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DIFFERENT VERIFICATION LEVELS IN THE EVALUATION OF THE SEISMIC CAPACITY OF MASONRY STRUCTURES

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Abstract

A variety of computational procedures has been developed, over the years, for the evaluation of the seismic capacity of the architectural heritage. Due to the peculiarities of box-type masonry structures, indeed, this problem is not suitable for the application of procedures commonly in use for r.c. and steel structures, based on the linear analysis of a frame model. The experience acquired in Italy following the seismic events in the last forty years has led to the development and subsequent codification of three different kinds of approach to this problem; in relation also to the progress of the computational tools, the procedures are associated to different levels of complexity and accuracy in the results.

Initially, attention was focused on simple computational tools, suitable for manual application. Within that context, the basic approach to the evaluation of the global shear resistance for a building was formulated. This kind of approach, although approximate, allows for a global and fast estimation of the seismic safety. For this reason, it has been recently revised and codified in the Italian Guidelines for the reduction of the seismic risk of heritage buildings.

In the following years, through the experience coming from new earthquakes, it was possible to identify typical, systematically recurring collapse mechanisms, involving limited portions of the building. On this basis, with reference to limit equilibrium conditions, the analysis of local collapse mechanisms was developed and, subsequently, formalized in the building code.

Finally, in parallel to the progress of computational tools, global analysis systems have been developed, based on frame equivalent models of masonry buildings, incorporating suitable schemes for masonry elements; such systems are normally used for static non linear analyses. This approach is recognized by the Guidelines as the most accurate evaluation of the building seismic capacity.

It is of interest to apply the above procedures to the same case study, in order to compare the three different evaluations of the seismic safety, and to compare the kind of information they can provide. In this work, this has been done with reference to an ancient masonry structure in the city of Naples, for which a detailed survey had been performed.

Keywords: masonry; seismic safety; computational tools; verification level

1. Introduction

A variety of computational procedures has been developed, over the years, for the evaluation of the seismic capacity of buildings belonging to the architectural heritage. Due to the peculiarities of box-type masonry structures, indeed, this problem is not suitable for the application of procedures commonly in use for r.c. and steel structures, based on the linear analysis of a frame model. The experience acquired in Italy following the seismic events in the last forty years has led to the development and subsequent codification of three different kinds of approach to this problem; in relation also to the progress of the computational tools, the procedures are associated to different levels of complexity and accuracy in the results.

Initially, attention was focused on simple computational tools, suitable for manual application. Within that context, the basic approach to the evaluation of the global shear resistance of a building was formulated. It was mainly based on the Turnsek-Cacovic [1] formula for the material shear resistance. This kind of approach, although approximate, allows for a global and fast estimation of the seismic safety. For this reason, it has been recently revised and codified in the Italian Guidelines for the reduction of the seismic risk of the architectural heritage [2].

In the following years, through the experience coming from new earthquakes, it was possible to identify typical, systematically recurring collapse mechanisms, involving specific portions of the building. On this basis, with reference to limit equilibrium conditions, the analysis of local collapse mechanisms was developed and, subsequently, formalized in the building code. Again, a global safety evaluation is possible, based on the recognition and the analysis of all the partial mechanisms which are likely to be activated during a seismic event.

Finally, in parallel to the progress of computational tools, global analysis systems have been developed, based on frame equivalent models of masonry buildings, incorporating suitable schemes for masonry elements; such systems are normally used for static non linear analyses. This approach is recognized by the above mentioned Guidelines as an accurate evaluation of the seismic capacity of the building.

The application of the above procedures to a single case study allows for a meaningful comparison of the results provided by the different procedures in terms of seismic safety, also clarifying the kind of information coming from each of them. In the present work, all this is done with reference to an ancient masonry building in the city of Naples, for which a detailed survey had been performed, providing an accurate definition of both geometric and material properties. As a consequence, a meaningful application of the above procedures has been possible, leading to reliable results for the seismic safety conditions of the building.

2. The case study

Within a national program addressing the seismic safety of museums, an interesting case was offered by a masonry building belonging to the royal house complex at Capodimonte in Naples (Italy). The four story building (see fig. 1-a), commonly denominated *Palazzotto Borbonico*, dates back probably to the beginning of the 18th century; in a plan view, the shape is approximately rectangular, with dimensions 24×15 m (see fig. 1-b) and corresponds to a reduced portion of a wider complex, which was partly destroyed and partly demolished during WW2. In 1980 an r.c. building was completed, adjacent to the old one, yet constituted by a dynamically independent structural unit; the safety evaluation, therefore, has to do with the old masonry portion only.

Masonry is composed by tuff stone blocks, with size of about 30 × 20 × 20 cm (length × width × height); tuff stone, which is made of volcanic ashes, is a relatively soft rock, often used in Italy since ancient times. In Naples, a specific cream colored variety has been in use for a long time.

According to the Italian Code provisions for existing structures [3], safety computations have to be based on preliminary studies aiming at reaching the best possible knowledge level for the building. Such studies should consist in a critical historical analysis, in the geometrical survey, in the material characterization, in the damage survey, and in the definition of seismic hazard data specific for the building location. As a result of these activities, the "confidence factor" can be defined in the range of values between 1 and 1.35. In the specific case, a value of 1.12 has been adopted, based on the knowledge level reached through the performed studies, briefly recalled in the following.

a) b)

Fig. 1 – The analyzed masonry structure: global view (a) and plan view (b).

From the analysis of available historical data, transformations which led the original building layout to the present one have been highlighted. Specifically, in relation to the floor slabs a complex situation has been described, with some timber slabs still preserving the original configuration, some others reinforced by steel beams and some others substituted with a concrete structure. This last case applies also to the present roof system, consisting in a flat concrete slab.

To the purpose of material characterization, both thermographic analyses and sonic testing have been performed. In general, a good homogeneity level has been found for masonry, with a regular texture for the tuff stone blocks, as also confirmed by endoscopic testing. No evidence of irregularities, like openings closed by infill masonry panels, is present over the entire building. From sonic testing, a propagation velocity of 1000 m/s has resulted, corresponding to high density material. Double and single flat jack testing has been performed at some locations at the lowest level; a value of about 0.22 N/mm^2 has been found for the vertical stress and of 1650 N/mm^2 for the elastic modulus at the same stress level.

As to the seismic hazard characterization for the specific site, detailed information come from the national hazard map, which specifies response spectrum data at the nodes of a 5×5 km grid, to be used in connection with an interpolation procedure to characterize the response spectrum at any specific location. All the same, geophysical field tests have been performed for an accurate and realistic definition of local seismological properties; as a result, a less severe acceleration response spectrum has been obtained, compared to the conventional one, as shown in fig. 2. This spectrum refers to the event characterized by a 10% exceedence probability in 50 years, i.e., by a 475 years return period. From field testing, also the local soil stratigraphy has been interpreted. An average shear wave velocity of 260 m/s has been obtained for the soil top layer with a thickness of 30 m; according to the Eurocode 8 [4] classification, this corresponds to a C class soil.

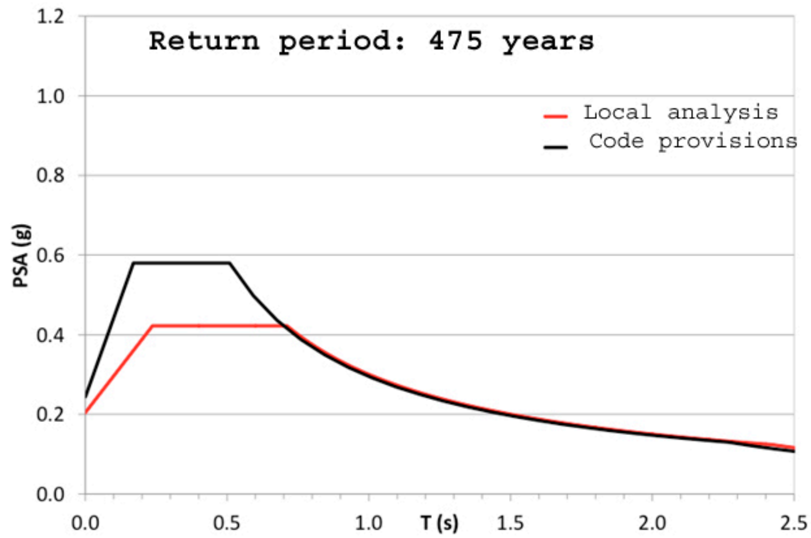

Fig. 2 – Response spectra.

3 Verification level 1: global shear capacity

A simplified procedure for a global evaluation of the building seismic resistance based on the shear capacity of masonry walls has been in use in Italy for almost 40 years after the Friuli earthquake of 1976. The procedure comes from the application of the material shear resistance, as expressed by the formula proposed by Turnsek and Cacovich [1], to the resistance evaluation of each single wall panel and, subsequently, of the entire system. In the well known formula shear resistance (τ_d) is expressed through the characteristic value (τ_k) amplified by a coefficient depending on the vertical compression stress (σ_0):

$$\tau_d = \tau_k \cdot \sqrt{1 + \frac{\sigma_0}{1.5 \cdot \tau_k}} \tag{1}$$

Assumptions on the wall panel stiffness (bending plus shear stiffness) and on limited ductility resources allow to define a capacity curve for the global wall system. On the basis of repeated experiences about the response of masonry buildings to earthquakes, the procedure has been revised over the years and a new formulation was incorporated in the above mentioned Guidelines in 2011 [2]. The procedure is intended for a double use: either for an easy estimation of the global resistance of the masonry building, or for a fast comparative evaluation of a group of buildings to the purpose of highlighting dangerous situations deserving urgent strengthening interventions.

Applying this method requires first to define the global shear resistance (F_{slv}) at each floor level (i) for both the horizontal directions. In the x direction:

$$F_{SLV,x,i} = A_{x,i} \cdot \tau_{di} \tag{2}$$

where A_{xi} denotes the total area of walls in the x direction and τ_{di} the design value for the material shear resistance according to (1). For the characteristic value τ_k, in the absence of specific experimental data, a meaningful reference is given by a table incorporated in the Italian Building Code [3], which specifies typical intervals for the mechanical properties of common masonry typologies. The table comes from a long classification work of the material varieties which are normally encountered, thus becoming a meaningful reference. For the tuff stone masonry, the typology defined as “soft stone blocks masonry” has been chosen as appropriate; taking the average of the maximum and minimum proposed values, after dividing by the confidence factor (1.12), a final value of 3.125 N/cm^2 has been obtained.

	$F_{SLV,x}$ *(kN)*	$F_{SLV,y}$ *(kN)*
Level 0	***<u>2596</u>***	*4389*
Level 1	*2698*	*4308*
Level 2	*3868*	*4134*
Level 3	*3954*	*7653*

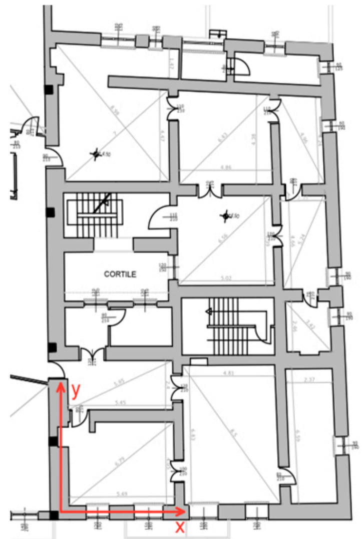

Fig. 3 – Building plan view for the identification of resisting walls in x and y directions (right) and shear capacity values (left) for all levels (base equivalent).

The global shear resistance is then modified through a number of coefficients, to take into account several aspects in a more or less conventional way:

$$F_{SLV,x,i} = A_{x,i} \cdot \tau_{0d} \cdot \frac{\mu_{xi} \cdot \xi_{xi} \cdot \zeta_{xi}}{\beta_{xi} \cdot \kappa_{xi}} \tag{3}$$

In brief, the meaning of single coefficients is the following:

- μ_{xi} conventionally accounts for the stiffness homogeneity of masonry panels; the variation range is between 1 and 0.8 with the lower value (0.8) corresponding to remarkable differences in the wall cross sections. In the present analysis, the value of 0.8 has been adopted in both the x and y direction;
- ζ_{xi} is a reduction factor to take into account a collapse modality characterized by bending plus compression rather than shear. Conventionally, a value of 0.8 is adopted if the collapse modality is not by shear. In this analysis values of 1 in the y-direction and 0.8 in the x-direction have been used;
- β_{xi} is a plan irregularity coefficient depending on the eccentricity between the mass and stiffness centers. The variation range is from 1 to 1.25. An analytical evaluation is possible for it; used values are 1.09 in x and 1.23 in y;
- ξ_{xi} conventionally reflects the presence of stiff rather than deformable floor beams. In the latter case the value is reduced from 1 to 0.8. In this study, the reduced value of 0.8 has been adopted in both directions;
- κ_{xi} is a transformation coefficient to convert the value of shear resistance at floor i into the equivalent value at the base of the building. In this way, resistance values computed at floor levels can be directly compared.

The force representing the best estimate of the building shear resistance in the weak direction (x in the case examined) is then converted into the equivalent spectral acceleration ($S_{e,SLV}$) dividing by the first mode effective mass (M*) and multiplying by the behavior factor (q):

$$S_{e,SLV} = \frac{F_{SLV} \cdot q}{M^*} = 2.34 \;\; m/s^2 \tag{4}$$

Following the indications given in the Guidelines, a value of 2.5 has been used for q, which represents an average situation from the point of view of the available ductility resources, the variation range for the parameter being from 2.25 to 2.8.

The final step consists in the conversion of the spectral acceleration, corresponding to the structural response, into the equivalent peak ground acceleration. This necessarily requires an iterative procedure; in the case here examined it leads to a ground acceleration of 1.13 m/s^2, which can be compared to the design acceleration prescribed for new structures at the same location (a_0). A safety coefficient of

$$f_a = \frac{a_{MAX}}{a_0} = 1.13/2.34 = 0.564 \qquad (5)$$

comes out as the final result of this procedure.

The above coefficient provides a global evaluation of the building seismic safety and, in this sense, it is of clear interest; it has to be considered, however, that it is based on some conventional assumptions, which limit its validity and call for the need of a detailed global analysis of the structure.

4 Verification level 2: local collapse mechanisms

From experiences repeatedly collected after earthquakes in terms of collapse modalities, it has been recognized that safety should not be verified with reference to the building global collapse only, but in relation to the possible activation of local collapse mechanisms as well. From the computational point of view, this kind of analysis can be performed easily, as it involves the identification of structural portions undergoing rigid body motion and the analysis of equilibrium at ultimate conditions. The partial collapse mechanism method has acquired importance in the seismic safety verification of buildings as a number of meaningful collapse modalities have been recognized and classified, in relation to current building typologies [5].

In the computational procedure, imposing equilibrium at ultimate conditions leads to the definition of the seismic load multiplier at collapse; this last has then to be converted into the corresponding acceleration considering the effective mass associated to the mechanism. Finally, going through the response spectrum, the associated peak ground acceleration can be defined. In case the mechanism takes place at some level in the building, assumptions have to be done on the mode shape and on the corresponding acceleration profile along the building height. All this has to do with the so called "linear kinematic analysis", which could be extended to the non linear range as a displacement analysis. In the following, reference is done to the linear analysis only.

Fig. 4 – Identification of possible partial collapse mechanisms.

In this study case, preliminary to the recognition of meaningful collapse mechanisms, it has to be underlined that basic conditions for the activation of local mechanisms are present. The masonry quality, indeed, as shown by thermographic and sonic testing, is characterized by good texture and effective connection of orthogonal walls; in addition to this, no meaningful crack pattern can be observed. Under such conditions, the activation of local collapse mechanisms is possible, in the sense of separation of masonry rigid blocks due to the formation of rupture lines along weak directions.

From a global view of the structural complex, also considering the arrangement of windows, the presence of perimeter tie beams, the slab orientation, the effect of transversal walls, two different situations have been identified for the activation of local mechanisms (see Fig. 4):

1. at the building South corner, with the separation of a corner edge including portions of both orthogonal walls. Three different modalities have been considered, with only the third floor involved, or the second and the third, or all floors (Fig. 6);
2. on the rear façade, top floor, with the out-of-plane collapse of the wall in two different conditions (Fig. 5):
   - wall overturning due to rotation around the base,
   - "vertical bending mechanism", with the formation of hinge lines at the top, at the bottom and along an intermediate line; for this last, three different locations have been considered, as suggested by the presence of window openings.

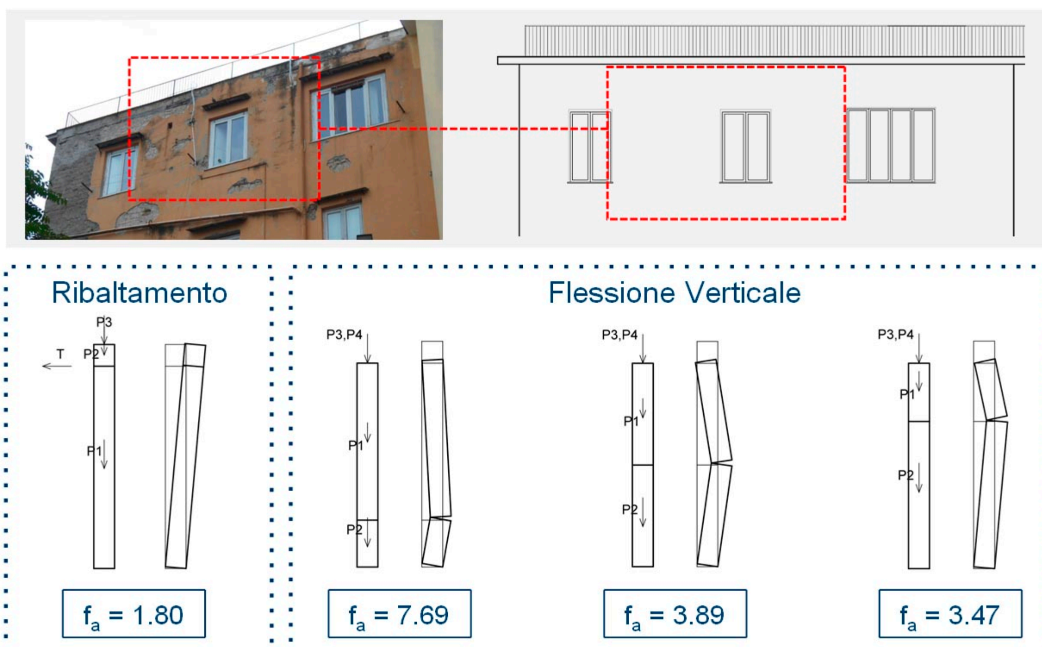

Fig. 5 – Partial collapse mechanisms for the rear façade.

A total of seven partial collapse mechanisms, therefore, have been analyzed. Results in terms of the f_a seismic safety coefficient, as defined in (5) are shown in Fig. 5 and Fig. 6. In most cases, high values have been obtained; specifically, out-of-plane mechanisms for the rear façade can not be easily activated due to the retaining action produced by the roof ring beam. As to the corner mechanisms, the safety coefficient is less than 1 only in the case when all the floors are involved. For this kind of verification, however, the more sophisticated non linear approach should be followed, which typically leads to more favorable results. In addition, it has to be considered that such reduced value by far exceeds the one found for level-1 verification; in the corresponding ultimate condition, therefore, the structure box-type behavior would not be compromised by any local collapse.

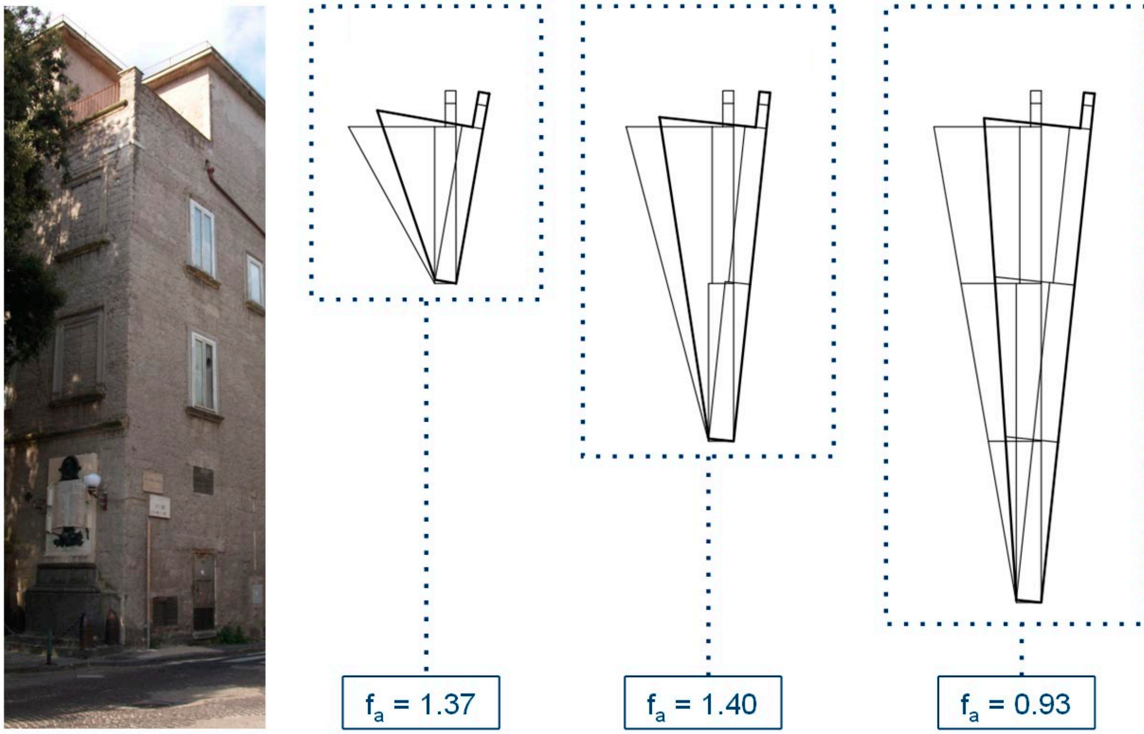

Fig. 6 – Partial collapse mechanisms for the South corner.

5 Verification level 3: global analysis

The highest level of complexity in the seismic safety evaluation is based on a Finite Element analysis of the entire building, through which a detailed description of the structure is performed and suitable formulations are used for the elements, in line with the peculiarities of masonry buildings [6]. For the present study case, a software system developed for the specific problem of safety verification in masonry buildings has been used [7].

Through a suitable interpretation of the geometric peculiarities of such structural systems, “3-Muri” automatically develops an equivalent frame model for the analysis of the seismic response. Specifically, on the basis of the arrangement of wall openings, three different situations are recognized for structural elements:

- vertical elements, as portions of cantilever walls, typically subject to axial, bending and shear actions;
- horizontal elements, corresponding to spandrel beams running at floor levels between upper and lower openings, subject not only to bending and shear actions, but to axial loads as well; they need to be compressed, indeed, in order to favour coupling of vertical cantilever walls;
- panel node elements, resulting from the intersection of the above vertical and horizontal elements.

The stiffness formulation includes both bending and shear contributions. Resistance limit states correspond to the following collapse modalities: compression under the effects of bending and axial load, or diagonal shear, or sliding shear. For normally sized wall panels, diagonal shear is the most common modality. Analysis capabilities include, as the basic option, non linear static analysis, in line with Eurocode 8 methodology, which is close to the Italian Building Code requirements.

For the case study here investigated, which corresponds to the geometrical model shown in Fig. 7(a), the computational model in Fig. 7(b) has been obtained. Details relative to a specific wall are shown in Fig. 7(c), where the above three kinds of elements, as automatically generated by the code, can be recognized from the colour: orange for vertical elements, green for horizontal, and light blue for panel nodes.

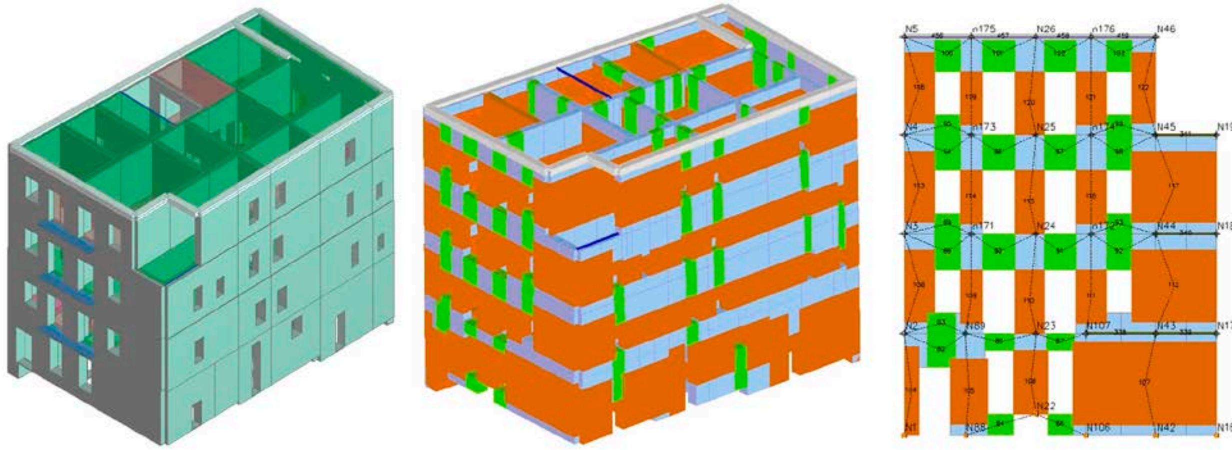

Fig. 7 – (a) Geometrical model (left), (b) computational model (center), (c) single wall model (right).

The computational procedure accounts for a variety of different situations, corresponding to: seismic motion direction (*x* or *y*) and sign (+ or -), inertia load distribution (proportional either to masses or to the first mode shape), mass eccentricity (no eccentricity or +/- a specified value). In total, 24 different situations, which are believed to include the meaningful condition for safety evaluation. It should be considered, however, that some assumptions, which may significantly influence the results, have by necessity an arbitrary nature; this applies to the selection of the *control node* to impose displacements and of the *performance point* for the representation of the capacity curve. A clear image of the result variability is given by the diagram in Fig. 8, where the seismic safety coefficient for each of the analyzed situations is shown, 24 values in total; they range from 0.653 to 1.100. Such coefficients correspond to the definition given in (5).

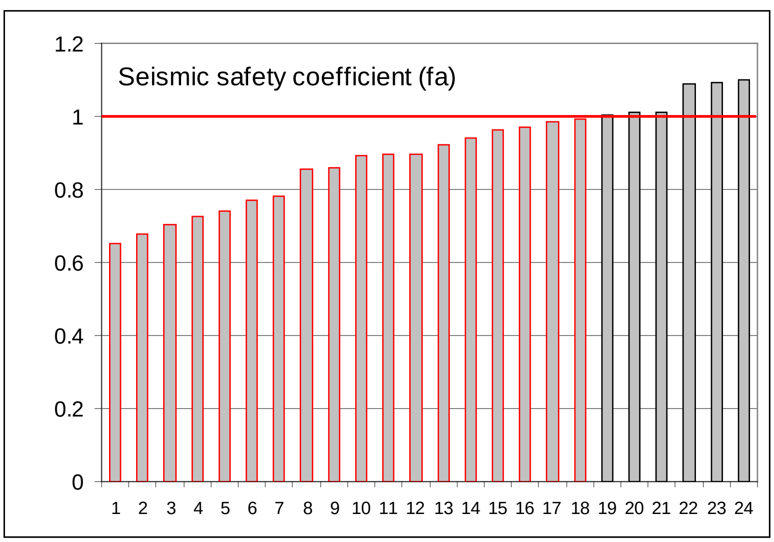

Fig. 8 – Seismic safety coefficients (f_a) for the analyzed situations.

Interesting considerations about the seismic performance of the analyzed building come from the application of the non linear computational procedure, which provides information both on the global system resistance and on the stress condition of single structural elements as well. In the following, results corresponding to the case of zero mass eccentricity are given.

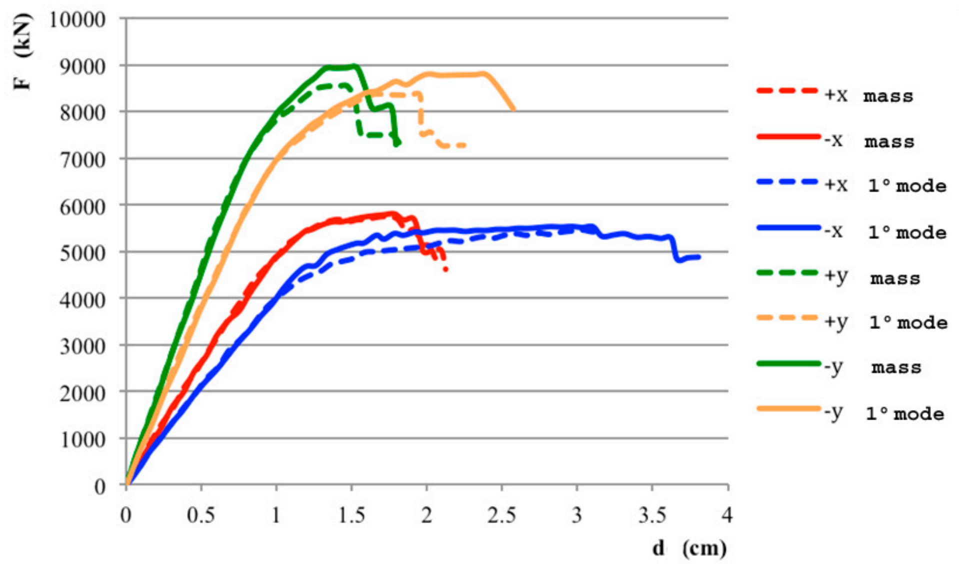

Fig. 9 – Capacity curves.

Capacity curves for the global structural system are shown in Fig. 9 for *+/- x* and *+/- y* directions of the seismic motion and for the two considered load distributions, first mode or mass proportional. As also indicated by level-1 analysis, a lower resistance is developed in the *x* (i.e., short) building direction, which is however associated to meaningful ductility resources. At a local scale, interesting results are relative to the most stressed wall, which can be recognized from the deformed plan view of the building (Fig. 10-left); the corresponding collapse modality is characterized by diagonal cracking at the first level (Fig. 10-center). This ultimate condition is due to the mass proportional load distribution and is associated to limited ductility resources. Under the effect of the alternative load distribution, collapse in compression due to bending and axial load is found, associated to a favorable ductile behavior. Results of this kind provide useful information about specific strengthening interventions for the improvement of seismic performance.

The final seismic safety verification can be easily done on the basis of the capacity curves through the well known procedure proposed by Fajifar [8] and adopted by Eurocode 8. A very similar approach can be found in the Italian Building Code, where differences are present in the criterion for the definition of the equivalent bi-linear capacity curve. In the following, reference is done to such code.

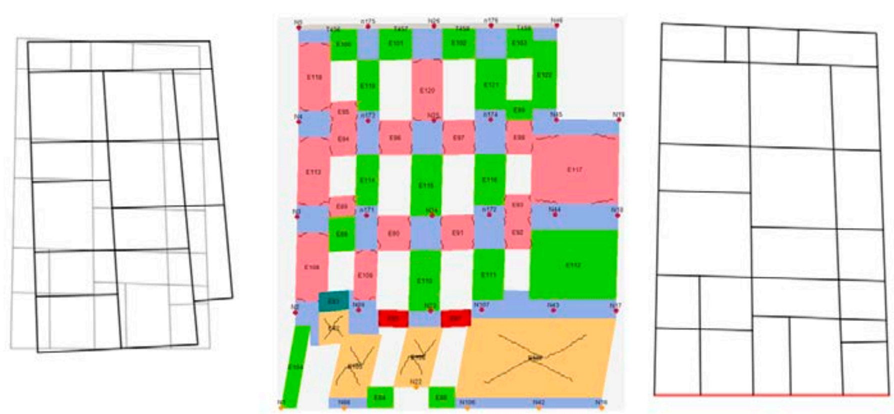

Fig. 10 – Plan rotation (left), most stressed wall (center), wall identification (right).

Capacity curves for the multi-degree-of-freedom system and for the equivalent single-degree-of-freedom system are shown in Fig. 11-left for the most unfavorable situation. Verification in terms of comparison between the bi-linear capacity curve and the response spectrum in the displacement-acceleration diagram is represented in

Fig. 11-right. From this, a value of 2.2 can be found for the behavior factor (q*), which corresponds to a required ductility ratio of 3.13, whereas a value of 1.77 is available. Correspondingly, the seismic safety coefficient expressed in terms of ratio between peak ground accelerations (demand / capacity, as defined in (5)) has a value of 0.653.

A few comments are worth making on the above results.

The minimum value obtained for the seismic safety coefficient (0.653) corresponds to the mass proportional load distribution; if the alternative load distribution is considered, i.e., the one proportional to the first mode shape, much higher ductility resources are present. This allows to conclude that the value adopted for the seismic safety coefficient corresponds to a conservative choice.

Both level-3 and level-1 safety verifications make reference to the global system response to seismic actions; the corresponding results can therefore be compared. Level-1 verification was based on a conventional reduction of the building shear capacity due to negative considerations on the real effectiveness of floor beams and on the uniformity of the wall cross sections. On the other hand, the value adopted for the behavior factor (2.5) was larger than the value coming from the static non linear analysis (2.2). By the end, level-1 and level-3 procedures provide similar values for the seismic safety coefficient, but the value coming from level-1 procedure is affected by conventional assumptions.

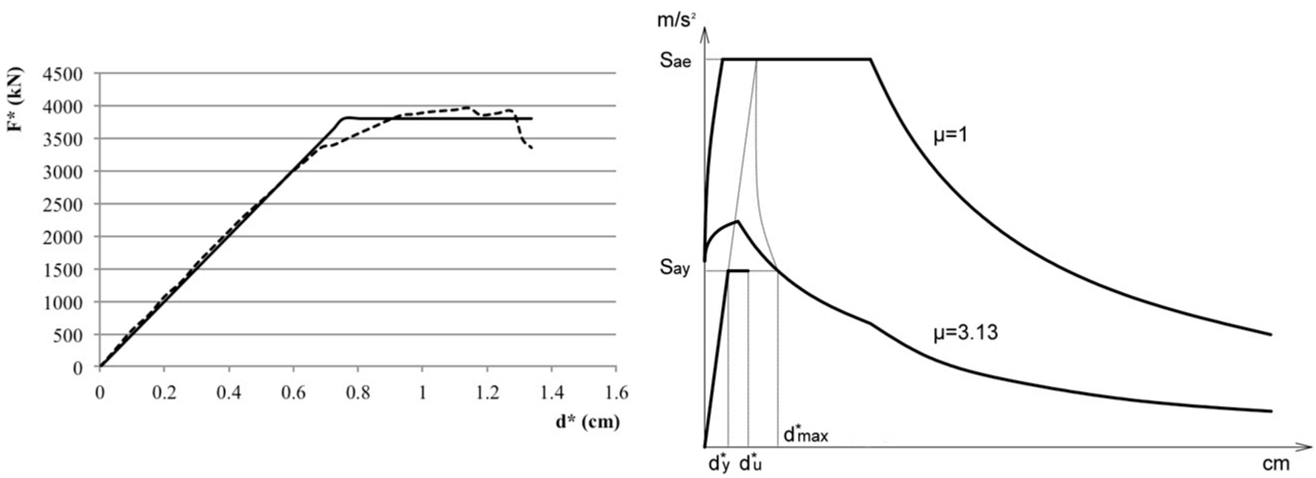

Fig. 11 – Left: capacity curves for the original system (dashed curve) and for the equivalent system (solid curve). Right: capacity and demand curves in the displacement-acceleration representation.

6. Conclusions

Following the prescriptions contained in the Italian Guide Lines for the seismic safety verification of buildings belonging to the monumental heritage, the three-level verification procedure has been applied to a historic masonry building. Correspondingly, three different values have been obtained for the seismic safety coefficient expressed with reference to the peak ground acceleration as capacity / demand ratio and are recalled in Table 2.

Table 2 – Safety factor values for the different verification levels.

Verification level	Seismic safety factor (f_a)	
Level-1: simplified evaluation of the building shear resistance	X-dir: **0.564**	Y-dir: **0.898**
Level-2: partial collapse mechanism analysis	**0.930**	
Level-3: detailed static non linear Finite Element analysis	X-dir: **0.653**	Y-dir: **0.897**

All of the three coefficients are lower than 1; high seismic vulnerability, therefore, is clearly highlighted for the examined case. A few comments can be done on this situation.

The safety factor corresponding to level-2 analysis presents a larger value than level-1 and level-3 factors; this means that the global building response is not compromised by local collapses. In other words, the box-type behavior of the masonry building is possible and the safety estimations corresponding to level-1 and level-3 need not to undergo restrictions.

The more sophisticated level-3 analysis confirms the collapse modality indicated by level-1 analysis: the building weak direction is the short one (x-axis), while the weak inter-story corresponds to the first one. It has also to be considered that similar values are obtained for the corresponding safety factors, with a lower value provided by level-1 analysis; a more conservative result, indeed, is expected from a simplified approach.

Results from the static non linear analysis look reliable, although based on some conventional assumptions; a large number of interesting detailed indications about the behavior of single structural elements are provided, from which suggestions on specific strengthening interventions can be easily obtained.

The real meaning of the simplified level-1 approach comes out clearly from the comparison with level-3 analyses: it provides a simple and fast way to get a global estimation of the building earthquake resistance, which might be useful in survey activities extended to a large number of buildings to the purpose of highlighting situations characterized by a pronounced seismic risk.

7. Acknowledgements

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