black 2004a; 2004b;

Makris and Psychogios 2006, Karavasilis et al. 2010) who showed using dimensional analysis (Langhaar 1951; Barenblatt 1996) that the normalized peak inelastic displacement response curves assume similar shapes for different values of the normalized yield displacement and concluded using the concept of self similarity that a single normalized peak inelastic displacement response curve can offer the peak inelastic displacement of the structure given the pulse period and amplitude of pulse-like earthquake ground motions. Mylonakis & Voyagaki (2006) developed closed form solutions for elastic-perfectly plastic SDOF systems subjected to simple waveforms and confirmed that the use of the strength reduction factor, R, complicates the results since parameter R is inherently rooted in the elastic response. The aforementioned ideas for estimating the peak inelastic response hinge upon the existence of predominant pulses in near-fault ground motions with distinct time scales; yet their extension to ground motions without predominant pulses is not apparent (Dimitrakopoulos et al. 2008).

In this paper, the maximum response of bilinear SDOF systems under ground motions without distinguishable pulses is revisited with dimensional response analysis by identifying a time scale and a length scale in the time histories of non-coherent earthquake records. Such time and length scales are used to normalize the strength, yield displacement and the peak inelastic displacement of the bilinear system.

The paper adopts the mean period of the discrete Fourier transform of the ground motion (Rathje et al. 2004; Dimitrakopoulos et al. 2008) as an appropriate time scale and examines two different length scales which result from the peak ground acceleration and the peak ground velocity. When the normalized peak inelastic displacement is presented as a function of the normalized strength and normalized yield displacement, the response becomes self similar and remarkable order emerges.

Accordingly, the paper proposes two alternative predictive response curves which involve solely the strength and yield displacement of the bilinear SDOF system in association with either the peak ground acceleration or the peak ground velocity, together with the mean period of the Fourier transform of the ground motion. The regression coefficients of the predictive master curves are based on 484 horizontal ground motions recorded at rock and stiff soil sites and are applicable to bilinear SDOF systems with post-yield stiffness ratio equal to 2% and inherent viscous damping ratio equal to 5%.

TIME AND LENGTH SCALE OF GROUND MOTIONS WITHOUT DISTINGUISHABLE PULSES

Based on formal dimensional analysis, Makris & co-workers (Makris and Black 2004a; 2004b; Makris and Psychogios 2006) derived a self-similar (master) curve that offers the peak inelastic SDOF displacement normalized to the energetic length scale of the predominant pulse of the earthquake ground motion (a measure of the persistence of pulse-like excitations to produce inelastic response). Such a length scale was defined as a_p/ω_p^2 with a_p the peak pulse acceleration, T_p the pulse period and ω_p ($=2\pi/T_p$) the pulse cyclic frequency. The present study extends the idea of defining a time scale and a length scale for earthquake ground motions without distinguishable pulses. Such time and length scales should result from the seismic signal rather from quantities masked by the response of substitute elastic SDOF systems, e.g., spectral ordinates or spectral corner periods.

Among numerous scalar ground motion intensity parameters, Riddel (2007) showed that the peak ground acceleration, *PGA*, peak ground velocity, *PGV*, and peak ground displacement, *PGD*, of both pulse-like and non-pulse-like ground motions present excellent correlation with peak inelastic displacements of SDOF systems with short, intermediate and long periods of vibration, respectively. Since most of real structures have period of vibration in the intermediate or short period range (i.e., period of vibration lower than 2.0 s.), the peak ground displacement has marginal importance. Recent works (Akkar and Ozen 2005; Akkar and Kucukdogan 2008) showed strong correlations of the *PGV* of ground motions without pulse signals with the maximum inelastic displacement demands of structures with intermediate period of vibrations, while Makris and Black (2004) showed that the self similar peak inelastic SDOF displacement curves scale better with the peak pulse acceleration rather than with the peak pulse velocity, indicating that *PGA* is a superior intensity measure of the pulse-like earthquake induced shaking.

Regarding the selection of a representative time scale, Rathje et al. (2004) examined various frequency parameters of a large strong motion data set containing both pulse-like and non-pulse-like ground motions and concluded that a reasonable measure of the frequency content of earthquake ground motions is the mean period T_m which is defined as

$$T_m = \frac{\sum_i C_i^2 \left(\frac{1}{f_i} \right)}{\sum_i C_i^2} \qquad (1)$$

where C_i are the Fourier amplitude coefficients of the entire accelerogram and f_i are the discrete fast Fourier transform frequencies between 0.25 and 20 Hz. It has been found that a stable value of T_m can be obtained for a frequency interval, Δf, lower than 0.05 Hz in the fast Fourier transform calculation (Rathje et al. 2004). The frequency interval of the fast Fourier transform is related to the time step, Δt, and the number of points, N, in a time series by $\Delta f=1/(N\cdot\Delta t)$. In order to obtain stable values of T_m, ground motions that do not satisfy the limit $\Delta f \leq 0.05$ Hz shall be padded with zeros. Recently, the mean period, T_m, given by Eq. (1) has been also adopted by Dimitrakopoulos et al. (2008) for the dimensional analysis of elastoplastic and pounding oscillators subjected to Greek earthquake records.

This paper also adopts the mean period, T_m, of the discrete Fourier transform of the ground motion as a representative time scale and examines two different length scales. The first length scale results from the peak ground acceleration, i.e., PGA/ω_g^2, while the second length scale results from the peak ground velocity, i.e., PGV/ω_g, where ω_g ($=2\pi/T_m$) is defined as the cyclic "mean" frequency of the non-pulse-like ground motion; consistent with the definition of the pulse cyclic frequency ω_p ($=2\pi/T_p$) of Makris and co-workers (Makris and Black 2004a; 2004b; Makris and Psychogios 2006).

It should be pointed out that the expected *PGA*, *PGV* and T_m of a site can be formally extracted with validated predictive equations published in literature (Rathje et al. 2004; Boore and Atkinson 2008) and therefore, with the proposed method the influence of important seismological parameters such as the moment magnitude, M_w, rupture distance, R_{rup}, and soil condition, are directly involved in the estimation of the peak inelastic response.

DIMENSIONAL ANALYSIS

For a given post-yield stiffness and inherent viscous damping ratios, the maximum displacement, u_{max}, experienced by a bilinear SDOF structural system under earthquake loading is assumed to be a function of the specific yield strength F_y/m (F_y the yield strength and m the mass), the yield displacement, u_y, the time scale, $\omega_g=2\pi/T_m$ and a length scale which is either the PGA/ω_g^2 or the PGV/ω_g. Accordingly,

$$u_{max} = f(F_y, m, u_y, PGA, \omega_g) \tag{2}$$

or

$$u_{max} = f(F_y, m, u_y, PGV, \omega_g) \tag{3}$$

According to Buckingham's Π-theorem (Lanhaar 1951; Barenblatt 1996), if an equation involving k variables is dimensionally homogeneous, it can be reduced to a relationship among k-r independent dimensionless Π-products where r is the minimum number of reference dimensions required to describe the physical variables. The five variables appearing in Eqs. (2) and (3) involve only two reference dimensions, that of length and time, and therefore, the number of independent dimensionless Π-products is: (5 variables)-(2 reference dimensions) = 3 Π-terms. The length and time scales, *PGA* and ω_g or *PGV* and ω_g, are selected to be the dimensionally independent repeating variables for normalizing u_{max}, F_y/m and u_y and therefore, Eqs. (2) and (3) reduce respectively to

$$\frac{u_{max}\omega_g^2}{PGA} = \varphi\left(\frac{F_y}{mPGA}, \frac{u_y\omega_g^2}{PGA}\right) \tag{4}$$

and

$$\frac{u_{max}\omega_g}{PGV} = \varphi\left(\frac{F_y}{mPGV\omega_g}, \frac{u_y\omega_g}{PGV}\right) \tag{5}$$

According to Eq. (4), the peak dimensionless inelastic displacement, $\Pi_{1,PGA} = u_{max}\omega_g^2/PGA$, should be predicted as a function of the dimensionless specific strength, $\Pi_{2,PGA} = F_y/mPGA$, and the dimensionless yield displacement $\Pi_{3,PGA} = u_y\omega_g^2/PGA$. According to Eq. (5), the peak dimensionless inelastic displacement, $\Pi_{1,PGV} = u_{max}\omega_g/PGV$, should be predicted as a function of the dimensionless specific strength, $\Pi_{2,PGV} = F_y/mPGV\omega_g$, and the dimensionless yield displacement $\Pi_{3,PGV} = u_y\omega_g/PGV$.

Fig. 1 plots the ground acceleration time histories, ground velocity time histories and Fourier amplitude spectra for the SHI000 (soil type B (FEMA 1997), rupture distance 19.2 km) ground motion recorded during the 1995 Kobe earthquake (M_w=6.9) and for the SVL270 (soil type B (FEMA 1997), rupture distance 24.2 km) ground motion recorded during the 1989 Loma Prieta earthquake (M_w=6.93). This figure shows that the peak ground accelerations and velocities of the two recordings are similar but SVL270 contains a significant high period signal that results in a mean period T_m=1.54 sec, while the SHI000 recording contains lower periods with a mean period T_m=0.76 sec.

Fig. 2 plots the dimensionless peak inelastic displacements of bilinear SDOF systems under the SHI000 and SVL270 ground motions, as a function of the dimensionless strength and for three values (0.1, 0.5 and 1.0) of the dimensionless yield displacement. The SDOF systems have inherent viscous damping ratio equal to 5% and post-yield stiffness ratio equal to 2%. The state determination of the bilinear force-deformation hysteresis and the integration of the nonlinear equation of motion were performed in MATLAB (1997). All the graphs show that in most cases as the dimensionless strength increases the peak displacement decreases for a constant yield displacement. What is most interesting in Fig. 2 is that the inelastic response curves computed for smaller values of Π_3 results in smaller values of Π_1; yet they assume a similar form regardless of the significant difference in the frequency content of the two ground motions used and regardless the values of *PGA* or *PGV* used for normalizing the response quantities. The paper proceeds by presenting statistics and associated predictive master curves for the peak inelastic response of SDOF bilinear systems under a large ensemble of ground motions without distinguishable pulses.

GROUND MOTION DATABASE

This study uses 484 horizontal strong ground motion recordings from the PEER (PEER) database with M_w and R_{rup} ranging from 5.0 to 8.0 and from 0 to 120 km, respectively. All the records were recorded at a site with a known soil condition, i.e., class B (rock sites), C (soft rock sites), and D (stiff soil sites) in accordance with the NEHRP 1997 (FEMA 1997) classification system. The records do not contain distinguishable pulses in their velocity and displacement time histories. This was confirmed by visual inspection and by making sure that none of the records used herein was reported as pulse-like in the results of Mavroeidis and Papageorgiou (2003) and Baker (2007). The peak ground acceleration of the records is greater than or equal to 0.045g.

The set of ground motion records used in this study is summarized in Table 1, listing the earthquake events and the number of records associated with each event. It is shown that the largest number of records is contributed by the 1999 ChiChi earthquake event, followed by the 1994 Northridge earthquake event. Nevertheless, no single earthquake event contributes more than one third of the record set.

Fig. 3 (top) shows the M_w and R_{rup} distribution of the ground motion record set. Different symbols were used to distinguish the record site soil conditions in that figure. It is noted that the records from larger M events are distributed in a wider range of R_{rup}. Fig. 3 (center) shows the statistical distribution of the mean period T_m in the record set. The record set is not dominated by a particular bin of T_m, while most of the records in the set have T_m ranging from 0.2 to 1.5 s. Fig. 3 (bottom) shows the statistical distributions of *PGV* and *PGA*. The number of records in each bin generally decreases with an increasing value of PGV or *PGA*.

DIMENSIONLESS PEAK INELASTIC RESPONSE

This section presents statistics on the peak inelastic response of bilinear SDOF systems in terms of the three dimensionless Π-products, $\Pi_{1,PGA} = u_{max}\omega_g^2/PGA$, $\Pi_{2,PGA} = F_y/mPGA$ and $\Pi_{3,PGA} = u_y\omega_g^2/PGA$ appearing in Eq. (4) and in terms of the three dimensionless Π-products, $\Pi_{1,PGV} = u_{max}\omega_g/PGV$, $\Pi_{2,PGV} = F_y/mPGV\omega_g$, and $\Pi_{3,PGV} = u_y\omega_g/PGV$ appearing in Eq. (5). The inherent viscous damping ratio and the post-yield stiffness ratio of the SDOF systems are equal to 5% and 2%, respectively.

For each of the 484 ground motions described in the previous Section of the paper, the peak inelastic response of bilinear SDOF systems with properties F_y/m and u_y which were tuned to correspond to 18 specific Π_2 values (0.2 to 3.0 with a step equal to 0.2, 3.0 to 4.0 with an increment of 0.5) and three specific Π_3 values (0.1, 0.5 and 1.0) was calculated. The corresponding Π_1 values were then obtained. It should be emphasized that different SDOF systems were excited by each of the ground motions since the ground motions have different values of *PGA*, *PGV* and T_m.

The lognormal distribution is known to be the most appropriate statistical representation for earthquake response and therefore, for a specific pair of Π_2 and Π_3, the central and dispersion values of Π_1 were obtained as the geometric mean, $\hat{\Pi}_1$, and the standard deviation, $\delta(\Pi_1)$, of the natural log of the N_g=484 (number of ground motions) sample values, respectively, i.e.,

$$\hat{\Pi}_1(\Pi_2,\Pi_3) = \exp\left(\frac{\sum_{j=1}^{j=N_g} \ln \Pi_{1,j}(\Pi_2,\Pi_3)}{N_g}\right) \tag{6}$$

and

$$\delta(\Pi_2,\Pi_3) = \sqrt{\frac{\sum_{j=1}^{j=N_g}\left(\ln \Pi_{1,j}(\Pi_2,\Pi_3) - \ln \hat{\Pi}_1(\Pi_2,\Pi_3)\right)^2}{N_g - 1}} \tag{7}$$

Fig. 4 plots the dimensionless peak inelastic displacements, $\Pi_{1,PGA}= u_{max}\omega_g^2/PGA$, for all the 484 earthquake records and for the three different dimensionless yield displacements $\Pi_{3,PGA}$=1.0 (top), 0.5 (center) and 0.1 (bottom). Fig. 5 plots the same information as Fig. 4 but with respect to the dimensionless terms $\Pi_{1,PGV}$, $\Pi_{2,PGV}$ and $\Pi_{3,PGV}$. It is observed that all geometric median response curves (heavy lines) assume a similar form, with the dimensionless peak displacement to decrease with Π_2 and increase with Π_3. It is interesting that exactly the same trend was observed by Makris & co-workers (Makris and Black 2004a; 2004b; Makris and Psychogios 2006) for pulse-like earthquake ground motions.

Fig. 6 plots the dispersion δ of the dimensionless peak inelastic displacements, $\Pi_{1,PGA}$ and $\Pi_{1,PGV}$, for the three different values of the dimensionless yield displacement. Both $\Pi_{1,PGA}$ and $\Pi_{1,PGV}$ exhibit in general dispersion values close to 30% for values of the dimensionless strength larger than 1. $\Pi_{1,PGA}$ exhibits lower and slightly lower dispersion values than $\Pi_{1,PGV}$ for values of the dimensionless strength lower and larger than 1, respectively. The dispersion values presented in Fig. 6 are small in comparison with the dispersion values ($0.4<\delta<0.5$ for highly inelastic response, i.e., $R>4$) exhibited by the inelastic deformation ratio presented in FEMA440 (2004).

The results presented in this section shed light on the dimensionless peak inelastic response of bilinear SDOF systems under non-pulse-like ground motions and can be also used for deriving a predictive equation for Π_1 as a function of Π_2 and Π_3. One may ask whether the SDOF systems used to create the response curves of Figs. 4 and 5 have periods of vibration and strengths within realistic practical limits since their specific strengths, F_y/m, and yield displacements, u_y, were selected to provide specific values of Π_2 and Π_3 for each recording. Fig. 7 (top) plots the strength reduction, R, distributions of the SDOF systems used to create Figs. 4 and 5, while Fig. 7 (bottom) plots the T distribution of the same SDOF systems. Note that the SDOF systems used to create Figs. 4 and 5 have the same period of vibration $T=(2\pi/\omega_g)(\Pi_3/\Pi_2)^{0.5}$. It is observed that unrealistic T and R values exist in the response databank described in this section since most of the structural systems in practice have $R\leq8$ and $T\leq2.5$ s. In order to provide a realistic design-oriented context for the present research effort, the proposed predictive master curves to offer Π_1 as a function of Π_2 and Π_3 are derived from a contracted response databank in the next section.

PROPOSED MASTER CURVES

The peak inelastic displacements were computed for bilinear SDOF systems having inherent viscous damping ratio equal to 5%, post-yield stiffness ratio equal to 2%, and with the following strength reduction factors R=2, 4, 6 and 8. For each of the 484 recordings used in this study and each R value, the peak inelastic displacements were calculated for 34 specific periods of vibration (0.1 to 1.0 s. with a step of 0.05 s., 1 to 2.5 s. with a step of 0.1 s.). Given the peak inelastic displacement, the period of vibration and the strength reduction factor of the SDOF system, the specific yield force and the yield displacement are easily calculated for given mass and therefore, the corresponding dimensionless products Π_1, Π_2 and Π_3 are readily available.

The previous section showed that as Π_2 increases the Π_1 decreases, while the rate of decrease is smaller the larger is the Π_3. Accordingly, the form of the master curve initially proposed by Makris & co-workers (Makris and Black 2004a; 2004b; Makris and Psychogios 2006) is adopted

$$\Pi_1 = (p + q\Pi_3^r)\Pi_2^s \tag{8}$$

as a good candidate for approximating the response databank, with p, q, r and s constant parameters to be determined. The Levemberg-Marquardt algorithm (MATLAB 1997) was adopted for nonlinear regression analysis of the response databank (484 recordings * 36 periods * 4 strength reduction factors = 69696 points), leading to the explicit form of Eq. (4)

$$\Pi_{1,\mathrm{PGA}} = (0.04 + 2.040\Pi_{3,\mathrm{PGA}}^{0.54})\Pi_{2,\mathrm{PGA}}^{-0.39} \tag{9}$$

and to the explicit form of Eq. (5)

$$\Pi_{1,\mathrm{PGV}} = (0.004 + 1.22\Pi_{3,\mathrm{PGV}}^{0.39})\Pi_{2,\mathrm{PGV}}^{-0.34} \tag{10}$$

Fig. 8 plots the computed dimensionless peak inelastic SDOF displacements $\Pi_{1,\mathrm{PGA}}$ from 69696 nonlinear time history analyses together with the proposed master curve (heavy line-Eq. (9)), while Fig. 9 plots the computed dimensionless peak inelastic SDOF displacements $\Pi_{1,\mathrm{PGV}}$ together with the proposed master curve (heavy line-Eq. (10)).

The degree of accuracy of the proposed master curves (Eqs. (9) and (10)) was quantified with the overall cumulative normalized error (Makris and Psychogios 2006)

$$e = \frac{1}{N}\sum_{j=1}^{N}\left|\frac{\Pi_{1,exact}^{j} - \Pi_{1,app}^{j}}{\Pi_{1,exact}^{j}}\right| \tag{11}$$

where N is the number of responses (i.e., N = 69696 in this paper), $\Pi_{1,exact}$ is the dimensionless peak displacement computed with the nonlinear time history analysis and $\Pi_{1,app}$ is the dimensionless peak displacement which results from the master curve (either Eq. (9) or Eq. (10)). Eq. (9) gives e=0.45, while Eq. (10) gives e=0.48. If we perform the cumulative error calculation only for SDOF systems with T>0.5 sec, Eq. (9) gives e=0.38, while Eq. (10) gives e=0.30. Therefore, this paper highly recommends for design purposes the use of Eq. (10) for systems with T>0.5 sec.

To this end this paper compares the performance of Eq. (10) with the recommendations of FEMA440 (FEMA 2004) which adopts the relation of Ruiz-Garcia & Miranda (2003) for the inelastic deformation ratio, i.e.

$$C_R = 1 + \frac{R-1}{a \cdot T^2} \tag{12}$$

where a takes the values 130, 90 and 60 for NEHRP site classes B, C and D, respectively. Eq. (12) gives e = 0.39 with respect to the whole response databank, while for systems with T>0.5 sec, it gives e = 0.33. Therefore, the proposed dimensional Eq. (10) performs slightly better than the inelastic deformation ratio for systems with T>0.5 sec, while the inelastic deformation ratio performs better than both Eqs. (9) and (10) for systems with T<0.5 sec. Nevertheless, more work is needed in order to identify hidden and more effective time and length scales in the time histories of non-coherent recordings.

CONCLUSIONS

The maximum response of bilinear SDOF systems subjected to ground motions without distinguishable pulses was revisited with dimensional analysis by identifying a time scale and a length scale in non-coherent recordings. Such time and length scales were used to normalize the strength, the yield displacement and the peak inelastic displacement of structural systems with bilinear behavior.

The paper adopts the mean period of the discrete Fourier transform of the ground motion as a representative time scale and examines two different length scales which result from the peak ground acceleration and the peak ground velocity. When the normalized peak inelastic displacement is presented as a function of the normalized strength and normalized yield displacement, the response became self similar and remarkable order emerges.

Accordingly, the paper proposes two predictive master curves which involve solely the strength and yield displacement of the bilinear SDOF system in association with either the peak ground acceleration or the peak ground velocity, together with the mean period of the Fourier transform of the ground motion. The regression coefficients of the predictive master curves are based on 484 horizontal ground motions recorded at rock and stiff soil sites and are applicable to bilinear SDOF systems with post-yield stiffness ratio equal to 2% and inherent viscous damping ratio equal to 5%.

The proposed master curve which involves the peak ground velocity performs slightly better than the proposed inelastic deformation ratio of FEMA440 for systems with period of vibration longer than 0.5, while the inelastic deformation ratio performs better than both the proposed master curves for systems with period of vibration shorter than 0.5 sec.

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TABLES

Table 1. Ground motion record set used in this study (484 horizontal recordings)

Earthquake event	Date	M_w	No. of records
San Fernando	02/09/1971	6.61	2
Sitka	07/30/1972	7.68	2
Friuli-1	05/06/1976	6.50	4
Tabas	09/16/1978	7.35	4
Imperial Valley	10/15/1979	6.53	25
Mammoth Lakes	05/25/1980	6.06	2
Irpinia	11/23/1980	6.20	1
Irpinia-2	11/23/1980	6.90	5
Westmorland	04/26/1981	5.9	4
Coalinga	05/02/1983	6.36	2
Morgan Hill	04/24/1984	6.19	8
Hollister	01/26/1986	5.45	2
North Palm Springs	07/08/1986	6.06	4
Chalfant Valley	07/21/1986	6.19	4
San Salvador	10/10/1986	5.8	2
Baja	02/07/1987	5.5	2
Superstition Hill	11/24/1987	6.54	4
Loma Prieta	10/18/1989	6.93	55
Manjil	06/20/1990	7.37	2
Cape Mendocino	04/25/1992	7.01	5
Landers	06/28/1992	7.28	24
Bigbear	06/28/1992	6.46	2
Little Skull Mountain	06/29/1992	5.65	4
Northridge	01/17/1994	6.69	61
Kobe	01//16/1995	6.90	30
Hector Mine	10/16/1999	7.13	45
Duzce	11/12/1999	7.14	5
Kocaeli	08/17/1999	7.51	10
ChiChi	09/20/1999	7.62	158
Denali	11/03/2002	7.90	6