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Displacement Capacity of Ancient Structures through Non-Linear Kinematic and Dynamic Analyses

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ABSTRACT: In seismic prone areas, the topic of ensuring the buildings an adequate safety level is difficult to cope with. The public use related to several monumental buildings (churches, palaces, castles, etc.) sharpens the seismic risk, increasing the exposure factor. On this topic, in Italy, the recent seismic decrees (OPCM 3274/2003 and OPCM 3431/2005) have strongly modified the safety concept both for new-designed constructions and existing buildings. This code introduces the idea of design and verification through displacement-based analysis. Recent works propose an approach based on the Equilibrium Limit Analysis (kinematic theorem), which reliability needs to be verify through systematic numerical checks. For this purpose, non-linear dynamic analyses on the equivalent SDOF systems have been performed (Resemini et al. 2006). In this paper, the seismic performance of various church façades (Umbria region, Tuscany region) is evaluated using both non-linear kinematic and dynamic analyses. The estimated displacement capacity of the case studies is then compared to the surveyed damage pattern after the earthquakes in the '90s.

1 INTRODUCTION

The assessment of the consequences to monumental buildings, after recent Italian seismic events (Umbria and The Marches 1997, Piedmont 2000-2003, Apulia and Molise 2002, Lombardy 2004), has highlighted how, according to their architectonic complexity (geometry, constructive phases, transformations, etc.) and the poor tensile strength of the masonry, the damage and collapse often take place locally. Due to the dynamic action, the structure is subdivided into *macroelements* (Doglioni et al. 1994), which are characterized by a mostly autonomous structural behaviour on respect to the rest of the building. For the churches, representing the majority of the monumental heritage in Italy, the possibility of local failure is higher due to the presence of intrinsic vulnerability i.e. wide halls, long thin span vaults, slender towering or projecting parts, slender walls with large openings, different constructive phases, etc.

Observing the damage occurred in real cases, it was pointed out that, if the masonry shows good characteristics (regular texture, transversal connection between the leaves), the damage mechanisms develop as loss of equilibrium of rigid blocks capable of sliding and rotating.

From these considerations the use of the kinematic approach (Heyman 1966), based on the equilibrium limit analysis, has been adopted as a feasible criteria to check the safety of these local mechanisms. 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.

OPCM 3431/2005 seismic code proposes a displacement-based method, in which the structural capacity of the local mechanism is evaluated through the equilibrium limit analyses (kinematic theorem). In particular, kinematic analysis is recommended for the assessment of the horizontal acceleration that activates the mechanism and for the estimation of the ultimate capacity in terms of horizontal displacement.

Unfortunately, in case of monumental heritage, the OPCM 3431 seismic code does not propose a specific methodology, even though it points out the need for a quantitative evaluation.

The operational implementation of a displacement-based method for monumental building (both in terms of linear and non linear kinematic analysis) has been analyzed in the framework of recent works (Lagomarsino et al. 2004, Lagomarsino 2005).

The reliability of the proposed procedure needs to be verified and some aspects require further investigations. First of all, it has to be considered that some peculiar monumental structures (bell-towers, churches, obelisks) and their macroelements have high fundamental period of vibration still in the elastic range. Under the seismic action, their vibration period can further increase because of the nearly non-tensile strength of masonry, causing widespread cracking. The available experimental and numerical studies on this topic (Doherty et al. 2002) focus on lower fundamental period structures and, generally speaking, to structure typologies different from the monumental ones. Aiming both to verify the reliability of the previously proposed displacement-based simplified procedure for monumental building (Lagomarsino et al. 2004, Lagomarsino 2005) and to improve the representation, within his framework, of such "long-period" structures, systematic numerical checks via non-linear dynamic analyses on SDOF systems (equivalent to the macroelement) have been performed employing different input ground motions (Resemini et al. 2006).

In this paper the application of a simplified displacement-based approach, where overdamped spectra are used for the seismic demand representation, is proposed for the analysis of the expected response of different case studies, corresponding to churches damaged by Umbria-Marche (1997) and Lunigiana-Garfagnana (1995) earthquakes. In this way the reliability of the obtained results is evaluated from the comparison with the observed damage.

2 DISPLACEMENT CAPACITY OF MONUMENTAL BUILDINGS THROUGH NON-LINEAR KINEMATIC ANALYSES

The displacement-based method, proposed in the framework of OPCM 3431/2005, permits evaluating the expected seismic performance of a structure, assumed as an equivalent non linear single-degree-of-freedom (SDOF) system, by intersecting, in spectral coordinates (S_d, S_a), its seismic capacity curve with the seismic demand, described by the Acceleration-Displacement Response Spectra, adequately reduced in order to take into account the inelastic building behaviour. The intersection point, between seismic capacity and seismic demand curve, corresponds to the expected structural response for the earthquake, which the spectrum is related to and is referred as performance point (PP).

In case of overall seismic response of ordinary buildings, capacity curves are drawn implementing "pushover" analyses that lead to the evaluation of "pushover" curves, representing the building lateral load resistance (static equivalent base shear) versus its characteristic lateral displacement (peak displacement of the building roof).

As previously stated, if the object in study is the seismic performance of peculiar monumental buildings (bell-towers, obelisks, churches) or their macroelements, prone to local damage and collapse mechanisms, the capacity curves are evaluated by the use of the kinematic theorem of equilibrium limit analysis (Lagomarsino et al. 2004).

This procedure implies the a priori selection of the collapse mechanism that is the transformation of the structure in a kinematism, by positioning a sufficient number of hinges or sliding planes. Each resulting block is subjected to dead loads and to horizontal seismic action, proportional to the dead loads through a coefficient α. Under the hypothesis of non-tensile strength of masonry, unlimited compressive strength and rigid blocks, the seismic coefficient α_0 that induces the loss of equilibrium (and corresponds to the maximum strength) is obtained by the principle of virtual works (*Annex 11.C* in OPCM 3431/2005).

The virtual displacements are obtained by applying to the kinematism an infinitesimal variation of the equilibrium configuration (i.e. a rotation θ_k to one of the block k) and evaluating the rotations of the other blocks due to the kinematic mechanism, only considering geometry. The seismic performance of the structure is analysed till to the collapse by increasing the displacement d_k of a properly chosen control point k (at height y_{contr}) and applying the principle of virtual works to the corresponding configurations. The curve obtained through the incremental kinematic analysis can be transformed into the equivalent SDOF system capacity curve (Fig.1-a) by the identification of the equivalent modal mass coefficient m^* and the modal participation factor Γ (Annex 11.C, OPCM 3431/2005). Both the horizontal acceleration a_0 that activates the mechanism (representing the capacity in terms of strength) and the horizontal displacement d_0, representing the ultimate displacement, are computed. In Annex 11.C (OPCM 3431/2005), the capacity in terms of displacement (for the ultimate limit state) is given by 0.4 d_0.

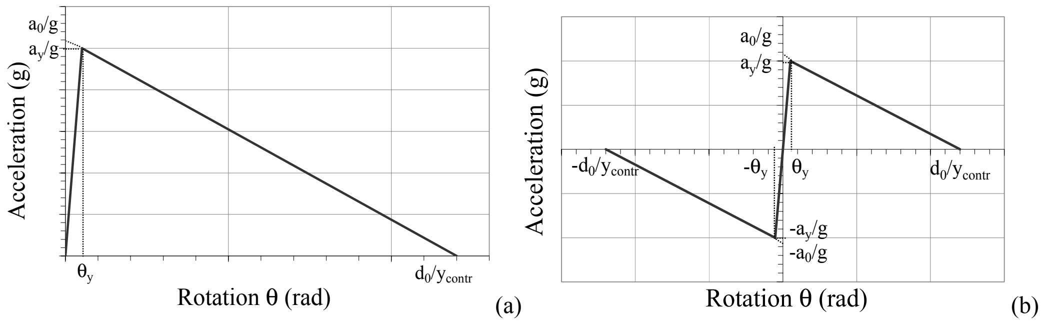


Figure 1 : Bi-linear capacity curve with initial ascending branch (approximated by a straight line), in terms of acceleration (unit of g) and rotation: (a) through the non-linear kinematic approach; (b) representing the reaction force r(θ) for the SDOF system for the dynamic analysis.

The capacity curve, obtained by transforming the seismic coefficient α and the control displacement d_k of the pushover curve, disregards the deformability of the macroelement that is involved in the collapse mechanism, as it is considered made by rigid blocks. Hence, an estimate of the vibration period T_0, associated to the mechanism in the phase preceding its activation, could be needed. The definition of the initial period is a difficult task dealing with kinematism of macroelements, as actually their initial dynamic behaviour is realistically related both to the element stiffness and to the structure stiffness, because of their interaction.

If the aim of the analysis is not only the checking of the ultimate displacement capacity of the structure (the one recommended by OPCM 3431/2005 seismic code), but also the assessment of which damage limit state the structure is expected to suffer due to an earthquake, this topic is even more significant. With regard to the representation of the seismic demand, some specific considerations are needed for the response spectra reduction (to account for non-linear characteristics of the motion) in the particular case of monumental buildings. First of all, it has to be pointed out that the use of inelastic spectra can not be proposed for monumental buildings or their macroelements being usually, as previously underlined, high period-structure at least after their cracking phase (T>2.5-4s). Their ductility is therefore expected to be higher then the limit value for which the reduction factors, function of the ductility (Fajfar 2000), can be employed. Moreover, with inelastic spectra, the resulting seismic demand is strongly influenced by the definition of the initial period T_0.

In order to overcome the limitation affecting the aforementioned procedures, the introduction of the overdamped elastic spectra (and secant stiffness corresponding to the performance point) in the simplified method (non-linear kinematic) in order to evaluate the maximum response in terms of displacement (or rotation θ) is applied in this paper. The reliability of the new proposed procedure has been investigated through systematic dynamic analyses (Resemini et al. 2006).

3 NON LINEAR DYNAMIC PROCEDURES FOR SDOF SYSTEMS EQUIVALENT TO MONUMENTAL BUILDINGS OR MACROELEMENTS

Referring to Resemini et al. (2006), the procedure for non-linear kinematic analysis of monumental structures, validated via non-linear dynamic analyses, is briefly described. Several SDOF systems equivalent to masonry monumental structures or to macroelements were tested: the study deals with the displacement response investigation for the medium-high period range, standing that rocking systems associated to monumental buildings may show large fundamental period of vibration (mainly in the inelastic phase).

The adopted model for the dynamic description of the equivalent SDOF system is similar to Housner's one, introducing the initial ascending branch for the overall stiffness (Fig. 1b).

The procedure involves the modification of the equations of motion, in particular of the reaction force $r(\theta)$, function of the rotation θ, substituting the relation of the capacity curve obtained by the non-linear kinematic method. The cyclic behaviour does not allow hysteretic dissipation, but, being non-linear elastic, the value of the equivalent viscous damping (beyond the initial range) have to be defined (it can be related to the rotation θ, through the secant period T corresponding to θ). The influence of the assumed damping relation on the maximum response of the structure has been examined.

Using the accelerometric database, provided by the research group of Milan Polytechnic (Italy), and statistically independent artificial acceleration time-histories (of different duration) matching EC8 response spectra, step-by-step dynamic analyses were performed, in case of different damping relationships.

The outputs from the dynamic analysis are in terms of displacement d_{max} (corresponding to the maximum rotation θ_{max}) and acceleration $a(T_{max})$, being T_{max} the secant period for θ_{max}. The results and the detailed formulation of these analyses are provided in Resemini et al. 2006.

4 VALIDATION OF THE PROPOSED NON-LINEAR KINEMATIC PROCEDURE VIA DYNAMIC ANALYSES

The proposed formulation to evaluate the maximum response in terms of displacement for monumental buildings or their macroelements makes reference to overdamped elastic spectra.

The use of overdamped spectra allows us to account for the influence of the equivalent viscous damping, that strongly increases when the initial phase is overcome, as clearly shown from experimental evidences about rocking elements (Doherty et al. 2002).

For the equivalent SDOF systems, the performance-point assessment resulting applying the proposed procedure is compared with the results obtained from the dynamic analyses (in terms of displacement). The equivalent viscous damping ratio ξ_{eq} to be adopted for the response spectra reduction has been evaluated in correspondence of the maximum rotation θ_{max} obtained by the dynamic analyses.

For this purpose, for each of the SDOF systems considered, the secant period corresponding to the resulting maximum rotation θ_{max} has been firstly assessed $T_{max}=T(\theta_{max})$. The equivalent viscous damping ratio $\xi_{eq}=\xi_{eq}(T_{max})$, corresponding to that period, has been evaluated according to the Type 3 correlation proposed for ξ_{eq} (Resemini et al. 2006). The factor for the elastic spectrum reduction has been computed according to the formula proposed by EC8 (eq. 3.6 in §3.2.2.2) as a function of the assessed ξ_{eq}. The obtained value for T_{max} on the overdamped spectra (reduced as a function of $\xi_{eq}=\xi_{eq}(T_{max})$) has provided the expected displacement response $S_d(T_{max})$, and the related acceleration $S_a(T_{max})$, according to the proposed non-linear simplified procedure. For the validation of the results obtained via the non-linear kinematic approach in terms of $S_d(T_{max})$ and $S_a(T_{max})$, the correspondence with the output from the dynamic analysis in terms of displacement d_{max} (corresponding to θ_{max}) and acceleration $a(T_{max})$ has been checked as explained in the following.

An important outcome (Resemini et al. 2006) 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 the artificial acceleration database), if the damping relation is almost independent from the initial period T_0 and an adequate upper bound is considered (Type 3 correlation).

For the validation of the simplified procedure, reference is made to a particular kind of result representation. In Fig. 2, an example is proposed; Fig. 2-a, using the Acceleration-Displacement Response Spectra (ADRS), shows:

- the elastic spectrum (damping ratio 5%);
- the capacity curves of two SDOF systems (SDOF1 and SDOF2), where the last point represents the value of d_{max} and the corresponding $a(T_{max})$ obtained by the dynamic analyses (representing the benchmark for these results).
- the overdamped spectra, for the two cases, where the reduction factor is related to the value of $\xi_{eq} = \xi_{eq}(T_{max})$.

In the case of SDOF1, the simplified procedure matches with the dynamic result (in a few words, the value of the equivalent viscous damping $\xi_{eq}(T_{max})$ corresponds to the correct spectral reduction). In the case of SDOF2, the simplified procedure does not provide a correct estimation. 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. In fact, the simplified procedure (without knowing the dynamic result) imposes to iterate, changing the secant period T, until the intersection of the capacity curve and the overdamped spectrum ensures the correspondence of the computed value of $\xi_{eq} = \xi_{eq}(T)$.

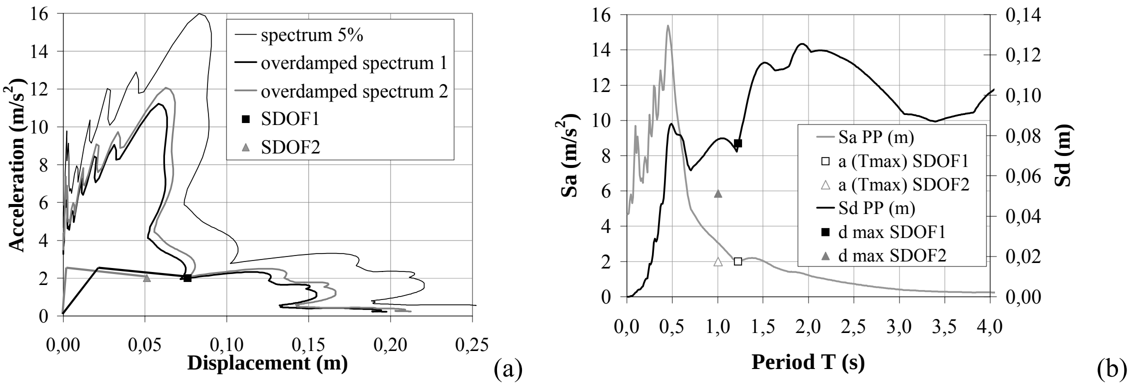


Figure 2 : Comparison between dynamic analysis and simplified procedure: (a) example in ADRS format; (b) example inserting an ordinate secondary axis.

In Fig. 2b, the same results are shown, inserting an ordinate secondary axis. The following quantities are represented:

- the displacement spectrum S_d vs. the period T; S_d is reduced by a factor function of the period, depending on $\xi_{eq}(T)$;
- the acceleration spectrum S_a vs. the period T; S_a is reduced by a factor function of the period, depending on $\xi_{eq}(T)$;
- the maximum displacement d_{max} resulting from the dynamic analysis vs. the period T;
- the acceleration $a(T_{max})$ 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, $a(T_{max})$ lays on the overdamped acceleration spectrum S_a (as in case of SDOF1).

4 IMPLEMENTATION AND COMPARISON OF THE PROPOSED PROCEDURE: THE CASE OF UMBRIA-MARCHE EARTHQUAKE (1997) AND LUNIGIANA-GARFAGNANA EARTHQUAKE (1995) - ITALY

As a further validation of the proposed approach for a non-linear kinematic analysis for monumental building, it is implemented in the following for the analysis of existing churches. Eight churches are analysed: referring to Table 1, churches 1 to 3 are located in Nocera-Umbra (Umbria) and churches 4-8 are located in various municipalities in Tuscany (Italy).

For these churches, damaged as a consequence of Umbria and Marche 1997 and Lunigiana and Garfagnana 1995 earthquakes, data on observed damage are available (GNDT Database 1997), surveyed according to a special form (Angeletti et al. 1997) that consider 16 different collapse mechanisms as possibly affecting the macroelements. Three levels (in this study transformed into five) for the surveyed damage related to the collapse mechanisms are accounted for in the form. All these churches, besides other issues, suffered some damage to the façade macroelement: the out-of-plane overturning mechanism developed, with less or more severe damage grade (Fig. 3).

(a) (b) (c)

Figure 3 : Damage for the overturning of the façade: (a) S. Giovanni church; (b) S. Maria church; (c) S. Bartolomeo church.

The information about the soil class of the church site, the macroseismic intensity I_{MCS} for the previously cited seismic events, the related PGA obtained through well-known relations (Margottini *et al.* 1992) can be found in Table 1. The reference peak ground acceleration (PGA) for the seismic zone (by OPCM3274/2003 decree) is $a_g = 0.25$ g.

4.1 Evaluation of the capacity curve through the Equilibrium Limit Analysis

For the case studies, the non-linear kinematic procedure is implemented for two different overturning mechanisms, typically identified in the post-earthquake survey: detachment between the façade and the transversal walls, in which the crack near the corner is sub-vertical (mechanism 1); failure in the transversal walls, in which the crack is oblique (mechanism 2), in Fig. 4. Both the kinematisms may occur involving the total or partial height of the wall.

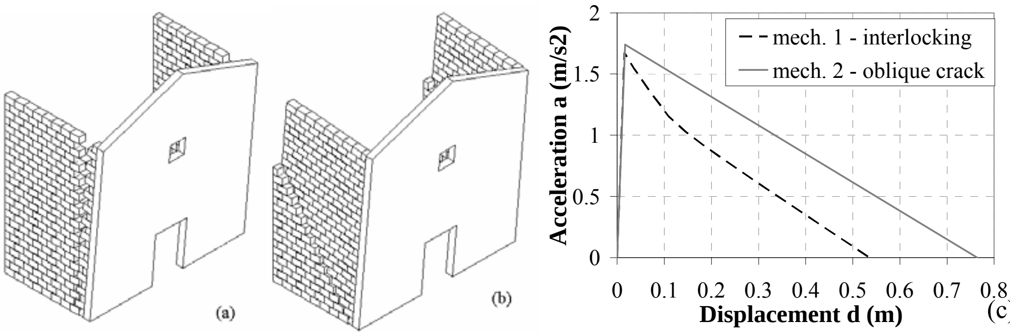


Figure 4 : Out-of-plane overturning of the façade macroelement: (a) mechanism 1; (b) mechanism 2; (c) capacity curves (SS. Giustina e Cipriano church).

The influence of the connection among orthogonal walls in the overturning mechanism was studied by various authors (de Felice and Giannini 2000, Restrepo-Vélez and Magenes 2004).

In particular, in the dissertation of de Felice and Giannini, the equilibrium limit method is adopted to evaluate the effective activation multiplier through a minimization procedure regarding some typical overturning mechanisms for masonry walls, studied as rigid blocks with unlimited compressive strength and non-tensile behaviour. However, the extremely simplified geometry, considered in the previously cited dissertation, is traceable in the building stock only in a few cases.

The mechanical model adopted in this study, even if based on geometrical data acquired through the survey form, photographic documentation and qualitative parameter about masonry characteristics, represents a development of the previous studies, in relation to various topics.

First of all, the evaluation of the effective activation multipliers for the mechanisms (Caprile 2005) accounts for the actual geometry of a church façade (i.e. openings, gamble shape, raising elements, etc.), the brickwork typology (i.e. dimension of the blocks and texture, in order to define the oblique crack in the transversal walls), the interlocking effect in the corner between the façade and the lateral walls (also due to friction on the sliding plane between the blocks).

Secondly, the model is not limited to the estimation of the activation multiplier, but the capacity curve is completely evaluated, through the non-linear kinematic approach.

In order to assess the displacement response, through the simplified and the dynamic analysis, the capacity curve of the SDOF system equivalent to the façade is defined (Fig. 4c). The procedure (see § 2) leads to the computation of the horizontal acceleration a_0 that activates the mechanism (representing the capacity in terms of strength) and the horizontal displacement d_0, representing the ultimate displacement.

In the study cases, for SDOF systems representative of church façades, the capacity properties in terms of acceleration a_0 and displacement d_0 are shown in Table 1. The initial period T_0 is imposed equal to a reference value of 0.4 s.

Table 1 : Main seismic parameters and data for capacity curve of the analysed churches.

No.	Church	Soil	I_{MCS}	PGA a_g (g)	Mech.	a_0 (m/s2)	d_0 (m)	y_{contr} (m)
1	S. Giovanni	B	8.5	0.16	1	1.2	0.30	8.2
					2	1.6	0.88	
2	S. Stefano	B	7	0.09	1	1.4	0.41	8.6
					2	1.5	0.90	
3	S. Maria	B	7.5	0.11	1	1.4	0.40	9.8
					2	2.0	1.09	
4	S. Maria Assunta	A	5.5	0.05	1	1.2	0.42	10.1
					2	1.8	1.00	
5	S. Bartolomeo Apostolo	B	5.5	0.05	1	1.5	0.33	11.2
					2	2.2	1.45	
6	S. Maria Maddalena	B	5.5	0.05	1	2.2	0.40	9.9
					2	1.9	1.12	
7	S. Michele Arcangelo	B	5.5	0.05	1	1.8	0.40	9.5
					2	1.8	0.91	
8	SS. Giustina e Cipriano	B	5.5	0.05	1	1.7	0.52	10.7
					2	1.7	0.77	

4.2 Displacement estimation through the simplified procedure and validation via dynamic simulations

The validation of the simplified procedure is based on the result comparison using 6 artificial acceleration time histories of different duration (15s and 20s) matching EC8 design response spectra. In Fig. 5, employing the result representation in Fig. 2-b, some results are proposed, using the EC8 response spectrum format, adequately reduced in order to consider the overdamping effect (continuous line).

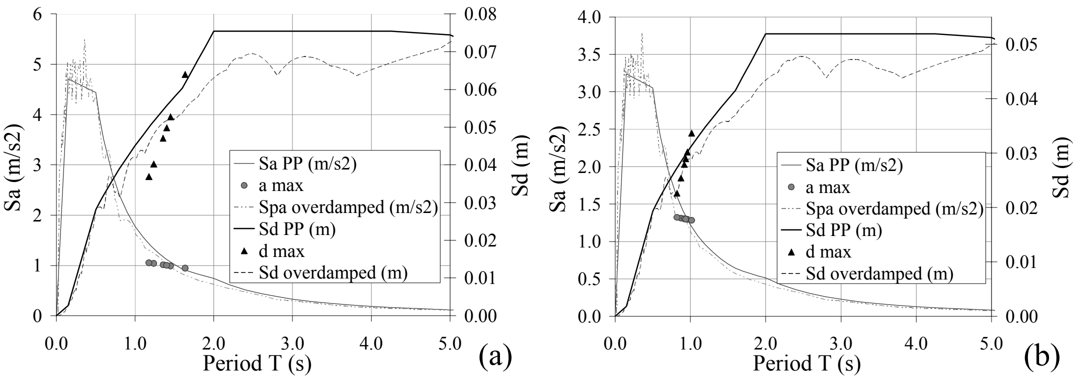


Figure 5 : Comparison between dynamic analysis and simplified procedure: (a) S.Giovanni church - mechanism 1; (b) S. Maria church - mechanism 1.

The average response spectrum of the 6 time histories (each derived by the maximum response of linear SDOF systems, evaluated in the time domain through a convolution integral, for which the damping ratio is computed on the basis of Type3 relation, as in Resemini et al.

2006) is also shown (dotted line): obviously, it does not perfectly match the target EC8 spectrum.

In particular, if the average response spectrum is accounted for, the forecasting obtained by the simplified procedure is quite well matching the dynamic results: each point represents the result in terms of displacement d_{max} (corresponding to θ_{max}) and acceleration $a(T_{max})$ for one time history. A certain scatter can be noted in Fig 5-a: in fact, the rather low input value of PGA leads the dynamic response to be confined in the non-linear low-medium period range, more affected by the input peculiarity (Resemini et al. 2006).

In some cases, the expected displacement is lower than d_y (the maximum displacement for the initial ascending branch). This trend can be noticed in many of the Tuscany churches.

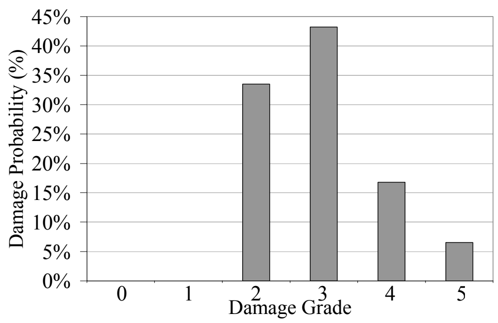
4.3 Damage estimation through the simplified procedure and comparison with the survey

In this study (making reference to Lagomarsino *et al.* 2004 for the description of the damage grade and OPCM 3431/2005), being d_0 the ultimate displacement and d_y the maximum displacement for the initial ascending branch, the mean threshold $S_{d,k}$ (k=1÷5) for each damage limit state may be so defined:

- Limit state 1 (slight damage): $S_{d,1}= 0.7\ d_y$;
- Limit state 2 (moderate damage): $S_{d,2} = d_y$;
- Limit state 3 (extensive damage): $S_{d,3} = 0.125\ d_0$;
- Limit state 4 (complete damage): $S_{d,4} = 0.25\ d_0$;
- Limit state 5 (collapse): $S_{d,5} = 0.4\ d_0$.

Through the simplified method, the deterministic evaluation of the mean damage grade is obtained considering the displacement of the performance point $S_d(T_{max})$ on each overdamped response spectrum of the 6 time histories, computing the average value $\overline{S_d(T_{max})}$ and associating the corresponding damage grade. A probabilistic assessment has been obtained, accounting for the variability of the $\overline{S_d(T_{max})}$ displacement due to the uncertainties affecting the: capacity curve evaluation, damage limit state definition and ground shaking prediction.

To this aim a lognormal cumulative distribution function has been assumed. According to this relationship, the probability of exceeding each defined damage limit state is evaluated as a function of the expected performance displacement $\overline{S_d(T_{max})}$ and of the displacement limit state threshold $S_{d,k}$ (HAZUS 1999). For the normalized standard deviation of the natural logarithm, a reference value $\beta_k = 0.7$ has been assumed. Nevertheless, it has been verified that the results are not remarkably affected by the variation of β_k parameter in the range from 0.6 to 1. Discrete probability damage distributions can be directly evaluated from the resulting cumulative density curves. As an example, in Fig.6-a, the damage distribution for the S. Giovanni church is shown.



(a)

Church No.	Mech.	Mean damage grade deterministic/ probabilistic	Damage grade (surveyed)
1	1	3/3.0	3.3 (mech 1)
2	1	2/2.0	5
3	1	2/2.2	3.3 (mech 1)
4	1	1/1.2	1.7 (mech 1)
5	1	2/1.3	1.7
6	2	1/1.0	0
7	1	1/1.1	1.7
8	2	1/1.1	0

(b)

Figure 6 : (a) Probability damage distribution for S. Giovanni church - mechanism 1; (b) Damage grade for the analysed churches.

The previously described procedures are developed for each of the two overturning mechanisms. The mechanism providing the higher value of the damage grade is selected, being considered the most vulnerable one. In Fig.6-b, these values are shown, for both the deterministic and the probabilistic evaluation (using a 0-5 damage grade scale), in relation to the surveyed ones. When possible (by the photographic documentation), one of the analysed typology (mechanism 1 and 2) has been associated to the surveyed damage mechanism. A good estimation can be noted in the majority of the churches. For S. Stefano church, the strong difference may be ascribed to the hammering effect on the façade due to the r.c. roof covering: this feature can not be accounted for in the mechanical method for the capacity curve definition and the high damage grade (collapse) surveyed is obviously not well approximated. Moreover, the “no damage” condition is not achieved through the proposed procedure, even if the computed mean damage grade for those cases is near the slightest one.

5 FINAL REMARKS

The proposed model for the seismic displacement forecast in case of monumental structures or their macroelements has been applied to case studies (churches located in the area of Umbria-Marche and Lunigiana-Garfagnana - Italy). An important outcome is the correspondence between the non-linear time-history analysis results and the simplified predictions through overdamped elastic spectra (employing the artificial acceleration database).

Moreover, the implementation in case of eight Italian churches (in particular, for the façade macroelement) gave us the possibility of comparing the mean damage grade (estimated by the non-linear kinematic approach using the overdamped spectra) to the surveyed damage pattern after the earthquakes in the ‘90s. It is worth noting that the definition of the thresholds for the damage limit states directly influences the results.

A further study on a wider building stock, possibly representative of various damage-grade conditions, should be needed in order to point out the method feasibility in the whole range and to review, if necessary, the definition of the thresholds. Moreover, the study should be extended to well-documented case studies: in particular, a detailed knowledge of the soil condition (to overcome site effects for seismic actions) is wished.

REFERENCES

Angeletti, P., Ferrini, M., Lagomarsino, S. 1997. Rilievo e valutazione della vulnerabilità sismica delle chiese: un esempio in Lunigiana e Garfagnana. *Proc. of the 8th Nat. Conf. ”Seismic Engineering in Italy”*. Taormina. September 1997. 2 (in Italian).

Caprile, P. 2005. *Non linear models for the seismic out-of-plane damage mechanisms in religious buildings*. Graduation Thesis. University of Genoa, Faculty of Engineering, Genoa (in Italian).

de Felice G., Giannini R. 2000. Out of plane seismic resistance of masonry walls. *J. of Earth.Eng.,* 5 (2), 253-271.

Doglioni, F., Moretti, A. & Petrini, V. 1994. *Le chiese e il terremoto.* Trieste: Edizioni LINT (in Italian).

Doherty, K.T., M.C. Griffith, N. Lam, J. Wilson, 2002. Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls, *Earth.Eng. and Struct. Dyn.*, 31, pp. 833-850.

Fajfar, P. 2000. A Nonlinear Analysis Method for Performance-Based Seismic Design, *Earth. Spectra*, 16, 3, pp. 573-592.

GNDT Database. *Seismic damage and vulnerability of the Umbria churches*. 1997. Research project - GNDT and the Italian Department of Civil Protection.

HAZUS. 1999. *Earthquake Loss Estimation Methodology - Technical and User Manuals*. FEMA.

Heyman, J. 1966. The stone skeleton, *International Journal of Solids and Structures*, 2, pp. 249-279.

Lagomarsino, S., Podestà, S., Resemini, S., Curti, E., Parodi, S. 2004. Mechanical models for the seismic vulnerability assessment of churches, *Proc. of IV SAHC*, Padova, Italy, A.A. Balkema, London (UK), 2, pp. 1091-1101.

Lagomarsino, S. 2005. Chapter7. Vulnerability assessment of historical buildings. In *Assessing and Managing Earthquake Risk - Geo-scientific and Engineering Knowledge for Earthquake Risk Mitigation: developments, tools, techniques. Series: Geotechnical, Geological, and Earthquake Engineering*, 2. Oliveira, Carlos Sousa; Roca, Antoni; Goula, Xavier (Eds.). ISBN: 1-4020-3524-1.

Margottini, C., Molin, D., Narcisi, B., Serva, L. 1992. Intensity versus ground motion: a new approach using Italian data. *Engineering Geology*, 33, pp. 45-48.

OPCM, no. 3274, 20 March 2003. Official Bullettin no. 105, 8 May 2003 (in Italian).

OPCM, no. 3431, 3 May 2005. Official Bullettin no. 107, 10 May 2005 (in Italian).

Resemini, S., Lagomarsino, S., Giovinazzi, S. 2006. Damping factors and equivalent SDOF definition in the displacement-based assessment of monumental masonry structures, *Proc. of 1st European Conf. on Earthquake Engineering and Seismology*, Geneva, Switzerland, 3-8 September 2006 (in press).

Restrepo-Vélez, L.F., Magenes, G. 2004. Experimental testing in support of a mechanics-based procedure for the seismic risk evaluation of unreinforced masonry buildings. in *Structural Analysis of Historical Constructions*

(C. Modena, P.B. Lourenço and P. Roca Eds), *Proc. of IV Int. Seminar SAHC*, Padova, Italy, A.A. Balkema, London (UK), ISBN 04 1536 379 9, Vol. 2, pp. 1079-1089.