Structural Analysis of Historical Constructions, New Delhi 2006
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

An "Innovative" Procedure for assessing the Seismic Capacity of Historical Tall Buildings: The "Torre Grossa" Masonry Tower

G. Bartoli, M. Betti, P. Spinelli and B. Tordini
Dept. of Civil Engineering - University of Florence, I-50139 Florence, Italy

ABSTRACT: Aim of this work is to propose a methodology for the evaluation of the seismic reliability of historical tall buildings. The evaluation of the seismic reliability of the masonry tower was done by a preliminary static and dynamic characterization of an elastic FEM, performed with respect to a series of "in situ" measures. By this method the identified model is used for the evaluation of the time-history of the global force acting on each sections due to a seismic load. After the evaluation of the time-history of each internal action, for some sections of the tower, the evaluation of the seismic reliability have been carried out by analyzing two limit states: (I) tower over-turning and (II) mechanical collapse of a masonry panel. The aim is to connect, for each of this limit states, an appropriate ground acceleration a_g able to assure their respect. The procedure is explained with respect to a case study: the "Torre Grossa" masonry tower.

1 INTRODUCTION

Due to their intrinsic geometric and material complexity, monumental buildings are by definition unique buildings, and they cannot be reduced to any standard structural scheme: this makes difficult to evaluate their seismic capacity. Furthermore, from a computational point of view, modeling of a masonry structure is also a difficult task, since masonry does not apparently respect anyone of the hypothesis assumed for other materials (isotropy, elastic behaviour, homogeneity), and appropriate constitutive laws for the materials are still not well developed. Modifications happened during centuries in the building history produced several uncertainties in the model definition (geometry, materials, connection etc.). Then a correct structural evaluation should be based on a deep knowledge of the: (i) building history and evolution, (ii) geometry, (iii) structural details, (iv) crack pattern and material damage map, (v) masonry construction technique and materials, (vi) material properties (Siviero et al. 1997) (Siviero et al. 1997). .This knowledge can be reached through both on site and laboratory experimental investigations, structural analyses with appropriate models and a final diagnosis. Since it is extremely difficult to get all the information necessary for a correct definition of a numerical (non-linear) model of an historical building, it is necessary therefore to have a simplified procedure for the evaluation of the seismic reliability that, taking into account only a reduced number of information, is able to address to the relevant aspect to the problem at hand.

Historical masonry buildings exhibiting a prevailing vertical character, such as towers and bell-towers, represent a structural typology with several common aspects: they are slender tall structures, which mainly have to support their own weight. These characteristics, together with all damages induced by several different factors during the years, make them particularly vulnerable with respect to (even small) base movements, such as those provoked by seismic actions or base settlements; the crack pattern which is inevitably present on these structures appear more or less typical of this kind of monuments (Salvatore et al. 2003, Giannini et al. 1996).

Moreover, the evaluation of structural reliability of towers and similar structures is quite demanding: these structures possess a low safety margin with respect to external actions, because of the high level of stress induced by the self weight if compared with the ultimate resistance of the materials utilized in the construction. The masonry characteristics and the compression level are besides responsible for the very low ductile overall behaviour of the whole construction. In the past years, several examples of sudden collapse of important towers have been experienced: in 1989 the Torre Civica in Pavia felt down (Macchi 1993, Binda et al. 1992), while in 1993 a collapse interested the bell-tower of St. Magdalena Church in Goch.

Herein it is proposed an original, to the authors' knowledge, methodology for the evaluation of the seismic reliability of historical tall buildings, with specific reference to medieval masonry towers. The proposed method is illustrated with reference to the seismic behaviour of a specific historical masonry tower: the medieval "*Torre Grossa*", in San Gimignano, the world famous town close to Siena (Italy).

2 METODOLOGY

The evaluation of the seismic reliability of the masonry tower was done by a preliminary static and dynamic characterization of an elastic FEM, performed with respect to a series of "in situ" measures. The dynamic identification allowed to evaluate the degree of constrain offered by the neighbour buildings, while the results obtained from the static identification have been used to tune up the mechanical properties of the smeared model.

After this preliminary identification, the elastic model of the whole structure has been used to evaluate the load acting to every section of the tower due to a specified earthquake (modeled by an appropriate accelerogram acting at the base). Loads acting at every section [z] of the tower were identified in global terms like shear force [T(z; t)], normal force [N(z)] and bending moment [M(z; t)]. After the evaluation of the time-history of each internal action, for a certain section of the tower, the evaluation of the seismic reliability has been carried out by analyzing two limit states:

- I limit state: tower over-turning (it is verified when the own weight combined with the seismic loads causes a resultant load which eccentricity is internal with respect to the cross-sectional area);
- II limit state: mechanical collapse of an external panel in its plane (it is verified when the seismic load acting on the tower is not able to produce a local cracking/crushing on an external panel of the tower).

Although masonry building exhibits a non linear behaviour, especially with respect to the severe stress tensile state induced by a seismic load, nevertheless the number of information (both mechanical and geometric) needed for an accurate and realistic non linear FEM analysis are difficult to obtain (Betti and& Vignoli, 2005a, b). In order to buildt an accurate non linear model of the construction it is important to know mechanical properties of each material, the crack pattern, the material damage map and also the actual distribution of the material along the building.

With the proposed methodology, with respect to some "in situ" measurement, a linear elastic model of the building has been done and it has been used in order to evaluate the load acting at each section of the tower. Next, the vulnerability of the construction moved to the analysis of the identified two limit states so that the non linear analyses moved from the whole model of the tower to the model of elementary panels. This is not a difficult task because the behaviour of the masonry panel is a well know problem, and an extensive literature exist for it (Tassios 1988). As a matter of fact, with respect to the elementary panels, it is possible to find in literature an extended series of experimental results that make the evaluation of ultimate load less uncertain, and then to evaluate the collapse surface.

The aim is to connect, for each of this limit states, an appropriate ground acceleration a_g able to assure their respect. The first limit state has been identified in the whole tower, while the second one is related to the behaviour of a single masonry panel with its actual non linear properties.

The respect of this two limit states result by the comparison between the resisting force *R* (evaluate upon geometrical aspects for the first L.S.; estimate upon the collapse behaviour of a masonry panel for the second L.S.) and the acting force *S* (obtained by the seismic load applied like ground acceleration time history). The seismic load acting of the tower's area is determined with reference to the EuroCode Recommendations [(EC8)].

3 CASE STUDY

The above discussed methodology of analysis is explained with reference to a specific case study: the medieval "*Torre Grossa*" (see Fig. 1a). This building is a tall masonry tower, dated as thirteenth century. It is the tallest and the most mighty of the towers preserved in the town of San Gimignano (Italy). The cross section is a square one measuring 9.5 x 9.5 meters, with an overall height of about 60 m. The walls are of variable thickness, between 2.6 m and 1.6 m. The sustaining wall is an infilled one ("*a sacco*" is the Italian term) with the external face made by stone masonry, and the internal layer constituted by brick masonry, with mortar layers nearly a centimetre thick. The internal filling is composed of heterogeneous material (remainders brick tied by a poor mortar). Up to the height of 20 m the tower is incorporated in a previous dated building, "*Palazzo Comunale*" (Town Hall). The floors have been realized through masonry vaults, while in the upper part of the tower a concrete floor is present, connected to the bottom part of the tower by a steel stair.

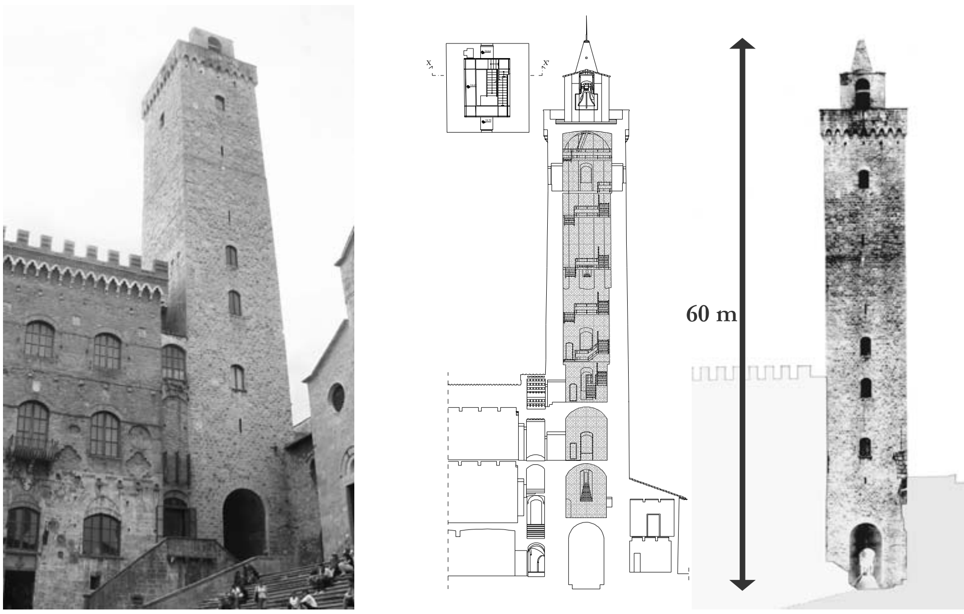


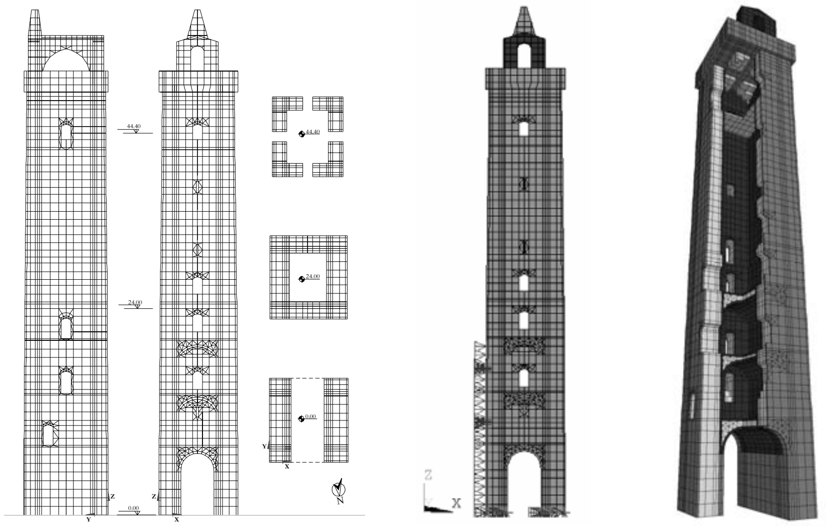
Figure 1 : (a) View of the tower with the neighbour "*Palazzo Comunale*" and (b) Section of the tower.

A FEM model of the masonry tower has been made taking into account the preliminary historical investigation and a geometrical relief (see Fig. 2a). Moreover, by the results of the "in situ" static and dynamic testing (that has been performed within the framework of the research contract called "San Gimignano Project", (Bartoli and& Mennucci, 2000) the numerical model has been identified in static and dynamic field. Particularly the dynamic identification has permitted to estimate the restrain degree offered by the neighbour *"Palazzo Comunale"* (Table 1).

Table 1 : Frequency and mode shape of the "in situ" experiment and 3D F.E. model

Modes shape	Direction	Experimental results [Hz]	Numerical result [Hz]
I	East - west	1.3060	1.2090
II	North - south	1.3310	1.3306
III	Torsional	3.4100	5.1836
IV	Vertical	---	5.9427
V	West - east	6.5500	6.2308
VI	North - south	7.6150	6.6617

The static identification has permitted an estimation of the elastic modulus of the internal filling; some creep analyses has been made (using an elastic time-dependent concrete model) in order to estimate an elastic modulus able to reproduce the actual compressive strength acting on the internal and external layer of the infill walls (Tordini, 2005). This is a crucial task since no experimental tests has been made on this heterogeneous infill material. Material properties adopted on the analyses are reported on Table 2.



(a) (b) (c)

Figure 2 : FEM model: (a) *mesh* details; (b) restrain offered by the neighbour building; (c) axonometric section.

Static and dynamic analyses have been carried out on the 3D model of the building structural complex using the F.E. computer code ANSYS. The masonry walls have been modelled by means of *solid45* (eight node isoparametric linear elastic elements). The 3D model consists of 24730 joints, 22417 3D *solid45* elements, corresponding to 74190 72350 dof.

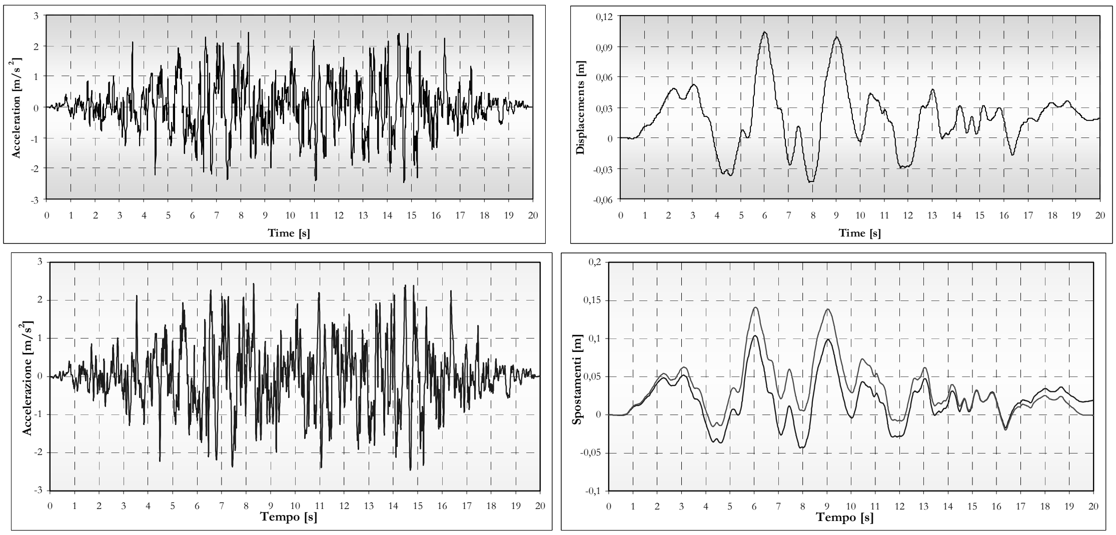
Table 2 : Material properties

Material	Elastic modulus E [kg/cm^2]	Own weight γ [kg/m^3]
external stone masonry	110000	2400
internal filling	16000	2000
internal brick masonry	30000	1800

3.1 I limit state

The earthquake load acting at the base of the tower has been modeledmodelled by an appropriate accelerogram generate by using the program SIMQKE. This accelerogram has a time length of 20 sec, and it has been generate according to the EuroCode 8 with respect to a ground class “B” and with a 475 return period Peak Ground Acceleration (PGA) equal to a_g=0,25g (see Fig. 3a). The seismic load has been applied on all restrained joints of the numerical model using the displacements time-history obtained by the design accelerogram. The size of the time step used is 0.02s. In this preliminary analysis the seismic load is assumed to act only in the x-direction: next steps well be devoted to the analysis of a load acting also in y-directions; due to the symmetry of the building no change on the results are expected.

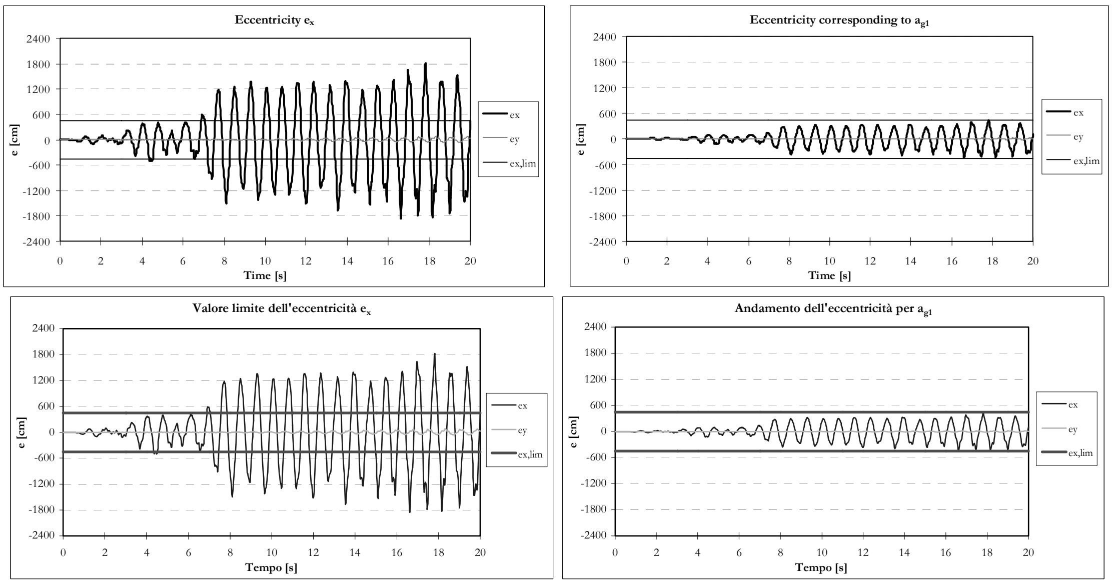
Loads acting at every section [z] of the tower due to the seismic load were identified in global terms like shear force [T(z; t)], normal force [N(z)] and bending moment [M(z; t)]. Due to the special configuration of towers (which act structurally as cantilever beams), the internal forces acting at each level are statically determined and therefore they can be estimated by a simple dynamic linear model.



(a) (b)

Figure 3 : Time history of (a) acceleration and (b) displacements.

Loads acting at every section [z] of the tower due to the seismic load were identified in global terms like shear force [T(z; t)], normal force [N(z)] and bending moment [M(z; t)]. Due to the special configuration of towers (which act structurally as cantilever beams), the internal forces acting at each level are statically determined and therefore they can be estimated by a simple dynamic linear model. After the evaluation of the time-history of each internal action, the time history of the eccentricity $e(z,t)=M(z,t)/N(z)$ has been evaluated (see Fig. 4a). With respect to the first limit state (tower over-turning) the tower is safe if, for each instant $|e_{max}| \leq |e_{lim}|$ where e_{max} is the maximum value assumed by e(z,t) during the loading time and e_{lim} is the value of the eccentricity of the normal force producing the over-turning (taken equal to the half length of the tower section)



(a) (b)

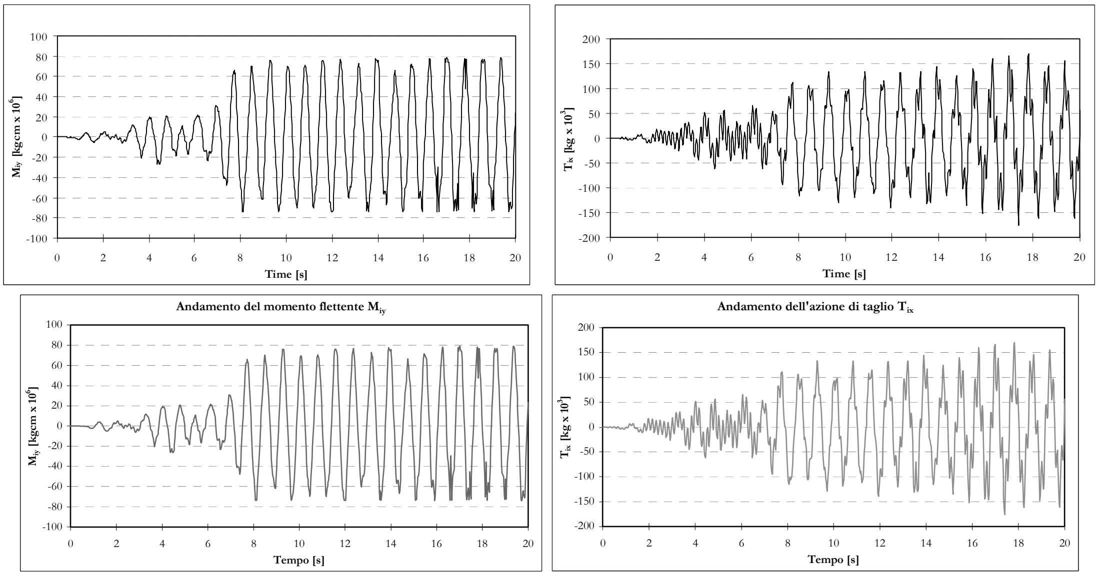
Figure 4 : (a) Time history of e(z;t) corresponding to a_g ; (b) time history of e(z;t) corresponding to a_{g1}

Aim of this first step is to evaluate the coefficient of reduction $\alpha=e_{max}/e_{lim}$ of the seismic input necessary to assure the respect of the first limit state. By this coefficient the maximum acceleration that the tower accept without over turning is $a_{g1}=a_g/\alpha$ (see Fig.4b).

3.2 II limit state

This limit state has been characterized to the aim to have a control on the local collapse mechanism (the so-called "second collapse mechanism"); by means of this limit state one checks that the seismic load doesn't produce a local crushing on the external layer of the infill wall of the tower (see Fig. 6a). In order to reduce the global load [T(z; t)], [N(z)] and [M(z; t)] acting on each section to the distinct elements that compose the entire section some hypotheses have been done. Particularly it has been assumed that masonry is an elastic linear no-tensile material and that cross sections will remain plane ones.

With this hypotheses, normal forces and bending moment acting on the all section are reported in terms of loads acting on each element distinguishing the case where the resulting load is whether internal or external to the internal core (that is the middle third of the section). Shear forces are reported only to those elements that are parallel to the seismic load considered.



(a) (b)

Figure 5 : (a) Time history of M(z;t); (b) time history of T(z;t).

With this hypotheses, normal forces and bending moment acting on the all section are reported in terms of loads acting on each element distinguishing the case where the resulting load is whether internal or external to the internal core (that is the middle third of the section). Shear forces are reported only to those elements that are parallel to the seismic load considered.

By this procedure, for each elementary area composing the tower section (see Fig. 6b), the loads acting are evaluated in terms of time history. Particularly the external stone masonry panels have been analyzed because these are the elements of the section that, with respect to the static loads, are the more critical (i.e. the elements with the high tensile stress). After that, for each area A_i the time histories of $T_{ix}(t)$ e $M_{iy}(t)$ are known, and they have been reported in a diagram, see Fig. 8a. Each point in this figure is representative of an instantaneous combination [T(z; t), M(z; t)] due to the time history earthquake loading.

This loads could be considered as those acting at top surface of an elementary panel (a stone masonry panel, see Fig. 6b). In order to assess if the loads previously evaluated are admissible for this panel the collapse domain has been determined.

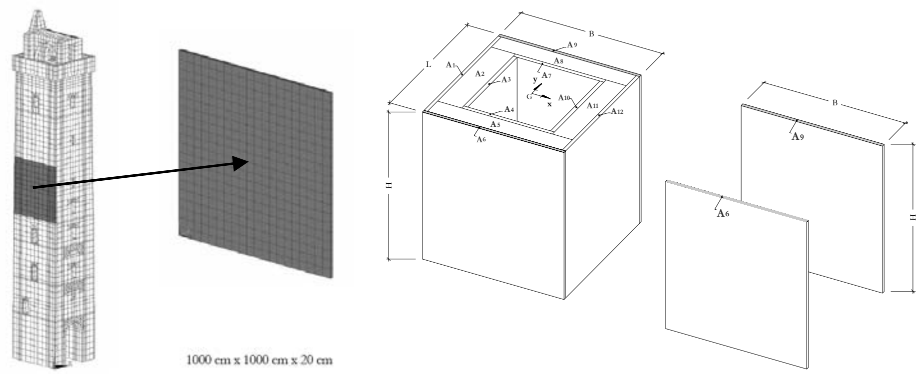
A numerical non-linear model with ANSYS of the analyzed square panel (B=H=9.5 m and thickness s=0.2 m) with brick elements has been realised. *Solid65*, eight nodes isoparametric elements with crushing and cracking features, have been used (see Fig. 7a, b). For these elements the hypothesis of non linear elastic behaviour has been adopted. Non linear properties of masonry have been modelled by using the yield Drucker-Prager criterion with associated flow rule. The yield surface has been assumed not to change with progressive yielding, hence there is no hardening rule and the material is elastic-perfectly plastic. The assumed failure surface is the Willam and Warnke surface. Table. 3 reports the selected values for the model parameters.

Table 3 : Yield criterion and failure surface setting.

Yield Drucker-Prager criterion		Willam and Warnke failure surface	
c	6.15 kg/cm^2	f_c	55 kg/cm^2
δ	10°	f_t	3.5 kg/cm^2
φ	51°	β_c	0.75
		β_t	0.15

f_c (uniaxial compressive strength); f_t (uniaxial tensile strength); β_c (shear transfer coeff. close crack); β_t (shear transfer coeff. open crack); c (cohesion); δ (dilatancy); φ (internal friction angle)

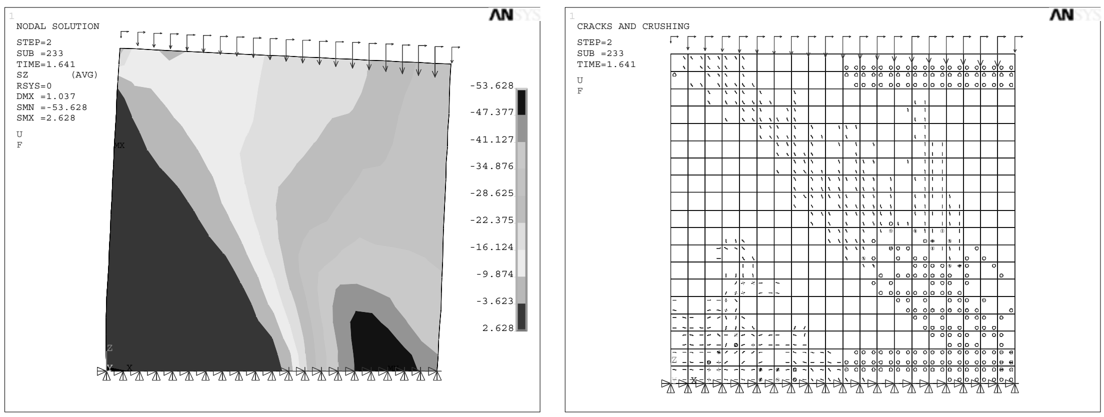
In order to obtain the single bending condition for the bottom surface of the panel, rigid restraint condition has been imposed.



(a) (b)

Figure 6: (a) Panel definition; (b) Elementary areas.

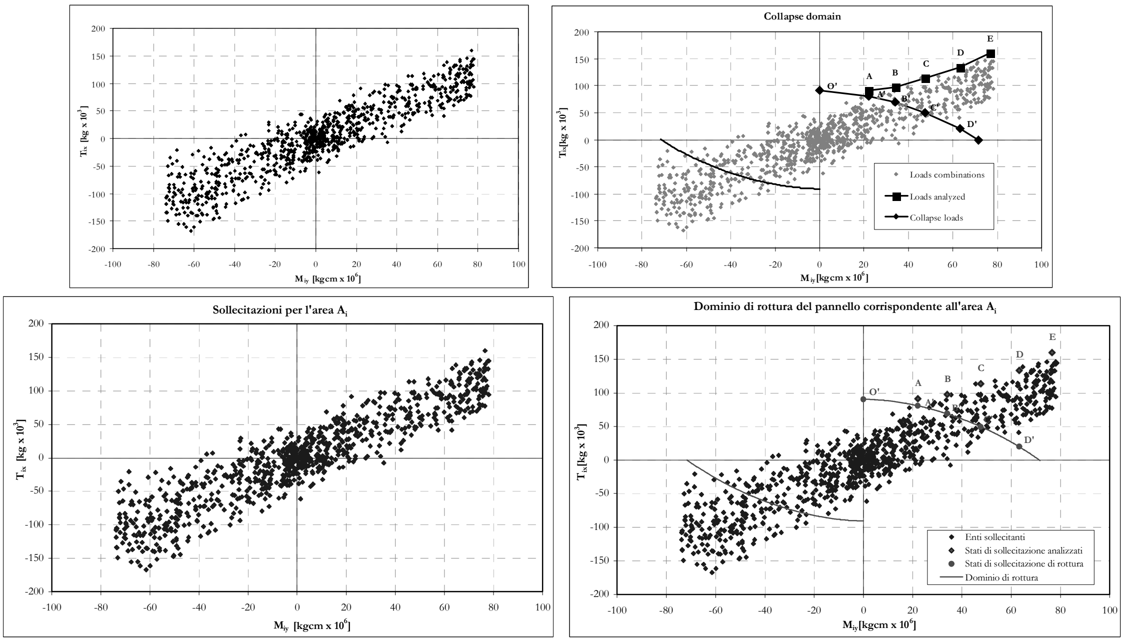
Load combination of Fig. 5a, b are used in order to develop the collapse analysis on the panel. Continuous line in Fig. 8b represents the boundary of the collapse domain for the selected panel; this boundary is evaluated by some simple analyses in non-linear field by the interpolations of some analysed combinations (points A'÷E' in Fig. 8b).



(a) (b)

Figure 7 : (a) Vertical stress σ_{zz} (kg/cm^2); (b) cracking pattern on the panel.

The transmissible acceleration a_{g2} it is estimated to be the acceleration that make all combination [T(z;t), M(z;t)] inside the collapse domain



(a) (b)

Figure 18 : (a) Load combination [T(z; t), M(z; t)]; (b) collapse domain (shear - bending)

If the couples [T(z; t), M(z; t)] are external to the collapse domain previously determined the input seismic load have been reduced accordingly, in order to assure the admissibility for the load acting on the panel. This iterative procedure is repeated until each couple [T(z; t), M(z; t)] is internal to the collapse domain. The final aim of this step is to obtain the admissible seismic input. The value of the maximum ground acceleration have been named a_{g2} and it is the value of the ground acceleration that assure the respect of the II L.S. Substantially the transmissible acceleration a_{g2} is estimated to be the acceleration that make all combination [T(z;t), M(z;t)] inside the collapse domain.

The proposed methodology for the evaluation of the seismic vulnerability of masonry tall building applied to the "Torre Grossa" has permitted to estimate two index of vulnerability of this tower. Results of the analysis showed that the tower is, how it was predictable, extremely sensible to seismic loads. With the proposed method it is possible to assess, from a quantitative point of view, its vulnerability. Particularly, the respect of the first limit state is assured (with respect to the three sections analysed) by a seismic load that is 17% of the seismic load defined by the Italian rule. Second limit state is verified when the value of the ground acceleration is $a_{g2} = 0.0268g$, approximately 10% of the value of a_g defined on the Italian rule..

4 CONCLUSIONS

A methodology for the evaluation of the seismic reliability of historical tall buildings has been proposed. The evaluation of the seismic reliability of a masonry tower was done by a preliminary static and dynamic characterization of an elastic FEM, performed with respect to a series of "in situ" measures. By this method, the identified model is used for the evaluation of the time-history of the global force acting on each sections due to a seismic load. Load acting at each section of the tower are identified in terms of global actions like shear forces [T(z; t)], normal forces [N(z)] and bending moments [M(z; t)].

After the evaluation of the time-history of each internal action, for some sections of the tower, the evaluation of the seismic reliability have been carried out by analyzing two limit states: (I) tower over-turning and (II) mechanical collapse of a masonry panel.

The aim is to connect, for each of this limit states, an appropriate ground acceleration a_g able to assure their respect. In order to explain the procedure a case study: the "Torre Grossa" masonry tower has been developed. It is important to point out that the proposed methodology has ve two main benefits: the first one is that only a reduced number of experimental measurements is necessary in order to calibrate the model; the second one is that since the whole model of the building is a linear, one the computational effort needed for the analysis is not heavy (non linear analyses are developed only on a reduced model of an elementary panel).

REFERENCES

Bartoli, G. and, Mennucci, A. 2000. Progetto S. Gimignano, la Torre Grossa: indagini conoscitive e diagnostica. In *Atti del primo convegno su la diagnostica per la salvaguardia dei beni culturali, S. Gimignano, Teatro Comunale, 6-7 Dicembre 1999.*

Betti, M., and Vignoli, A. 2005a. Dynamic and seismic performance of a basilica-type church under earthquake loading: the Farneta abbey. *Proceedings of the Ninth International Conference on Structural Studies, Repairs and Maintenance of Heritage Architecture, STREMAH 2005*, pp. 203-213.

Betti, M. and, Vignoli, A. 2005b. L'utilizzo del codice ad elementi finiti ANSYS per l'analisi strutturale di edifici monumentali in muratura. *Analisi & Calcolo* VI(21), Consedit Sas ed., pp. 27-31.

Binda, L. , Gatti, G., Mangano, G., Poggi, C. and, Sacchi-Landriani, G. 1992. The collapse of the Civic Tower of Pavia: a survey of the materials and structure. *Masonry International* N. 1, Stoke-on-Trent, 1992.

Carpinteri, A. Invernizzi, S. and Lacidogna, G. 2005. In situ damage assessment and nonlinear modelling of a historical masonry tower. *Engineering Structures* 27, p. 387–395.

Del Piero, G. 1983. *Il consolidamento delle costruzioni*, CISM, Udine.

Eurocode EC8 1994. Design provisions for earthquake resistance of structures. *European Committee for Standardisation*. Brussels, Belgium.

Giannini, R. , Pagnoni, T., Pinto, P.E. and, Vanzi I. 1996. Risk analysis of a medieval tower before and after strengthening, *Structural Safety* 18 (2/3), pp. 81-100.

Macchi, G. 1993. Monitoring Medieval Structures in Pavia. The collapse of the Civic Tower. *Structural Engineering International* N.1.

Salvatore, W., Bennati, S., and Della Maggiora, M. 2003. On the Collapse of a Masonry Tower Subjected to Earthquake Loadings. *Fourth International Conference on Earthquake Resistant Engineering Structures, ERES 2003, 22-24 September 2003, Ancona, Italy.*

Siviero, E., Barbieri, A. and, Foraboschi, P. 1997. *Lettura strutturale delle costruzioni*. CittàStudiEdizioni, Milano.

Tassios, T.P. 1988. *Meccanica delle murature.*, Liguori Editore, Napoli

Taddei, D., Corazzi, R. and, Petrini G. 1997. *Progetto San Gimignano.*, Alinea Ed., Firenze.

Tordini, B. 2005. La Torre Grossa di San Gimignano: modellazione strutturale e valutazione della sicurezza sotto l'azione sismica. *Bachelor Thesis*, University of Florence.