DYNAMIC RESPONSE OF ROCKING MASONRY ELEMENTS TO LONG PERIOD STRONG GROUND MOTION

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ABSTRACT :

Damage and collapse in masonry structures often take place locally. Due to the dynamic action, the damage mechanisms develop as loss of equilibrium of rigid blocks capable of sliding and rotating. Actually the rocking mechanisms of masonry elements, typically non-linear, show high displacement capacity until collapse, high fundamental period of vibration in the non-linear range that can further increase because of the nearly non-tensile strength of masonry. Recent works (e.g. Resemini *et al.* 2006) propose an approach based on the Equilibrium Limit Analyses (kinematic theorem), in order to describe these out-of-plane mechanisms. Nevertheless, the chaotic dynamic response of these rocking elements needs further studies. The rocking behaviour and its relation to different input and structural parameters are investigated by means of non-linear numerical simulations, with special attention to the long period range. Using a non-linear dynamic model, several step-by-step dynamic analyses on the equivalent SDOF systems are performed. In these analyses, the ground motion characterization is a crucial topic: from a selection of digital recorded accelerograms (Paolucci *et al.* 2008, Cauzzi and Faccioli, 2008), long period strong ground motions representative of different conditions (magnitude, focal distance, etc.) are used as input for the structural analyses, both dynamic and simplified (proposing a method based on overdamped response spectra). The aim is to verify the applicability of response spectra parameters to simplified procedures for the structural verification of damage mechanisms in masonry structures. Moreover, the influence on the response of ground motion characteristics (*PGA*, *PGV*, *PGD*, Housner intensity, etc.) is analysed.

KEYWORDS: rocking, masonry, non-linear analyses, long period ground motion

1. INTRODUCTION

The seismic response of rocking elements (such as water tanks, industrial equipment, etc.) has been object of wide studies, involving structural and seismological features (Housner 1963, Yim *et al.* 1980). These works introduced and developed the idea of studying a free-standing block resting on a rigid base and subjected to a horizontal ground motion as a single-degree-of-freedom (SDOF) oscillator, whose period depends on the amplitude of rocking. However, some works (Makris and Konstantinidis 2003) pointed out the physical diversity between the equivalent SDOF system and the slender block in rocking motion, i.e. the inverted pendulum, highlighting the critical issues in using approximate design methodology (FEMA 2000) to estimate block rotations by performing iterations on the true or design displacement response spectrum.

Nevertheless, as also observed by Sorrentino and Masiani (2007), wide information about seismic hazard are correlated to the response parameters of a damped linear-elastic SDOF system (e.g. response spectra). From this point of view, as simplified methods are needed in order to assess rocking structures, numerical investigations correlating those quantities to the rocking behaviour of a free-standing block may be useful.

In particular, the present paper deals with seismic safety of masonry elements, prone to overturning during an earthquake. Indeed, damage occurred during seismic events pointed out that, if the masonry shows good characteristics, the damage mechanisms develop as loss of equilibrium of rigid blocks capable of sliding and rotating. On the one hand, this leads to the adoption of the kinematic approach, based on the equilibrium limit analysis, as a feasible criteria to describe the behaviour of these local mechanisms under equivalent static loads (Restrepo-Vélez and Magenes 2004, Lagomarsino *et al.* 2004). On the other hand, actually the out-of-plane mechanisms, typically non-linear, show high displacement capacity until collapse. As a matter of fact, since an earthquake is a dynamic action, the static loss of equilibrium does not correspond to the collapse, and the kinematism is able to sustain some horizontal action even after its activation. Several researches were devoted to the dynamics of masonry rocking elements, representative of columns, simple architectural components or portion of walls: experimental testing of single blocks or block assemblages (Ceradini 1992, Lam *et al.* 1998), theoretical works and numerical simulations of the out-of-plane response (e.g., Psycharis *et al.* 2000, Doherty *et al.* 2002, Sorrentino *et al.* 2006).

2. OUTLINE AND SCOPE OF THE WORK

Various researches introduced the idea of verifying the out-of-plane safety of local mechanisms through displacement-based analysis. In recent works (Lagomarsino *et al.* 2004, Resemini *et al.* 2006), the attention was focused on the definition of an equivalent SDOF system representative of a kinematic chain of rigid blocks (Fig. 1-a, b), schematizing plausible collapse modes of monumental buildings or their macroelements (e.g., façades, bell-towers or triumphal arches). These structures have high fundamental period of vibration still in the elastic range. The period can further increase because of the nearly non-tensile strength of masonry causing widespread cracking (T>2.5-4s). The curve representing the structural capacity (Fig. 1-c) is derived in terms of horizontal acceleration a_0 that activates the mechanism (representing the capacity in terms of strength) and horizontal displacement d_0, representing the ultimate displacement (see Resemini *et al.*, cit., for details). Moreover, the application of a simplified displacement-based approach, where overdamped spectra are used for the seismic demand representation, is therein proposed.

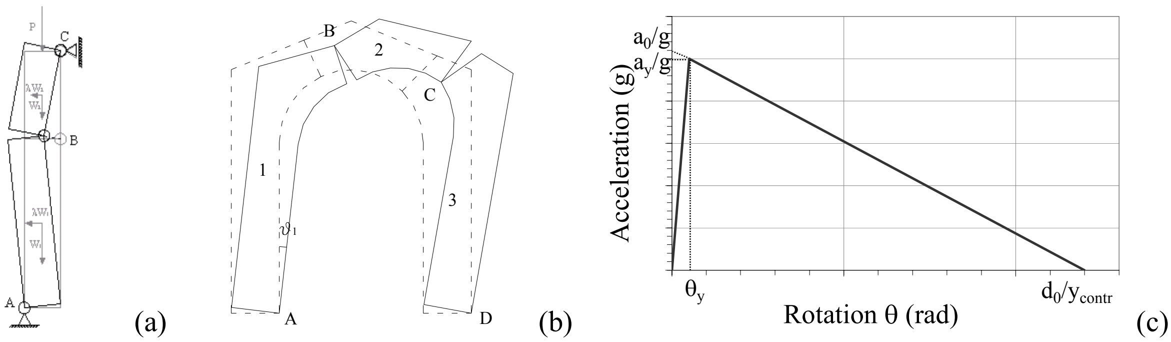

Figure 1 a) Two blocks kinematism for a wall (in presence of a constraint in the upper part); b) Three blocks kinematism for a triumphal arch; c) Bi-linear capacity curve with initial ascending branch, in terms of acceleration (unit of g) and rotation.

In this paper, in the framework of these earlier studies, assuming the previously introduced representation, both for the structural capacity and for the seismic demand, the reliability of the procedure to estimate mechanism displacements d (or rotations $\theta = d/y_{contr}$, where y_{contr} is the height of the control point) for a given seismic input is checked. The rocking behaviour and its relation to different input and structural parameters are investigated by means of non-linear numerical simulations, with special attention to the long period range. Using a non-linear dynamic model, several step-by-step dynamic analyses on the equivalent SDOF systems are performed. In these analyses, the ground motion characterization is a crucial topic: from a selection of digital recorded accelerograms (Paolucci *et al.* 2008, Cauzzi and Faccioli, 2008), long period strong ground motions representative of different conditions (magnitude, focal distance, etc.) are used as input for the structural analyses, both dynamic and simplified (overdamped response spectra).
Furthermore, the influence on the response of ground motion characteristics (*PGA*, *PGV*, *PGD*, Housner intensity, etc.) is analysed.

3. DISPLACEMENT FORECAST OF THE EQUIVALENT SDOF OSCILLATOR

The implementation of the proposed models requires the definition of a substitute-structure model for the macroelements, of a representative non-linear elastic force-displacement relationship, to be obtained performing a non-linear kinematic analysis, and lastly the proposal of a representative damping relationship to be employed both in the dynamic analyses and in the simplified procedure (overdamped response spectra).

3.1 Assessment of the dynamic response of the SDOF non-linear system

The adopted model for the description of the equivalent SDOF motion, is similar to Housner's one (Housner 1963), introducing the initial elastic branch representing the overall stiffness (Fig. 1-c). The oscillator is characterised by elastic behaviour, with softening range, where the energy dissipation in the rocking response (originates from the impacts) is translated into an equivalent viscous damping coefficient and accounted for in the reaction force assessment. The cyclic behaviour does not allow hysteretic dissipation, but, the value of the equivalent viscous damping is related to the rotation θ, through the secant period T corresponding to θ.
The equations of motion are modified by substituting the relation of the capacity curve obtained. For damped forced vibrations, the equation of motion for the equivalent oscillator is:

$$\ddot{\theta} + \gamma\,\dot{\theta} + p^2\, r(\theta) = -p^2\,\frac{a_g(t)}{g} \tag{3.1}$$

where $\theta=\theta(t)$ is the rotational degree of freedom and $\dot{\theta}$ and $\ddot{\theta}$ are, respectively, its first and second time derivative, γ is the damping coefficient defined as a function of the equivalent viscous damping ξ_{eq} (see § 3.2), $r(\theta)$ is the reaction force function (depending on both a_0, d_0 and T_0 initial period evaluated for the initial ascending branch), $a_g(t)$ is the ground acceleration (m/s^2), g is the gravity acceleration (m/s^2). The coefficient p^2 is evaluated for undamped free vibration, resulting in g/y_{contr}.
An ad hoc numerical program has been implemented in order to obtain the numerical solution of the dynamic motion for the non-linear oscillator. The explicit method proposed by Runge-Kutta has been assumed for the numerical integration of the equation of motion.

3.2 Displacement-based procedure to evaluate the SDOF system performance

The simplified displacement-based approach introduces overdamped elastic spectra for the evaluation of the maximum response in terms of displacement (or rotation θ). The use of overdamped spectra allows us to account for the influence of the equivalent viscous damping that strongly increases in the long period range when the initial phase is overcome, as clearly shown from experimental evidences about rocking elements (Doherty *et al.* 2002).
The displacement forecast procedure is the classical one, except that iterative process is not needed, because an ad hoc damping relation (period depending) is predefined. This damping relation (Resemini *et al.* 2006) is almost independent from the initial period T_0 (ξ_{eq} =0.05 up to 0.4 s), linearly increasing up to an adequate bound (ξ_{eq} =0.2): an important outcome of cited works is represented by the correspondence, in the medium-large period range, between the non-linear time-history analysis results and the simplified predictions through overdamped elastic spectra (employing artificial acceleration time histories).

4. A SELECTION OF REAL ACCELEROGRAMS AS INPUT

The recent advances in seismic design of structures, placing strong emphasis in displacement considerations (see Priestley *et al.*, 2007) and capacity design concepts, stimulated, in Italy, a national research project called S5, sponsored by the Department of Civil Defence, initiated in mid-2005 and concluded in July 2007. S5 was carried out jointly with other projects (see legacy.ingv.it/progettiSV/), for which general consistency of inputs and outputs was a key requirement. The project was coordinated by Prof. E. Faccioli (Politecnico di Milano, Italy) and Dr. A. Rovelli (Italian National Institute of Geophysics and Volcanology INGV-Roma1, Italy) and its main aims were (a) to define a model of the seismic action in terms of arbitrarily damped Displacement Response Spectra (*DRS*), including local soil effects, extending to long periods and suitable for introduction in seismic codes; (b) to provide national hazard maps displaying the *DRS* values needed for design, for selected return periods. As a prerequisite for the complete fulfillment of the Project objectives, an innovative procedure for assessing the reliability of long-period response spectral ordinates from digital accelerograms was introduced by Paolucci *et al.* (2008). Based on these new technique, a worldwide digital database of about 1160 records from 60 earthquakes was assembled (Cauzzi and Faccioli, 2008) and used to derive long-period ground motion prediction equations in terms of arbitrarily damped *DRS* over a broad period range (T < 20 s). Within the framework of the cited S5 Project, the prediction equations by Cauzzi and Faccioli (cit.) were easily implemented into the CRISIS2003 computer program (Ordaz *et al.*, 1991) for probabilistic seismic hazard analyses. The result of this process was a set of Uniform Hazard (UH) displacement spectra in the period range 0.1 s – 10 s for all the Italian *Comuni* (the smallest local administrations), along with a number of displacement hazard maps for Italy for selected values of the vibration period T (Faccioli and Villani, 2008). The maps and the UH spectra are currently available on-line at the following Internet site: http://progettos5.stru.polimi.it. The hazard maps at 475 yr return period show that the long period spectral displacement (T = 10 s) does not exceed 10-12 cm over a large portion of Northern and Central Italy; only in the most seismically active regions of the Country (i.e. the Southern Calabrian Arc), with maximum expect magnitude values higher than 7, a displacement demand of about 20 cm is reached (Faccioli and Villani, cit.).

Table 1 List of the 17 real accelerograms selected for analyses in the present study

Ref.	Event Date and Time (UTC)	Station	EC8 ground type	*R* (km)	M_W	*D* (s)	*PGA* (m/s^2)	*PGV* (m/s)	D_{10} (cm)	*SI* (cm)
L	Near Miyiakejima Island (Japan) 2000_July_02 20:03	TKY011 (NS)	B	26.9	5.6	9.9	0.53	0.02	1.18	9.0
L	Near Niijima Island (Japan) 2000_Aug_18 01:52	TKY011 (NS)	B	29.2	5.7	16.6	0.18	0.02	1.76	7.5
L	Rumoi Region (Japan) 2004_Dec_14 05:56	HKD117 (EW)	B	48.8	5.7	35.4	0.17	0.01	1.43	7.5
L L L	Yamaguchi Prefecture (Japan) 1997_June_25 09:50	SMN012 (NS) YMG010 (NS) YMG015 (NS)	B B B	29.7 39.9 56.3	5.8 5.8 5.8	9.7 9.0 17.4	0.83 1.68 0.29	0.03 0.04 0.02	1.57 1.45 1.55	9.1 8.3 7.7
L	East Off Izu Peninsula (Japan) 1998_May_03 02:09	TKY008 (NS)	B	27.2	5.5	11.3	0.65	0.02	1.19	10.6
M	Mid_Niigata_Prefecture (Japan) 2004_Oct_23 08:56	NIG015 (NS)	B	54.6	6.6	38.3	0.67	0.05	6.91	27.2
M	NW_Off_Kyushu (Japan) 2005_March_20 01:53	FKO008 (NS)	B	66.6	6.6	15.2	1.20	0.10	8.16	47.7
M	South Iceland 2000_June_17 15:40	105(EW)	B	21.2	6.5	7.2	2.07	0.12	4.61	40.8
M	South Iceland (aftershock) 2000_June_21 00:51	105(EW)	B	25.8	6.4	7.9	1.66	0.10	9.05	48.3
H	South Iceland 2000_June_17 15:40	108(EW)	A	19.8	6.5	4.3	1.24	0.20	27.16	60.3
H H H	South Iceland (aftershock) 2000_June_21 00:51	108(EW) 305 (EW) 306 (EW)	A A A	31.8 25.0 25.0	6.4 6.4 6.4	8.6 9.5 8.4	0.26 0.54 1.07	0.12 0.12 0.12	17.76 15.80 15.87	27.2 42.4 45.4
H	Irpinia (Italy) 1980_Nov_23 18:34	Bisaccia (EW)	A	32.3	6.9	48.2	0.71	0.19	22.78	80.6
H	Southern Iwate Prefecture (Japan) 2008_June_13 23:43	MYGH02(EW)	B	28.1	6.9	8.9	2.30	0.15	14.75	77.4

With this background, and following Faccioli and Villani (cit.), the attention is focused in this work on the seismic demand in terms of long period *DRS* for three sites in Italy with different hazard exposure: the city of Reggio Calabria (a high seismicity site in Southern Italy), the town of Gubbio (medium, Cental Italy) and the city of Turin (low, Northwestern Italy). The long-period (T = 10 s) spectral displacement is equal to 21.3 cm, 6.8 cm and 1.5 cm for Reggio Calabria, Gubbio and Turin, respectively. Rather than generating a set of spectrum-compatible synthetic accelerograms, a different approach is used in the present study: we set as targets the UH spectra for 475 yr return period at the cited sites and search the database by Cauzzi and Faccioli (cit.), extended with other recent earthquakes from Japan (namely the Noto 2007 M_W 6.7 earthquake, the Niigata 2007 M_W 6.7 event and the Iwate 2008 M_W 6.9 one), in order to find spectra from real accelerograms that closely match the target one. Such kind of research is not straightforward, mainly because of the smoothed shape of the UH spectra, if compared with those obtained by real accelerograms. Nevertheless, 17 real accelerograms could be selected, as listed in Table 1. The Japanese K-net (www.k-net.bosai.go.jp) network is the largest contributing source to the selected dataset, while the rest of accelerograms are from the IC European Strong Motion Database (www.isesd.cv.ic.ac.uk/ESD/, Ambraseys *et al.*, 2002) and (one, MYGH02) from the KiK-net Japanese network (www.kik.bosai.go.jp). The reference seismicity level for each accelerogram is given in the first column of Table 1, i.e. L = low, M = medium, H = high. In Table 1, the focal distance R, the earthquake magnitude M_W, the duration D (based on Arias intensity), the peak ground acceleration PGA, the peak ground velocity PGV and the spectral intensity SI (Housner) are given for each record. As a representative measure of the long-period characteristics of the selected accelerograms, the 5%-damped spectral displacement at T = 10 s (hereafter D_{10}) is chosen, rather than the peak ground displacement PGD (see also Cauzzi and Faccioli, cit.). Following Paolucci *et al.* (cit.) and Cauzzi and Faccioli (cit.), the present selection contains only spectra having a probability $P \geq 0.9$ of the long period DRS drifts to be less than 15%. All the records were written by digital instruments but one, namely the Bisaccia record of the 1981 Irpinia (Italy) earthquake, which was inserted in the databank only after a careful scrutiny of its long-period characteristics. All the acceleration records were obtained on ground type B or A, according to the four main ground categories contemplated in the Eurocode 8. Note from Table 1 that high values of long-period spectral displacements do not generally correspond to high values of PGA. See also Faccioli and Villani (cit.) for further discussion about this issue.

5. VALIDATION OF THE SIMPLIFIED PROCEDURE VIA DYNAMIC ANALYSES

For each one of the 300 equivalent SDOF systems considered (having 0.2 s $\leq T_0 \leq$ 0.4 s and 0.1 m/s$^2 \leq a_0 \leq$ 1.0 m/s^2; 0.02 m $\leq d_0 \leq$ 0.2 m), the dynamic analyses are implemented using (without scaling) the 17 accelerograms; the displacement demand (performance-point assessment) resulting applying the proposed simplified procedure has been compared with these results.
For the validation of the forecast, reference is made to a special kind of result representation, inserting an ordinate secondary axis (Fig 2-a). The following quantities are represented:

- the displacement spectrum S_d vs. the period T; S_d is the response spectrum derived by the maximum response of linear SDOF systems, for which the damping ratio is $\xi_{eq}(T)$. The dynamic response of the SDOF systems is evaluated in the time domain through a convolution integral;
- the acceleration spectrum S_a vs. the period T; S_a is computed referring to S_d;
- the maximum displacement $d_{max}=d(T_{max})$ resulting from the dynamic analysis vs. the period T;
- the acceleration $a_{max}=a(T_{max})$ corresponding to the maximum displacement d_{max} vs. the period T.

The simplified procedure matches with the dynamic result when, for the calculated T_{max} value, d_{max} lays on the overdamped displacement spectrum S_d and, contemporaneously, a_{max} lays on the overdamped acceleration spectrum S_a (as in case of SDOF1).
The results for the three groups of records (high, medium and low seismicity) are shown in Fig. 2-b, c and d, in which the average overdamped response spectra are also shown. Non negligible scatter can be noticed between the displacement estimation for the 300 SDOF systems in case of low seismicity area (Fig. 2-b), and the d_{max} values seem to exceed the overdamped spectra. In this case, the input characteristics (in comparison to the typical value of capacity of the analysed masonry structures) are not severe enough to push the structure in the long period range of oscillation; the models, in the short period range (around and little beyond the initial period T_0), are extremely sensitive to the acceleration parameter and small variations in its values produce large differences in the response. As a consequence, being the spectral acceleration values (T < 2 s) rather disperse, the scatter in the obtained displacements is high and the forecast seems to be not good.
If the attention is focused on medium-large period range (1.0-6.0 s) and particularly on large period range (2.5-6.0 s), the displacement prediction in case of medium seismicity records (Fig. 2-c) is reasonably correct in average. On the other hand, in case of high seismicity, even if the results obtained by the simplified procedure are on the

safe side, some discrepancy may be highlighted. Grouping the records on the basis of other parameters (e.g. *PGA*, *PGV*, *SI*) did not provide better estimates with the present dataset.
It is worth noting that the data in Fig. 2 do not allow us to quantify the error between the dynamic and the simplified method, standing that (without knowing the dynamic result) the simplified procedure may find the intersection of the capacity curve and the overdamped spectrum for a value of the secant period (namely T_{PP}) different from T_{max}.

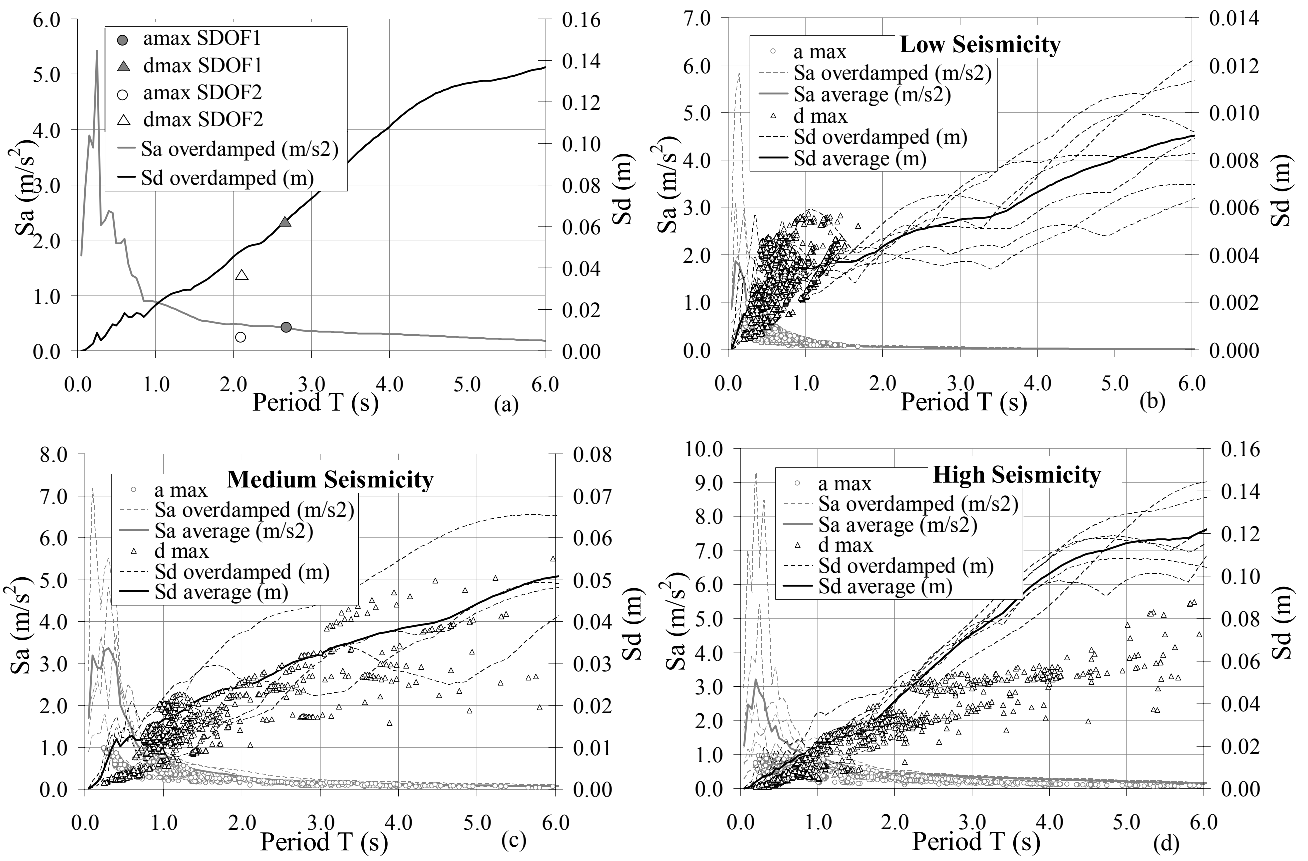

Figure 2 Comparison between dynamic analysis and simplified procedure: (a) example of a single earthquake; (b) low seismicity records; (c) medium seismicity records; (d) high seismicity records.

6. FURTHER OUTCOMES AND FINAL REMARKS

One of the aims of this work has been to clarify some issues about the dynamic response of a "long period" rocking system representative of masonry elements, trying to point out the ground motion parameters governing its behaviour. Based on the selected 17 accelerograms, it resulted (Fig. 3-a) that *PGA* (representative of the high frequency content of ground motion) and duration *D* are not clearly related to the maximum response displacement. In case of *D*, it is worth noting that the dynamic model does not account for damage accumulation, so a poor correlation may be reasonably supposed. The increase of the maximum displacement value d_{max} is better related to input parameters such as *PGV* and spectral displacement D_{10} (Fig. 3-b, c). In case of Housner intensity *SI* (Fig. 3-d), contrary to other studies (Sorrentino *et al.* 2007), the trend is less clear. Nevertheless, focusing the attention on a few sets of oscillators characterised by the same ultimate displacement capacity d_0 (0.06 m/0.1 m/0.16 m), interesting results can be shown in terms of d_{max} distribution, assumed as lognormal, in case of period T_{max} < 7 s (16^{th} / 50^{th} / 84^{th} percentiles in Fig. 4). In this case, d_{max} may be directly related to the capacity d_0 and Housner intensity *SI* seems to be the best quantity to describe the behaviour. This parameter is computed integrating the 2%-damped pseudo-velocity spectrum between 0.1-2.5 s and it is representative of the damaging potential of the whole ground motion, rather than its peak values; for the "long period" structures in study, it would be interesting to analyse a similar quantity (*SPv0-4*) obtained integrating between 0-4 s.
Some troubles related to the non absolute smoothness of the spectra of the recorded time histories could be investigated

in future works, enlarging the accelerometric database in order to make the results statistically meaningful.

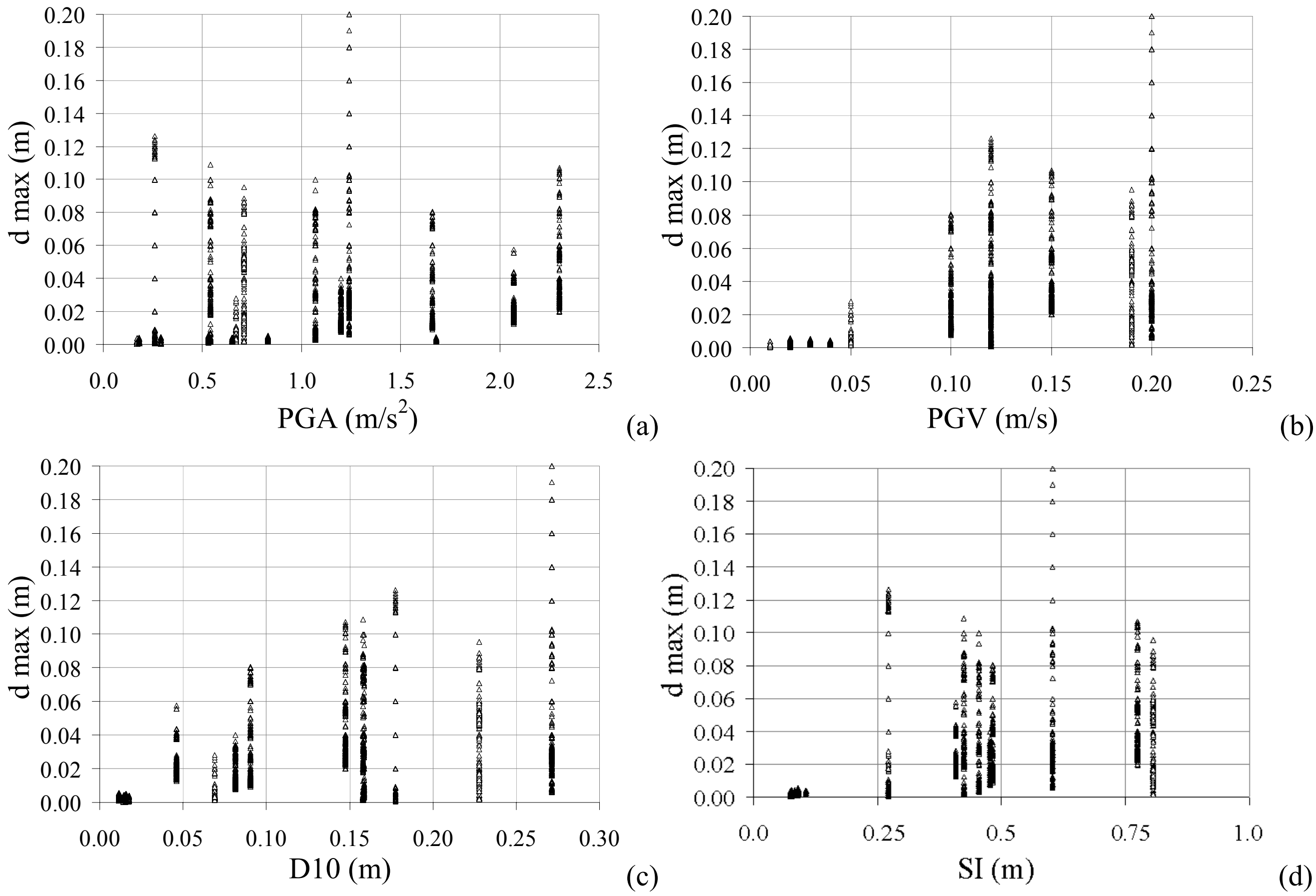

Figure 3 Dynamic maximum displacement value d_{max} vs.: (a) *PGA*; (b) *PGV*; (c) D_{10}; (d) *SI*.

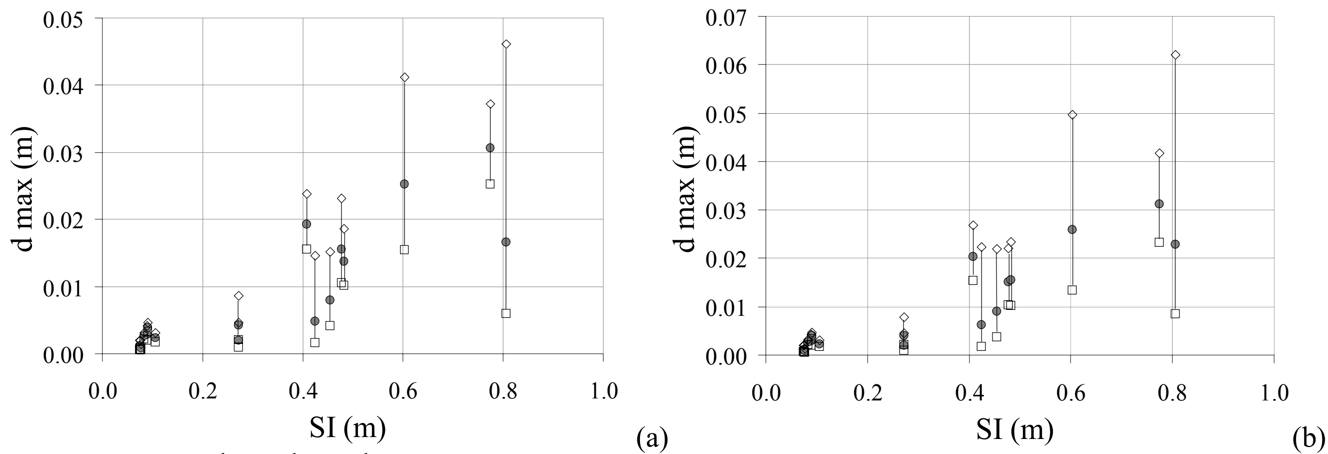

Figure 4 16th / 50th / 84th percentiles of d_{max} value vs. *SI* (T_{max} < 7 s): (a) d_0 =0.06 m; (b) d_0 =0.16 m.

Another objective of the proposed study has been the validation (in the long period range) of the displacement-based approach through overdamped elastic spectra, by means of comparisons with dynamic analyses. In fact, once obtained that the dynamic response of a "long period" equivalent oscillator is quite well correlated to the displacement demand, the need of a simplified but reliable procedure to assess the structural safety arises. An important outcome is represented by the correspondence between the non-linear time-history analysis results and the simplified predictions through overdamped elastic spectra, if the damping relation is period-depending and an adequate upper bound is considered. Indeed, the possibility of employing overdamped elastic spectra (obtaining results on the safe side) allows us to overcome difficulties and uncertainties in defining

the structural initial period and the behaviour factor q (to be used in the performance–based assessment procedures using inelastic spectra).
The best agreement between the two approaches is found with the medium seismicity records in the dataset. Non negligible scatter can be noticed between the displacement estimation for the SDOF systems in case of low seismicity area, but from the point of view of structural assessment and design, the maximum reference seismic demand in terms of displacement (D_{10} < 2 cm) is reasonably lower than the typical value of ultimate capacity of rocking masonry structures.

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