Computers and Structures 180 (2017) 13–26

Contents lists available at ScienceDirect

Computers and Structures

journal homepage: www.elsevier.com/locate/compstruc

Simplified and refined methods for seismic vulnerability assessment and retrofitting of an Italian cultural heritage masonry building

A. Formisano [a,*], A. Marzo [b]

[a] Department of Structures for Engineering and Architecture, University of Naples "Federico II", Naples, Italy
[b] Technical Unit of Seismic Engineering, ENEA Centre, Bologna, Italy

ARTICLE INFO

Article history:
Accepted 18 July 2016
Available online 10 August 2016

Keywords:
Emilia-Romagna earthquake
Cultural heritage building
Usability check
Seismic vulnerability
Italian Guidelines on Cultural Heritage
Repairing interventions

ABSTRACT

In the paper the usability check and the seismic vulnerability appraisement and repairing of a masonry building with cultural and artistic value located in the Municipality of Cento (Ferrara, Italy) after the last Italian seismic events (Emilia-Romagna, 2012 May 20th and 29th), are reported and discussed.

After some indications on the mentioned earthquakes are given, the case study has been presented and useful information on both historical news and geometrical properties have been given. So, the usability check of examined building has been done through the identification of damages occurred under earthquake.

Later on, seismic vulnerability assessment of the building on the basis of both the simplified LV1 and the refined LV3 analysis levels given by Italian Guidelines on Cultural Heritage has been performed. The analysis results, compared each other in terms of seismic safety factors, have shown that building has an average vulnerability degree, so to require local repairing and strengthening interventions, which have been herein presented. Finally, the effectiveness of the proposed local interventions has been proved by numerical analysis in the non-linear field.

1. The 2012 Emilia-Romagna earthquake

In May 2012, two major earthquakes occurred in the Emilia-Romagna Italian region.

The first earthquake, having magnitude 5.9, happened on 2012 May 20th at 04:03 local time with epicentre between Finale Emilia and San Felice sul Panaro. Two aftershocks of magnitude 5.2 followed and seven people were killed. The second earthquake occurred on 2012 May 29th at 9:00 with a magnitude of 5.8 and produced twenty deaths and wide damages, particularly to buildings already weakened by the first seismic event. The epicentre was in Medolla at a depth of about 10 km from the ground, where the fault rupture was observed.

However, a very huge seismic sequence happened, especially within the districts of Ferrara and Modena, before and after the aforementioned earthquakes [1].

Immediately after these earthquakes, usability checks on public and private buildings, made of masonry, steel and r.c. (casted and prefabricated) structures, were quickly performed by members of the Italian DPC-ReLUIS research project, analogously to the activity carried out in L'Aquila and its neighbourhoods [2–4]. The aim of this activity was to know both the conditions of constructions and damages they suffered in order to evaluate if they were able to withstand or not a further earthquake with the same features of the occurred one. Results of this activity are reported in detail into specific papers and reports [5–18] investigating the behaviour of masonry buildings, that can be either isolated or clustered, the latter representing the largest part of Italian and some European historical centres, which the attention of different researchers is focused on [19–24].

In the current paper, which is an updated version of [25], the usability check and the related seismic vulnerability assessment and repairing, according to the Italian Guidelines on Cultural Heritage's analysis levels, of a cultural heritage building located in Cento, a city in the district of Ferrara, is illustrated in detail.

2. The case study

2.1. Historical news and geometrical features

The case study is an isolated masonry palace with an internal oratory located in Cento, a city in the district of Ferrara, one km far from it historical centre.

* Corresponding author.
E-mail address: antoform@unina.it (A. Formisano).

http://dx.doi.org/10.1016/j.compstruc.2016.07.005
0045-7949/

According to the sure first news about the building, it was erected around 1578, year when the pastoral visit of the Cardinal Paleotti was performed. The confraternity of St. Bartolomeo kept there the relics of St. Zenone up to 1587. During XVIII century the building was subjected to different property transfers, whereas in XIX century it became a subsidiary building of the church of St. Maria of Penzale.

Nowadays the building, which had in the past the role of public oratory, has a configuration similar to the 1760 dated one, when it was radically transformed from its original layout.

The oratory is included into a larger construction, having irregular L-shaped plan scheme with area of about 450 m^2, and developing on two levels with average height of 3 m, which delimitate the same oratory on three sides (Fig. 1). Initially, the building, composed of two heads brick walls and covered by a timber pitched roof, hosted a religious school, which underwent some internal modification in 1960, while, at the moment, it is used for residential purpose. Some external views of the building are reported in Fig. 2.

The main facade is divided into three parts by columns: in the central zone there are a door and a window, both of them framed by a stucco configuration, whereas the lateral zones, with lower height, are connected to the central part by means of two spirals (Fig. 3a). The facade is completed in the upper part by a cornice surmounted by a tympanum with segmental arches; on the roof a small bell tower is located.

The oratory has a rectangular plan covered by a barrel vault in the entrance, a vault in the central zone and a hemispherical dome with square layout in the presbytery. The central zone is characterised by four Corinthian columns which sustain a wide frame bracket.

Lateral walls are decorated with 18th century paintings attributed to Stefano Ficatelli (Fig. 3b), whereas the altar is adorned with the representations of Santa Liberata and San Rocco (Fig. 3c), made by an unknown author of the same century previously mentioned.

Actually the building complex, thanks to its significant historical-artistic features, is covered by the bond of the Superintendence of Cultural Heritage of the province of Ferrara, which consider it as a particularly interesting construction according to the Italian Law n. 1089 promulgated on 1939 June 1st [26].

2.2. Earthquake damages and usability check

The Emilia-Romagna seismic events were characterised by both horizontal waves, which produced rotation and translation of the building due to its irregular plan shape, and vertical waves, which affected flexible timber floors and masonry piers and spandrels.

The seismic wave movements led to:

(1) Detachment of plaster between walls and floors due to the poor connection degree between timber beams and masonry walls (Fig. 4a);
(2) Detachment of plaster among orthogonal walls, caused by the reduced degree of junction between the walls themselves (Fig. 4b);
(3) Cracks in the floor ceilings (Fig. 4c);
(4) Cracks in vaults and arches (Fig. 5a);
(5) Lesions in the church walls caused by their overturning mechanism (Fig. 5a);
(6) Diagonal cracks into masonry walls (Fig. 5b);
(7) Lesions in masonry piers above openings due to lack of effective lintels (Fig. 6a);
(8) Vertical cracks on walls due to the load transferred by timber beams to the same walls (Fig. 6b);
(9) Collapse of ceilings caused by excessive deformation of timber beams which, consequently, lost their support (Fig. 6c).

The usability check performed by the first Author on the inspected building after the earthquake has indicated that the major damages occurred in the oratory, where the initial

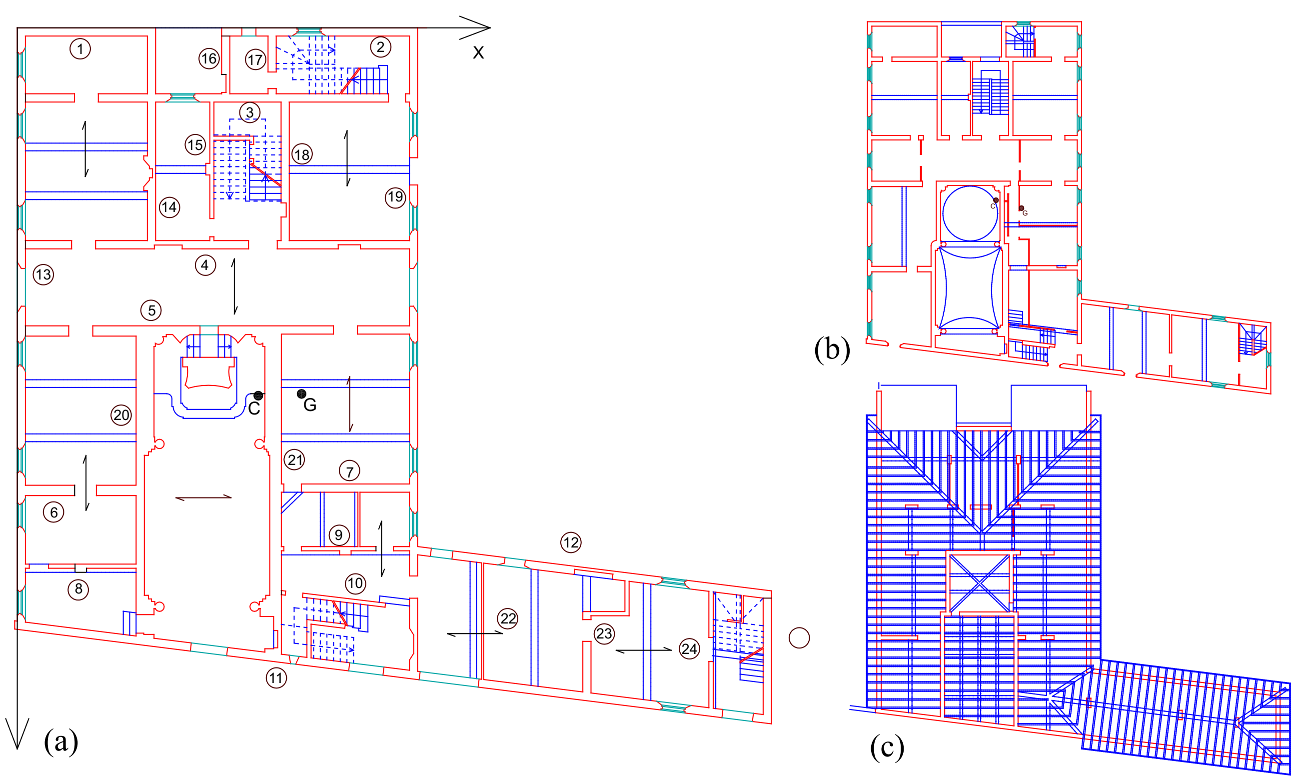


Fig. 1. Ground (a), first (b) and roof (c) floors of the investigated masonry building.

Fig. 2. Views of the building facades related to its residential part.

(a) (b) (c)

Fig. 3. Main facade (a) of the building, lateral painting (b) and altar (c) of the oratory.

(a) (b) (c)

Fig. 4. Cracks between floor and wall (a), between orthogonal walls (b) and in the ceiling (c).

detachment of orthogonal walls, representative of a first mode collapse mechanism, and large cracks within arches and vaults, makes it completely unusable.

On the other hand, the remaining part of the building complex shows diffused but not heavy damages, such as diagonal and vertical cracks, the latter due to masonry crushing, light overturning mechanisms of walls, horizontal cracks among walls and floors and cracks into partitions, which produce a light dangerous situation to be eliminated with both prompt intervention activities (propping of floors) and some local intervention measures (tying among walls, repairing of cracks, new wall-floor connections) aiming at eliminating the construction seismic deficiencies.

Therefore, on the basis of the detected damages and the consequent post-seismic survey activity, the building was declared partially unusable, that is *grade C* according to the AEDES form [27].

A detailed indication of the crack pattern detected in the building is depicted in Fig. 7.

Fig. 5. Cracks into arch and vault (a) and pier diagonal cracks (b) in the oratory.

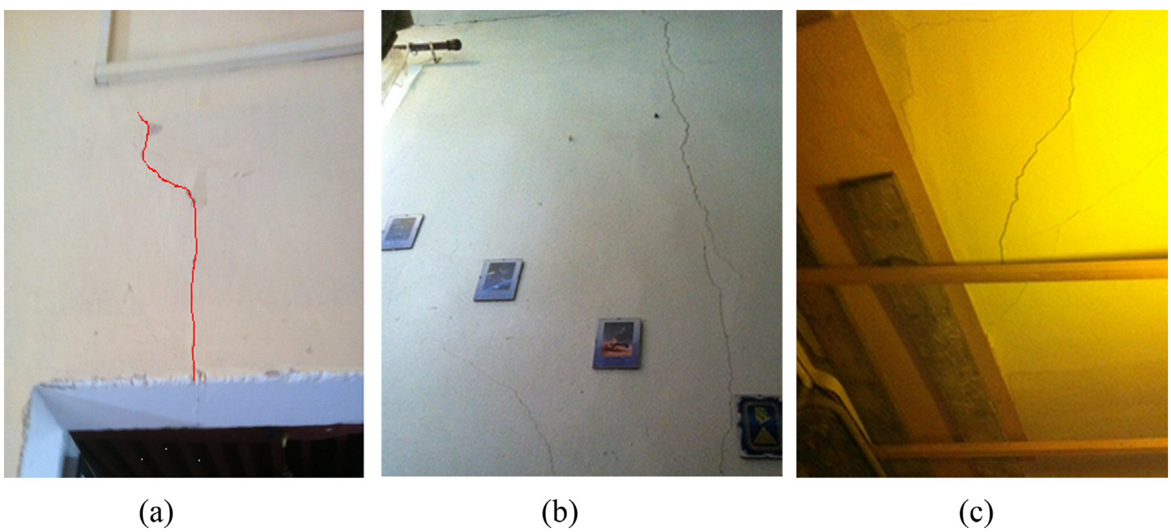


Fig. 6. Crack above door (a), vertical cracks into a wall (b) and propping of a damaged ceiling (c).

3. Seismic vulnerability assessment

The seismic vulnerability assessment of the building has been made by the analysis level LV1 proposed by the DPCM 09/02/11 [28] in the par. 5.4.2 "Palaces and villas and other structures with intermediate walls and horizontal floors".

The main hypothesis of this analysis level is that the achievement of Life Safety Limit State occurs with the global behaviour of the structure, when collapse of walls within their plane appears.

The investigated building is behaved in a global way, even if some local mechanisms not producing any wall collapse have been recorded. In addition, its seismic behaviour is conditioned by the interaction of the oratory with the remaining part of the building, which give rise to the so-called building aggregates, whose response towards earthquakes is not very simple to be understood, so that wide studies have been developed on this topic [19–24,29–33].

Therefore, with reference to the condition attaining the cited ultimate limit state, the simplified mechanical model foresees the collapse acceleration of the elastic response spectrum on the basis of the following equation:

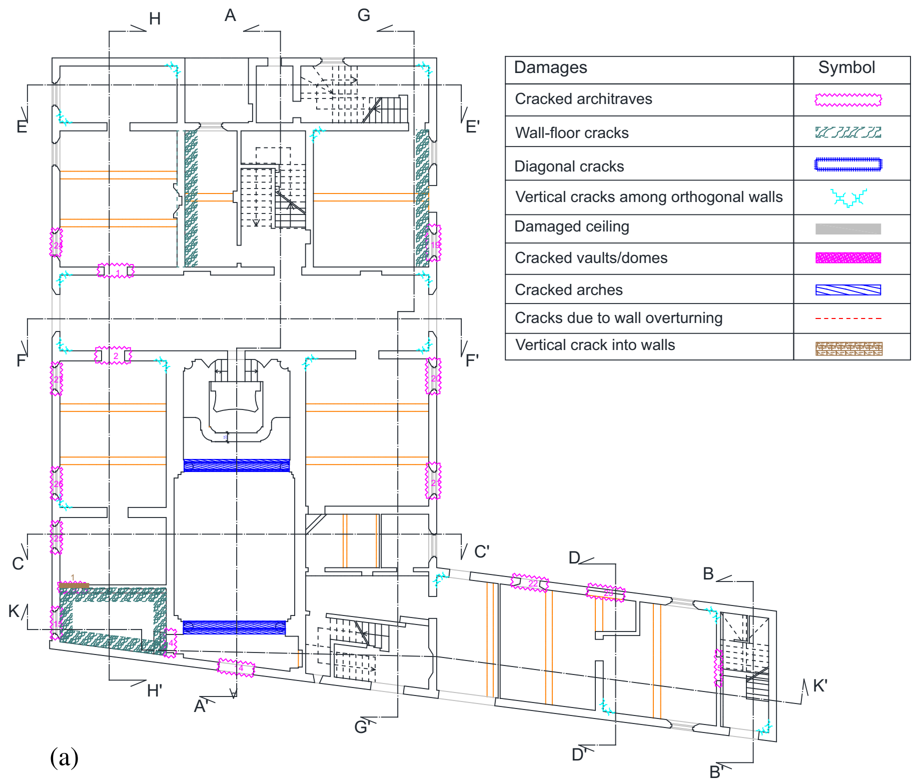
$$S_{e_{SLV}} = \frac{q \cdot F_{SLV}}{e^{*} \cdot M} \tag{1}$$

where

- F_{SLV} is the building shear resistance;
- q is the behaviour factor, taken between 3.0 and 3.6 for buildings regular in elevation with number of levels equal or greater than two [34]. In this case, in order to stay on the safe side, a q-factor equal to 3 has been used.
- m is the total seismic mass;
- e^{*} is the participating mass ratio mass related to the first vibration mode.

The building shear resistance is considered as the lesser of those valued along main directions of load-bearing walls. For each direction considered, the analysis method hypothesizes that collapse occurs into masonry piers when the average shear stress reaches a given shear strength of masonry:

$$F_{SLV,xi} = \frac{\mu_{x,i} \cdot \xi_{x,i} \cdot \zeta_{x,i} \cdot A_{x,i} \cdot \tau_{di}}{\beta_{x,i} \cdot k_i} \qquad F_{SLV,yi} = \frac{\mu_{y,i} \cdot \xi_{y,i} \cdot \zeta_{y,i} \cdot A_{y,i} \cdot \tau_{di}}{\beta_{y,i} \cdot k_i} \tag{2}$$



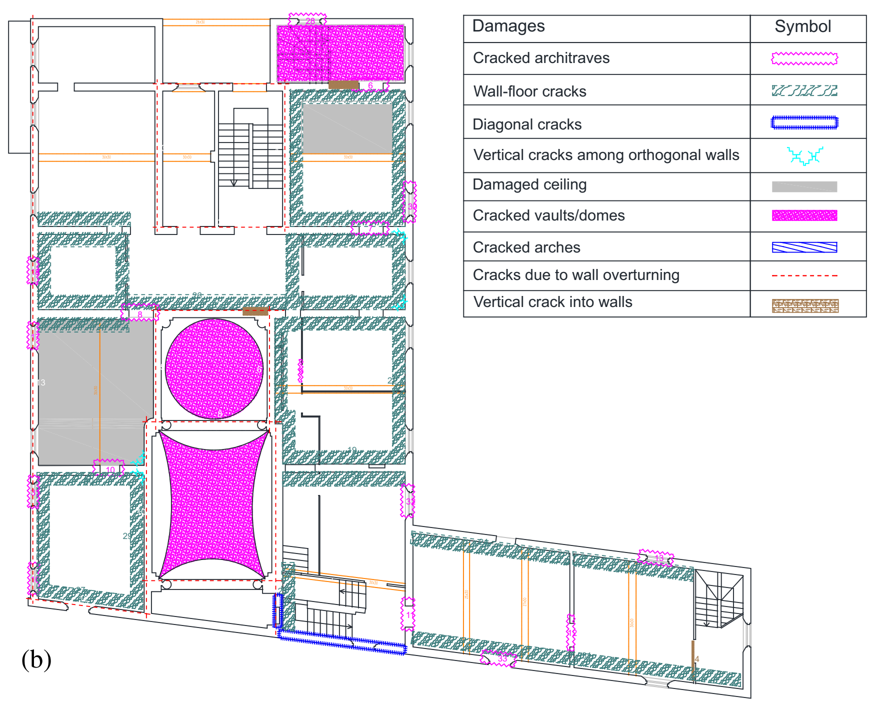


Fig. 7. Damage patterns at the ground (a) and the first (b) floors.

where

- A_{xi} and A_{yi} are shear resistant areas of the i-th floor walls located in x and y directions, respectively;
- β_{xi} and β_{yi} are plan irregularity factors related to the i-th floor which, in this case, have been determined for each level after that both the building centre of mass and centre of stiffness have been assessed with reference to the numbering of walls and axis orientation illustrated in Fig. 1. Such factors assume values of 1.01 and 1.25 in directions x and y, respectively, at the ground level, and 1.09 and 1.25 in directions x and y, respectively, at the 1st level.
- μ_{xi} and μ_{yi} are coefficients considering, at the i-th floor, the stiffness and strength homogeneity of masonry walls in directions x and y, respectively. They are expressed by the following relationships:

$$\mu_{xi} = 1 - 0.2 \cdot \sqrt{\frac{N_{mxi} \cdot \sum_j A^2_{xi,j}}{A^2_{xi}} - 1} \geqslant 0.8$$
$$\mu_{yi} = 1 - 0.2 \cdot \sqrt{\frac{N_{myi} \cdot \sum_j A^2_{yi,j}}{A^2_{yi}} - 1} \geqslant 0.8 \quad (3)$$

which always provide in the current case values equal to 0.8.

- ξ_{xi} and ξ_{yi} are coefficients associated to the main type of collapse mechanism expected in masonry walls of the *i*-th floor. They assume value of 1 in case of shear collapse and 0.8 in case of compression-bending moment collapse. In the current case, given the prevalence of stocky piers, unitary values have been recruited.
- ζ_{xi} and ζ_{yi} are coefficients related to the spandrel resistance of the *i*-th floor masonry walls arranged respectively in directions *x* and *y*: their values are 1.0 or 0.8 in case of strong spandrel and weak spandrel, respectively. In the examined case, considering cracks into piers, the value of 0.8 has been taken into account.
- τ_{di} is the design value of the masonry piers shear strength at the *i*-th floor, defined as (formula 5.4 of Italian M.D. 14/01/08):

$$\tau_{di} = \tau_{0d} \cdot \sqrt{1 + \frac{\sigma_{0i}}{1.5 \cdot \tau_{0d}}} \quad (4)$$

where τ_{0d} is the design shear strength of masonry (assessed taking into account the confidence factor, assumed equal to 1.35) and σ_{0i} is the average normal stress on walls at the i-th floor. For the examined building, τ_{di} assumes values of 0.071 and 0.072 N/mm^2 in directions *x* and *y*, respectively, at the ground level and 0.077 and 0.081 N/mm^2 in directions *x* and *y*, respectively, at the 1st level.

First, according to the axis reference system depicted in Fig. 1a, the building centroid (x_g = 11.10 m and y_g = 9.41 m at the ground floor and x_g = 11.24 m and y_g = 13.55 m at the first floor) and centre of stiffness (x_c = 9.41 m and y_c = 14.33 m at the ground floor and x_c = 9.39 m and y_c = 13.00 m at the first floor) have been evaluated.

Afterwards, the shear resistance of the building at the ground and first storeys along two analysis directions have been calculated as follows:

– Ground story

$$F_{SLV,gx} = \frac{0.8 \cdot 1 \cdot 0.8 \cdot 40.476 \cdot 0.071 \cdot 10^6}{1.01 \cdot 0.5} = 3642 \text{ kN} \quad (5)$$

$$F_{SLV,gy} = \frac{0.8 \cdot 1 \cdot 0.8 \cdot 40.452 \cdot 0.072 \cdot 10^6}{1.25 \cdot 0.5} = 2989 \text{ kN} \quad (6)$$

– First story

$$F_{SLV,lx} = \frac{0.8 \cdot 1 \cdot 0.8 \cdot 38.831 \cdot 0.077 \cdot 10^6}{1.093 \cdot 1.00} = 1751 \text{ kN} \quad (7)$$

$$F_{SLV,ly} = \frac{0.8 \cdot 1 \cdot 0.8 \cdot 34.7184 \cdot 0.081 \cdot 10^6}{1.25 \cdot 1.00} = 1440 \text{ kN} \quad (8)$$

Therefore, accelerations correspondent to the above shear resistances have been estimated:

– Ground storey

$$Se_{,SLV,x} = \frac{3 \cdot 3642}{0.90 \cdot 668.5} = 18.2 \frac{\text{m}}{\text{s}^2} \quad (9)$$

$$Se_{,SLV,y} = \frac{3 \cdot 2989}{0.90 \cdot 684} = 14.6 \frac{\text{m}}{\text{s}^2} \quad (10)$$

– First storey

$$Se_{,SLV,x} = \frac{3 \cdot 1751}{0.90 \cdot 914.5} = 6.4 \frac{\text{m}}{\text{s}^2} \quad (11)$$

$$Se_{,SLV,y} = \frac{3 \cdot 1440}{0.90 \cdot 1031.1} = 4.6 \frac{\text{m}}{\text{s}^2} \quad (12)$$

The collapse acceleration is consequently the one correlated to the first story shear strength in direction *y*, since it assumes the lower value among all others.

The vibration period of the building, calculated according to NTC08 provisions, is 0.285 s, it being comprised within the range [T_B, T_C] of the examined site.

The acceleration on rigid ground (type *A*) a_{SLV}, allowing the attainment of the Life Safety Limit State, can be calculated as follows:

$$a_{SLV} = \frac{S_{e,SLV}(T_1)}{SF_0} = \frac{4.6}{1.00 \cdot 2.59 \cdot 9.81} = 0.18 \text{ g} \quad (13)$$

Finally, the acceleration factor ($f_{a,SLV}$), defined as the ratio between the rigid ground acceleration a_{SLV} and that corresponding to the reference return period ($a_{g,SLV}$ = 0.407 g), the latter also referring to the subsoil *A*, is considered as a seismic safety factor and is calculated as follows:

$$f_{a,SLV} = \frac{0.18 \text{ g}}{0.407 \text{ g}} = 0.44 \quad (14)$$

According to the provisions of the Italian Ordinance n. 51 [35], considering both the significant damages detected and the value of the appraised acceleration factor $f_{a,SLV}$ comprised between 0.3 and 0.5, the examined building shows an average vulnerability level. This allows to intervene on the building with local repairing and strengthening interventions, which will be illustrated in Section 6.

4. Numerical vulnerability assessment

The numerical seismic vulnerability evaluation of the case study has been carried out also through the LV3 assessment level proposed in the DPCM 09/02/11 by performing non-linear static analyses with the TREMURI computer program [36,37]. The three-dimensional model of the building is based on the identification of an equivalent frame consisting in vertical (piers) and horizontal (spandrels) macroelements. The intersection areas between horizontal and vertical elements are modeled as rigid nodes. The nonlinear behaviour of masonry piers is assumed as elastic-perfectly plastic with initial cracked elastic stiffness; the strength criteria depend on the possible failure modes, i.e.: flexure-rocking, sliding shear and shear-diagonal cracking. The formulation is consistent with the recommendations included in several seismic codes [34,38,39], since strength criteria defined for both bending and shear failure modes can be easily implemented and adopted to define the lateral strength of the different structural elements.

Relatively to the plastic branch, the effects of cyclic actions are taken in account through the degradation of the stiffness, while the ultimate limit state in terms of displacement is based on the failure of the generic panel through the maximum drift (d_u), which depends on the prevailing failure mode occurred in the panel. For existing buildings, the Life Safety Limit State values of the ultimate drift are assumed to be 0.6% and 0.4% of the inter-story height, corresponding to the bending and shear failure modes, respectively.

Regarding the floor elements, the computer program allows to take into account the deformability in their plane through membrane finite elements with equivalent stiffness properties. Two types of floors are placed within the building: vaults and floors with timber beams and planks. For the former type, an equivalent horizontal stiffness, on the basis of the geometrical configuration, the thickness, the material features and the connection system to walls, has been defined.

The TREMURI model of the palace is depicted in Fig. 8.

From modal analysis it was found that the first vibration period is in the direction y and is equal to 0.243 s, a value very close to that achieved from code indications (0.285 s) and used for LV1 analysis level. In Fig. 9 the plan deformed shape referred to this mode is illustrated, it involving especially the "short leg" of the palace, which, as expected, appears to be the most deformable construction part.

The pushover analysis was conducted considering two systems of horizontal forces applied at the level of floors and acted in the two orthogonal directions coinciding with the principal axes of the building:

(a) a system of forces proportional to masses;
(b) a system of forces proportional to the first vibration mode.

Such systems of static forces were applied according to 24 different possible load conditions to take account of the variability of the verses and of the accidental eccentricities of the mass centre. Table 1 shows the results related to the worst load conditions for the x and y directions. The reference parameters for the verification are the capacity and demand displacements, respectively d_u and d_{max}, while q^* is the ratio between the system shear force, supposed indefinitely elastic, and the yielding strength of the equivalent nonlinear system (with the limitation $q^* < 3$); the parameter α_u has the same meaning of $f_{a,SLV}$ (see Eq. (14)), it being the ratio between capacity/demand in terms of Peak Ground Acceleration (PGA).

The results of LV3 analysis confirm that y is the weak direction, also when the nonlinear behaviour and the displacement capacity are taken into account. In fact, the safety factor is less than one, as well as the coefficient q^* is greater than the allowable limit (3). Also in the direction x the seismic check is not satisfied, with a safety factor slightly greater than the direction y one. As in the LV1 analysis level, also with the refined LV3 assessment level, the soft-story mechanism at the first level, where flexural failure is prevailing for the masonry piers, occurred (Fig. 10).

Under the qualitative point of view, the walls damaged by the earthquake in the numerical model show substantially the same crack types detected after the seismic event (Fig. 11). For the sake of example, if the comparison between Figs. 10 and 11 is done, it is noticed that some damaged spandrels, identified with letters a, b and c in Fig. 10, have flexural failure modes as those identified on the building facades.

5. Comparison of results provided by LV1 and LV3 analysis levels

The results of the seismic analysis levels LV1 and LV3 are compared and summarised in Table 2, with reference to the weak direction y.

Regarding the capacity/demand relationship in terms of PGA corresponding to the achievement of the Life Safety Limit State, it is noticed that the value of this parameter (f_a) calculated by the LV1 analysis level is less than that (α_u) obtained from LV3 one. Therefore, the approximated analysis level is too on the safe side. This is due to the high value of e^* (see Eq. (1)) used for computing the building acceleration capacity $S_{e,SLV}$, which is right for regular building, but it is not appropriate for not regular constructions as the examined one. As a consequence, the equation to calculate the e^* factor proposed by the code should be modified when not regular buildings are considered.

Also the base shear according to LV1 is lower (of about 12%) than that calculated with the LV3 global analysis level. Therefore, the LV1 analysis level appears to be more conservative than the other level LV3, because it assumes substantial simplifications for describing the structural behaviour: the seismic capacity of the building, in fact, is measured in terms of forces rather than displacements, so that the strongly nonlinear behaviour of the structure is not properly considered. Actually, the same order of magnitude of the two parameters is mainly due to the presence of flexible diaphragms in the palace, which strongly reduce its seismic response with respect to the real stiffness conditions given by floors considered in the LV3 analysis. In addition, it must be noted that the safety parameter is more meaningful in terms of risk classification than in terms of structural response characterisation.

About the comparison of failure modes in the direction y, LV3 provides a story mechanism at the first level (Fig. 10), which is the same collapse mechanism attained with the LV1 analysis level. With both analysis levels, the prevailing failure mode for the masonry piers in the direction y is of flexural type. This confirms the reliability of the simplified analysis level adopted from the Italian Guidelines on Cultural Heritage in predicting the seismic behaviour of historical and remarkable masonry constructions.

6. Local repairing interventions

In the following the local intervention techniques used for building complex reparation are described, illustrated and correlated to the damage patterns detected there after the earthquake.

6.1. Cracks in masonry piers

After the plaster is removed, the cracks found in the masonry walls are eliminated by replacing the damaged stones with new ones, which are connected by means of lime mortar to the existing masonry apparatus.

6.2. Cracks in masonry spandrels above doors

The cracks found above the openings into masonry spandrels are abolished by replacing cracked stones with new elements. In addition local reinforcement of the openings below the damaged spandrels is made by means of steel profiles. In particular,

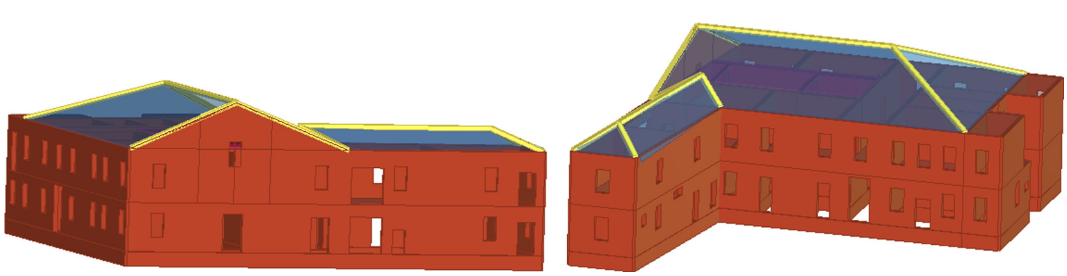


Fig. 8. The TREMURI model of the investigated building.

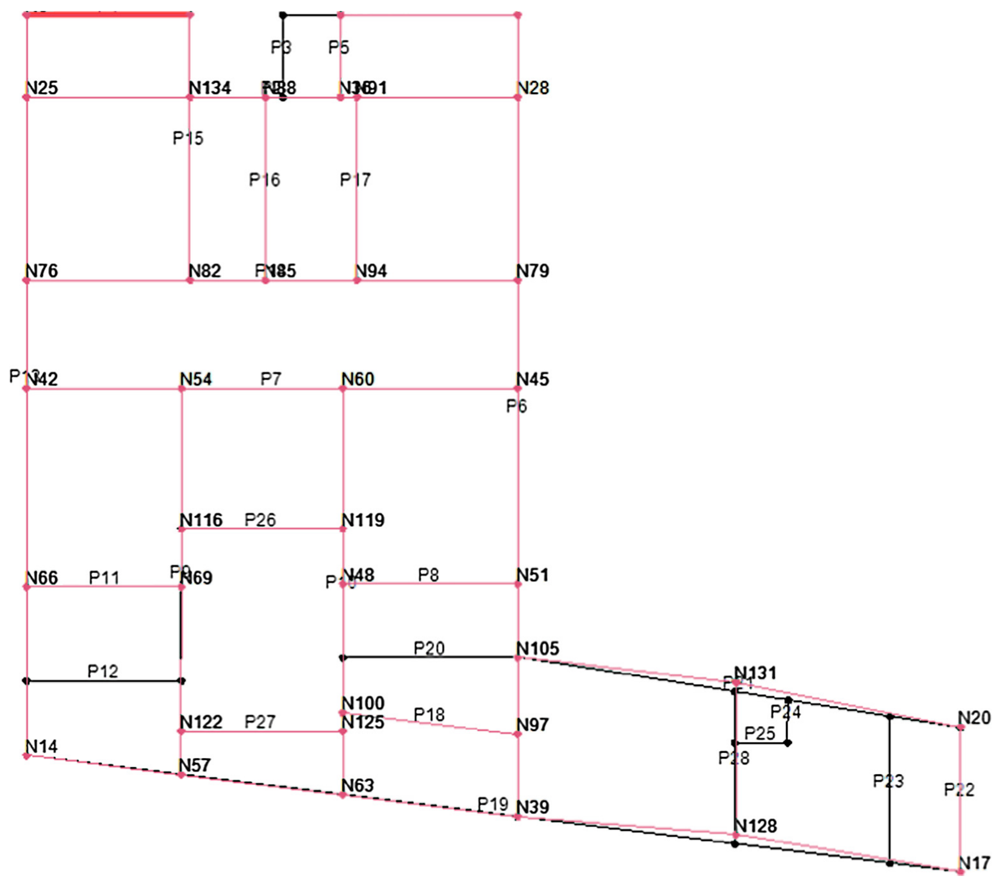


Fig. 9. The building 1st level deformed shape related to the 1st vibration mode.

Table 1
Results related to the worst load conditions deriving from LV3 analysis level.

Dir.	Load cond.	Ecc. (cm)	d_u (cm)	d_{max} (cm)	q^*	α_u
$-x$	1st mode	133.9	0.56	0.68	2.293	0.877
$+y$	1st mode	145.6	1.32	0.87	3.438	0.873

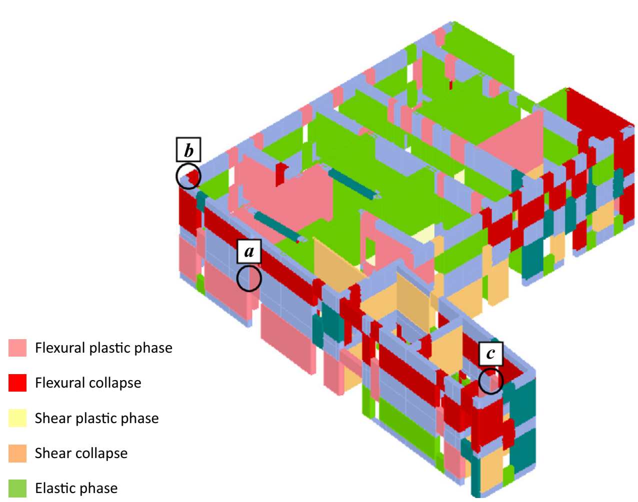


Fig. 10. Flexural failure mode at the first floor according to the LV3 analysis level.

UPN160 profiles and coupled angles 160 × 80 × 10 mm (Fig. 12) are used for 160 mm and 320 mm thick walls, respectively.

These profiles, supplied and put in place under form of columns and beam composing a frame, are connected to the lintels and masonry through M16 threaded steel bars with pitch of 300 mm. The bars are inserted into holes previously arranged in masonry lintels and walls and then sealed with epoxy resin.

6.3. Cracks in masonry spandrels above windows

The repairing of cracks in masonry lintels is achieved through the insertion of iron wedges into lesions, which are then closed with strongly adhesive lime mortar until rejection. This intervention requires in advance the careful skiving and cleaning of lesions, as well as the support of openings.

6.4. Local shear strengthening of walls and reinforcement of arches, vaults and domes

The local shear reinforcement of masonry walls is performed with glass fibre net and lime mortar (Fig. 13).

The intervention is divided into the following phases:

1. Demolition and removal of existing plaster and all inconsistent or incoherent parts.
2. Preparation of support for the application of composite materials (tissues), carried out by spreading polymeric bi-component fibre-reinforced structural mortar with low elastic modulus (i.e. structural lime and pozzolan mortar cement-free compatible with old walls) and thickness of 2 cm after the surface cleaning and dusting with scrub brush and/or vacuum cleaner; preparation of epoxy resin for adhesion improving between existing support and carried over mortar; rounding of any edges with a minimum radius of 1 cm.

Fig. 11. Some spandrels damaged after earthquake: agreement between real collapse mechanisms and numerical ones (see Fig. 10).

Table 2
Comparison between LV1 and LV3 analysis results.

	α_u	$f_{a,SLV}$	F_{SLVy} (kN)
LV1	–	0.44	1440
LV3	0.873	–	1631

3. Application through tackling or wrapping of bi-directional (mesh 12.5 × 12.5 mm) alkali-resistant glass fibre tissue for structural reinforcement. In the longitudinal joints the tissue must be overlapped by at least 25 cm, whereas the transverse joints should be overlapped by at least 10 cm.
4. Supply and application of aramid fibre connectors or similar devices for the improvement of the anchorage of glass fibre tissues, to be performed by the creation of 14–16 mm diameter holes in the masonry wall with length up to 500 mm. After insertion of connectors inside holes filled by resin, the anchor is created by means of the staple impregnation with epoxy adhesive on the reinforcing layer already positioned and application of a further reinforcement layer above the staple, having length equal to the same staple plus 10 cm.
5. Application of saturation epoxy adhesive on the fibre tissue.
6. Removing any excess resin.
7. Shedding quartz for bonding the finishing plaster layer.
8. Formation of the plaster so to ensure the filling of all cavities and the total incorporation of the reinforcing mesh.
9. Levelling and finishing of surfaces with a sponge towel, taking care of the wet curing of these surfaces at least for 24 h.
10. Completing the plaster by means of smoothing and painting.

As for shear reinforcement masonry walls, also arches, vaults and domes are locally strengthened by means of the application of alkali-resistant glass fibres.

6.5. Connection among walls

It involves the use of metal tie-beams made of 20 mm diameter steel bars anchored to the internal walls, by means of ribbed 200 × 200 × 10 mm steel plates, and to the external walls, by means of steel end pallets consisting of 300 × 50 × 15 mm rectangular plates (Fig. 14).

This intervention is based on the following operations:

– Creation of holes in the walls for the insertion of metal tie-beams;
– Connection of the tie-beams to the interior walls and exterior ones with plates positioned in pairs on both sides of each wall and end steel pallets, respectively;
– Connection among tie-beams by means of special steel turnbuckles.

6.6. Reconstruction of ceilings

The most damaged existing ceilings, made of "arelle" composed of timber bending and gypsum/lime plaster, are demolished and replaced with drywall ceilings composed of sheets secured by self-tapping screws to a structure made up of galvanised steel sections, having thickness of 6/10 mm and placed each to other at a distance of 600 mm.

6.7. Timber beam – masonry wall connections

In order to make the connection between timber floors and masonry walls, the main emerging floor beams are connected to the walls by iron blades having 80 × 5 mm angle cross-section (Fig. 15).

The blade is fixed to the beams with galvanised steel lag screws having a minimum length of 80 cm, and to the walls by means of M16 steel bars having length of 25 cm and put inside holes previously filled with epoxy resin.

The interventions listed before have been applied for seismic repairing of the study building, as illustrated in Fig. 16, where their placement into plan layouts of the examined construction is depicted.

In order to evaluate the benefits provided to the examined construction by the applied interventions, pushover analyses through the TREMURI program have been carried out on the repaired building (Table 3).

About the interventions on the building numerical model, the reinforcement of walls, arches, vaults and domes has been done by improving the mechanical properties of masonry by an increasing coefficient of 1.5, as stated by the explicative circular of the Italian seismic code for constructions [40]. On the other hand, steel tie-beams have been automatically considered in the program by associating their presence to the corresponding walls interested by this intervention.

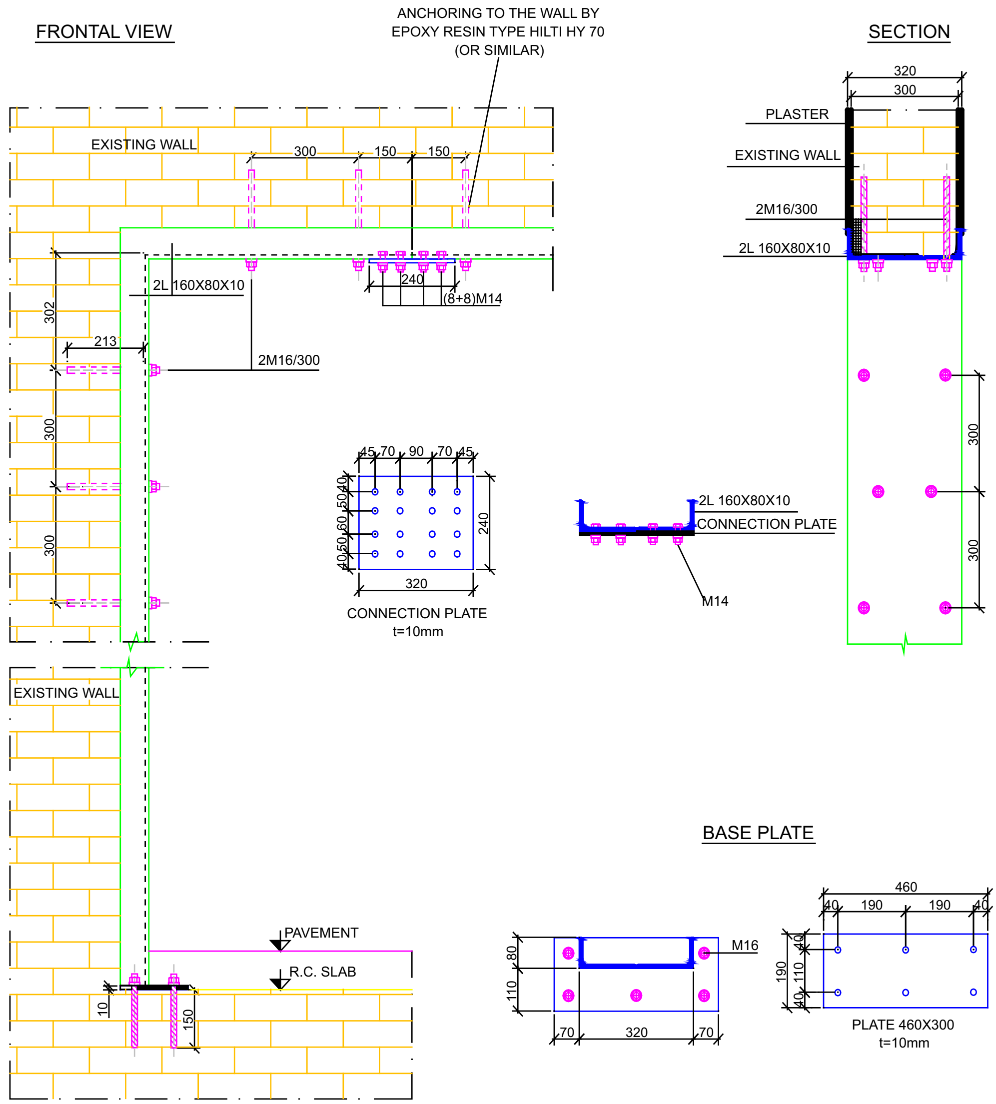


Fig. 12. Details of the opening confinement.

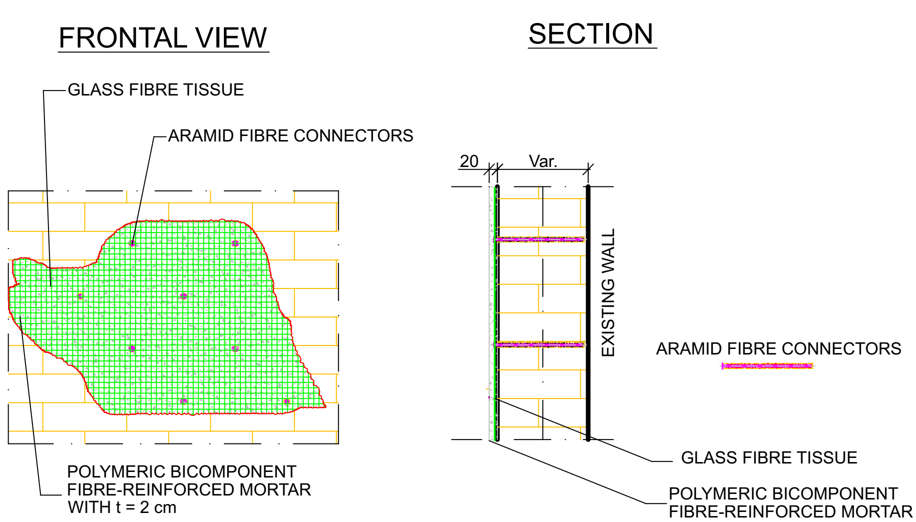


Fig. 13. Reinforcement of masonry wall by glass fibre tissues.

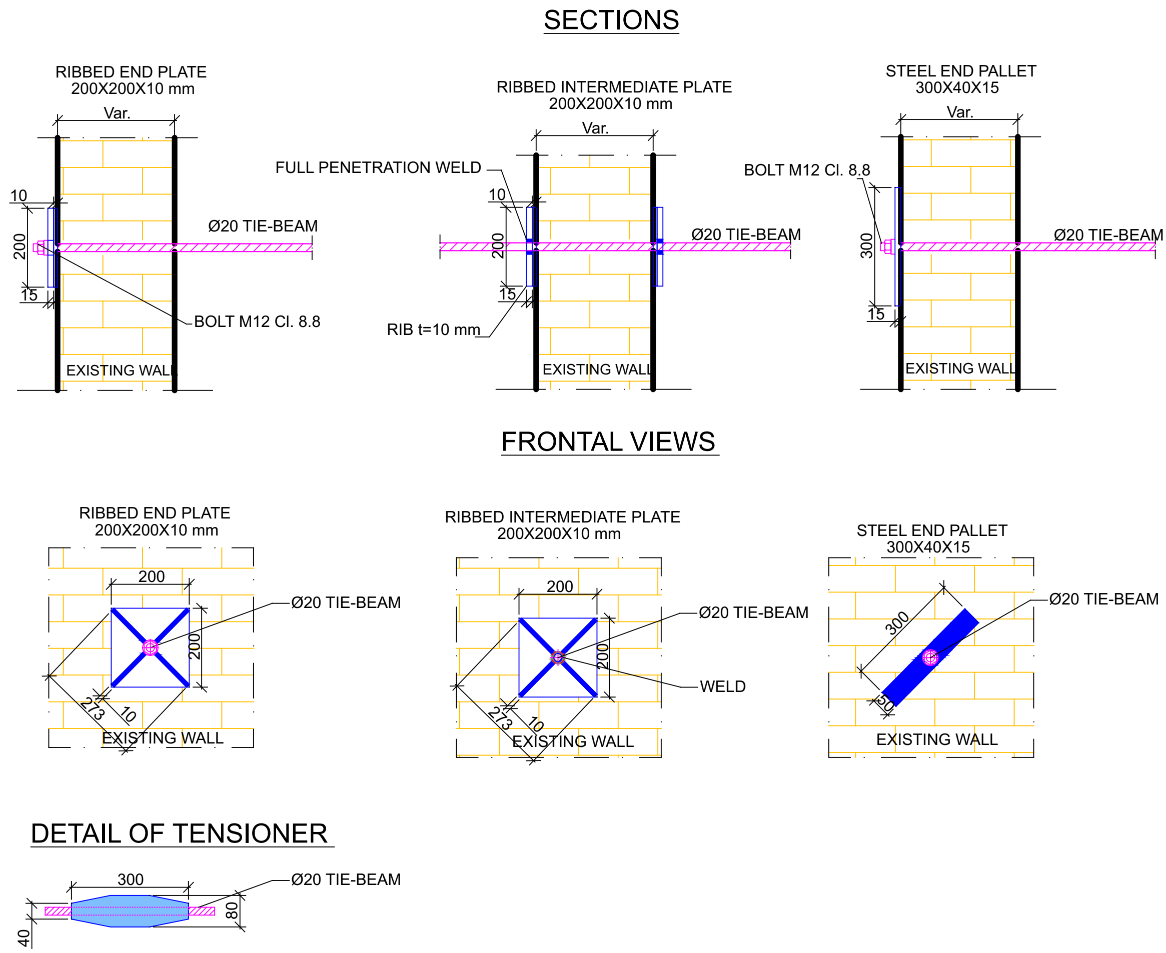


Fig. 14. Details of steel tie-beams anchorage plates.

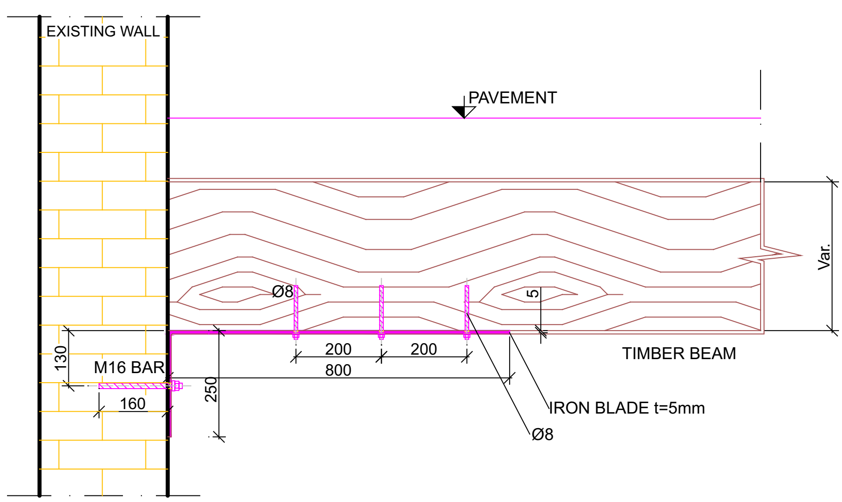
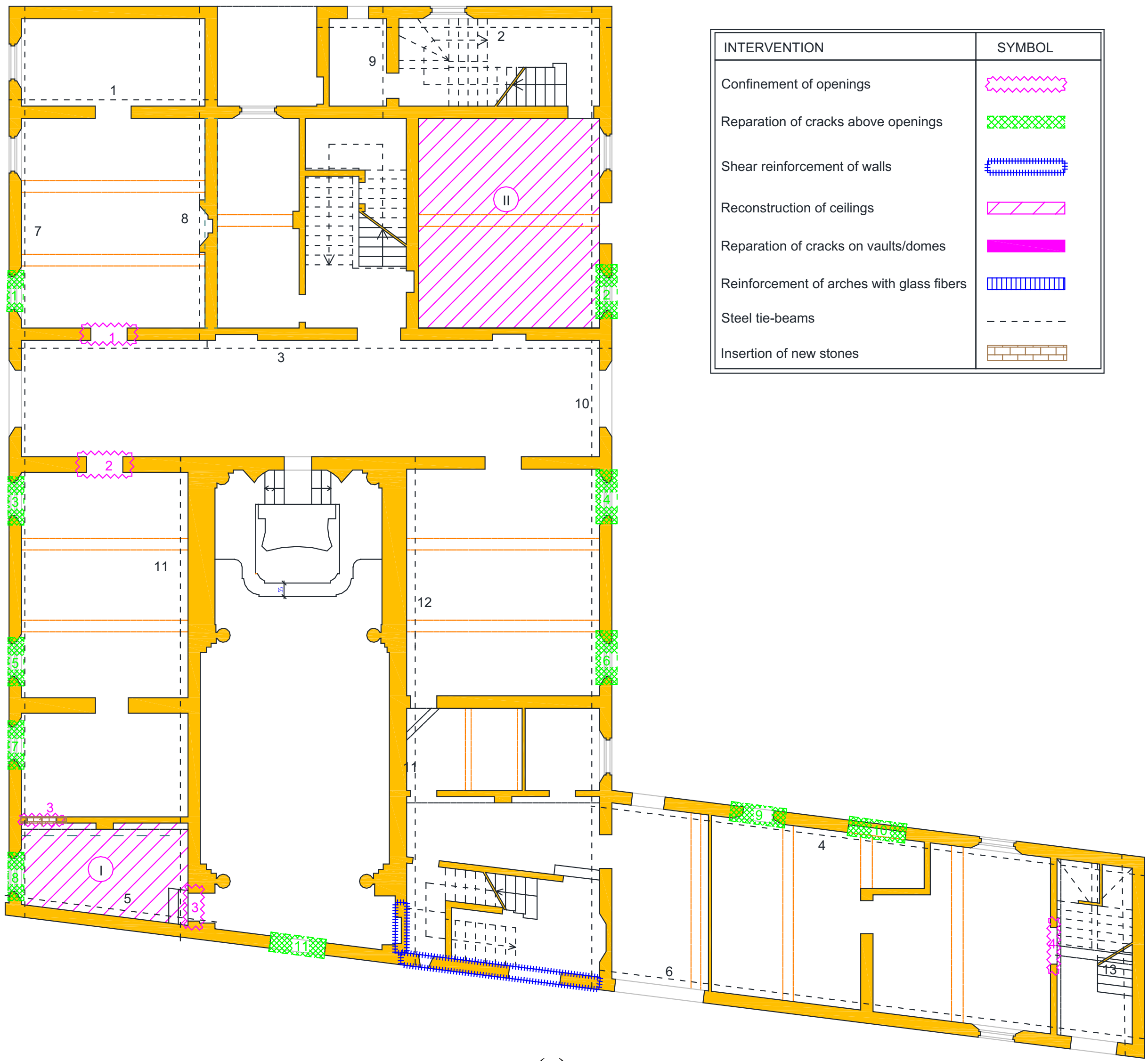


Fig. 15. Detail of the wall-to-floor connection.

From the performed analyses it is apparent that local interventions, even if they do not allow the building complete retrofitting, have also improved its global behaviour, with improvements greater in direction x (8%) than in direction y (2%).

This slight improvement of the building behaviour after retrofitting is due to the limited interventions done. In fact, as significant structural interventions able to improve the global in-plane behaviour of the building, only shear reinforcement of few walls and reinforcement of the arches, vaults and domes of the oratory by glass fibre nets have been done. In particular, the best benefit due to the aforementioned interventions is revealed in direction x, since shear reinforcement of walls has been made in that direction only. On the other hand, the wide use of steel tie-beams has been made to avoid out-of-plane local mechanisms of masonry walls and, therefore, does not condition the building global behaviour.



(a)

Fig. 16. Placement of local repairing interventions at the ground (a) and first (b) floors.

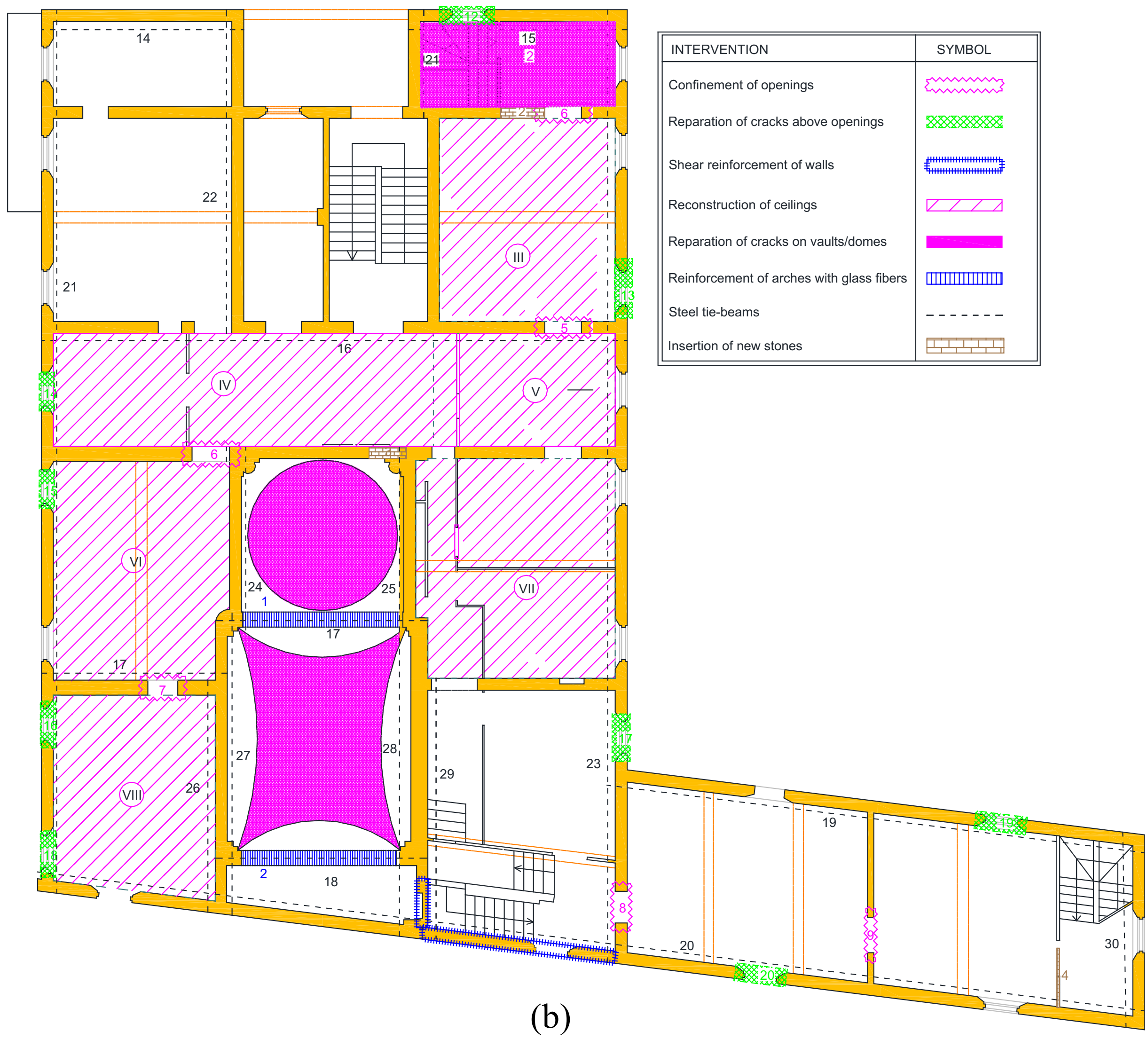
7. Conclusions

The paper dealt with the usability check and the seismic vulnerability evaluation and repairing of a building of Cento, a city in the district of Ferrara (Italy), having historical and cultural importance so to be considered as an example of the Italian cultural heritage.

The damages suffered by the building after the recent Emilia-Romagna earthquake have shown its poor seismic behaviour, which has been occurred without local first mode mechanism, even if a not very efficient connection degree both between walls and floors and among orthogonal walls sometimes has been noticed. Furthermore, in the oratory included in the building, some cracks into vaults and arches, as well as incipient local overturning mechanisms of lateral walls, have been recorded. Other than these structural problems, some cracks into paintings and frescoes have been noticed, they requiring appropriate and delicate restoring interventions.

The seismic vulnerability analysis of the building carried out through the simplified approach (LV1 analysis level) given by Italian Guidelines on Cultural Heritage, together with the visual observation of detected damages, has demonstrated that the examined structure has an average vulnerability degree. In addition, the global non-linear numerical analysis of the building performed according to the refined approach of the abovementioned code (LV3 analysis level) has shown the real structural behaviour, also showing that the simple analysis level is too on the safe side in predicting the building response under earthquake. In fact, the participating seismic mass considered by the code is not appropriate for irregular buildings, as the inspected one, and tends to underestimate the construction capacity acceleration. Therefore, the building seismic safety factor is reduced too much in comparison to that deriving from LV3 analysis level.

Finally, repairing and strengthening local interventions, such as shear reinforcement of masonry walls, vaults and domes by glass fibres, steel tie-beams, connection systems among walls and floors, confinement of openings, substitution of cracked stones with new ones, closure of cracks and reconstruction of damaged ceilings, have been foreseen in order to improve locally the seismic behaviour of the investigated building. The effectiveness of such interventions has been demonstrated by pushover analysis on the



(b)

Fig. 16 (*continued*)

Table 3
Results related to the worst load conditions deriving from LV3 analysis level on the repaired building.

Dir.	Load cond.	Ecc. (cm)	d_u (cm)	d_{max} (cm)	q^*	α_u
$-x$	1st mode	133.9	0.60	0.65	2.298	0.949
$+y$	1st mode	145.6	1.32	0.89	3.380	0.888

repaired building, whose seismic behaviour, as expected, resulted to be slightly improved with respect to that of the original building.

Further developments of the study will be devoted to the implementation of a more general procedure to be incorporated within the Italian Guidelines on Cultural Heritage in order to correctly foresee the seismic behaviour of irregular palaces and villas.

Acknowledgements

The financial support given by the DPC-ReLUIS 2010-2013 research project for the usability check of the investigated building is gratefully acknowledged.

References

[1] http://en.wikipedia.org/wiki/2012_Northern_Italy_earthquakes.
[2] Formisano A, Di Feo P, Grippa MR, Florio G. L'Aquila earthquake: a survey in the historical centre of Castelvecchio Subequo. In: Proc of the COST action C26 final conference "Urban Habitat Constructions under Catastrophic Events", Federico M. Mazzolani, Chair, Naples, 2010 September 16–18. London: CRC Press, Taylor & Francis Group; 2010. p. 371–6. ISBN: 978-0-415-60685-1.
[3] Indirli M, Kouris LA, Formisano A, Borg RP, Mazzolani FM. Seismic damage assessment of unreinforced masonry structures after the Abruzzo 2009 earthquake: the case study of the historical centres of L'Aquila and Castelvecchio Subequo. Int J Archit Heritage Conserv Anal Restor 2013;7(5):536–78.
[4] Krstevska L, Tashkov L, Naumovski N, Florio G, Formisano A, Fornaro A, et al. In-situ experimental testing of four historical buildings damaged during the 2009 L'Aquila earthquake. In: Proc of the COST action C26 final conference "Urban Habitat Constructions under Catastrophic Events", Federico M. Mazzolani, Chair, Naples, 2010 September 16–18. London: CRC Press, Taylor & Francis Group; 2010. p. 427–32. ISBN: 978-0-415-60685-1.
[5] Cattari S, Degli Abbati D, Ferretti D, Lagomarsino S, Ottonelli D, Rossi M, et al. The seismic behaviour of ancient masonry buildings after the earthquake in Emilia (Italy) on May 20th and 29th. Ing Sismica 2012;Anno XXIX(2–3):87–111.
[6] Faggiano B, Formisano A, D'Aniello M, Landolfo R. Steel constructions in the frame work of the Emilia earthquake. Progettazione Sismica 2012;3:189–200 [in Italian].
[7] Formisano A. Seismic damage assessment of school buildings after 2012 Emilia-Romagna earthquake. Ing Sismica 2012;Anno XXIX(2–3):72–86.

[8] Marzo A, Marghella G, Indirli M. The Emilia-Romagna earthquake: damages to precast/prestressed reinforced concrete factories. Ing Sismica 2012;Anno XXIX (2–3):132–45.
[9] Novelli VI, D'Ayala D. Assessment of the most damaged historic centres of the Region Emilia-Romagna due to the earthquake of the 20th and 29th of May 2012. Ing Sismica 2012;Anno XXIX(2–3):59–70.
[10] Savoia M, Mazzotti C, Buratti N, Ferracuti B, Bovo M, Ligabue V, et al. Damages and collapses in industrial precast buildings after the Emilia earthquake. Ing Sismica 2012;Anno XXIX(2–3):120–30.
[11] Mucciarelli M, Liberatore D. Guest editorial: the Emilia 2012 earthquakes, Italy. Bull Earthq Eng 2014;12(5):2111–6.
[12] Gallipoli MR, Chiauzzi L, Stabile TA, Mucciarelli M, Masi A, Lizza C, et al. The role of site effects in the comparison between code provisions and the near field strong motion of the Emilia 2012 earthquakes. Bull Earthq Eng 2014;12 (5):2211–30.
[13] Dolce M, Di Bucci D. National Civil Protection Organization and technical activities in the 2012 Emilia earthquakes (Italy). Bull Earthq Eng 2014;12 (5):2231–53.
[14] Penna A, Morandi P, Rota M, Manzini CF, da Porto F, Magenes G. Performance of masonry buildings during the Emilia 2012 earthquake. Bull Earthq Eng 2014;12(5):2255–73.
[15] Sorrentino L, Liberatore L, Decanini LD, Liberatore D. The performance of churches in the 2012 Emilia earthquakes. Bull Earthq Eng 2014;12 (5):2299–331.
[16] Cattari S, Degli Abbati S, Ferretti D, Lagomarsino S, Ottonelli D, Tralli A. Damage assessment of fortresses after the 2012 Emilia earthquake (Italy). Bull Earthq Eng 2014;12(5):2333–65.
[17] Sorrentino L, Liberatore L, Liberatore D, Masiani R. The behaviour of vernacular buildings in the 2012 Emilia earthquakes. Bull Earthq Eng 2014;12 (5):2367–82.
[18] Masi A, Santarsiero G, Gallipoli MR, Mucciarelli M, Manfredi V, Dusi A, et al. Performance of the health facilities during the 2012 Emilia (Italy) earthquake and analysis of the Mirandola hospital case study. Bull Earthq Eng 2014;12 (5):2419–43.
[19] Da Porto F, Munari M, Prota A, Modena C. Analysis and repair of clustered buildings: case study of a block in the historic city centre of L'Aquila (Central Italy). Constr Build Mater 2013;38:1221–37.
[20] Carocci C. Small centres damaged by 2009 L'Aquila earthquake: on site analyses of historical masonry aggregates. Bull Earthq Eng 2012;10(1):45–71.
[21] D'Ayala D, Speranza E. Definition of collapse mechanisms and seismic vulnerability of historic masonry buildings. Earthq Spect 2003;19(3):479–509.
[22] Giuffrè A, et al. Safety and conservation of historical centres: the case of Ortigia. Rome: Laterza Ed.; 1993 [in Italian]. ISBN 8842042501.
[23] Vicente R, Parodi S, Lagomarsino S, Varum H, Mendes Silva JAR. Seismic vulnerability and risk assessment: case study of the historic city centre of Coimbra, Portugal. Bull Earthq Eng 2011;9(4):1067–96.
[24] Sorrentino L, Paviglianiti P, Liberatore D. "Behavior of a building cluster in Paganica under the 2009 L'Aquila earthquake: comparison between two nonlinear models" (in Italian), OpenSees days "Modelling, analysis and calculation of structures in seismic zone". In: Proc of the 1st Italian conference, Rome, May 24–25; 2012. p. 237–44.
[25] Formisano A, Marzo A. Seismic vulnerability assessment and retrofitting of a cultural heritage building in Cento, Ferrara, Italy. In: Topping BHV, Iványi P, editors. Proceedings of the twelfth international conference on computational structures technology. Stirlingshire [UK]: Civil-Comp Press; 2014. http://dx.doi.org/10.4203/ccp.106.82 [Paper 82].
[26] Regional Direction on Cultural Heritage. Preservation of the artistic-historic interest heritage. Law n. 1089 promulgated on 1939, June 1st; 1939 [in Italian].
[27] http://www.protezionecivile.gov.it/resources/cms/documents/Scheda_AEDES.pdf.
[28] DPCM 09/02/11. Evaluation and reduction of seismic risk of cultural heritage with reference to the Technical Standards for Constructions promulgated by the Ministry of Infrastructure and Transport on 2008 January 14th; 2011.
[29] Formisano A, Mazzolani FM, Florio G, Landolfo R. A quick methodology for seismic vulnerability assessment of historical masonry aggregates. In: Proc of the COST action C26 final conference "Urban Habitat Constructions under Catastrophic Events", Federico M. Mazzolani, Chair, Naples, 16–18 September 2010. London: CRC Press, Taylor & Francis Group; 2010. p. 577-82. ISBN: 978-0-415-60685-1.
[30] Formisano A, Mazzolani FM, Florio G, Landolfo R, De Masi G, Delli Priscoli G, et al. Seismic vulnerability analysis of historical centres: a GIS application in Torre del Greco. In: Proc of the COST C26 action final conference: "Urban Habitat Constructions under Catastrophic Events". Federico M. Mazzolani, Chair, Naples, 2010 September 16–18. London: CRC Press, Taylor & Francis Group; 2010. p. 583–8. ISBN: 978-0-415-60685-1.
[31] Formisano A, Florio G, Landolfo R, Mazzolani FM. Numerical calibration of a simplified procedure for the seismic behaviour assessment of masonry building aggregates. In: Proc of 13th international conference on civil, structural and environmental engineering computing, Creta, 6–9 September, 2011.
[32] Formisano A, Castaldo C, Mazzolani FM. Non-linear analysis of masonry building compounds: a comparison of numerical and theoretical results. In: Topping BHV, Iványi P, editors. Proceedings of the fourteenth international conference on civil, structural and environmental engineering computing. Stirlingshire (UK): Civil-Comp Press; 2013. http://dx.doi.org/10.4203/ccp.102.66 [Paper 66].
[33] Formisano A, Florio G, Landolfo R, Mazzolani FM. Numerical calibration of an easy method for seismic behavior assessment on large scale of masonry building aggregates. Adv Eng Softw 2015;80:116–38.
[34] Ministerial Decree of Public Works (M. D.) (14/01/2008). New technical codes for constructions. Official Gazette of the Italian Republic n. 29 published on 2008, February 4th; 2008.
[35] Italian Ordinance n. 51 "Criteria and ways to assign funds for repairing and restoration with seismic retrofitting of residential building units showing significant damages due to the 2012 May 20th and 29th seismic events and declared as unusable (level E0)". President of the Emilia-Romagna Region as Delegate Commissioner, published on 2012 October 5th; 2012.
[36] Lagomarsino S, Podestà S, Resemini S, Curti E, Parodi S. Mechanical models for the seismic vulnerability assessment of churches. Proc of the 4th international seminar on structural analysis of historical constructions (SAHC04), Padua (Italy), vol. 2. London: Taylor & Francis; 2004. p. 1091–101. 2004 November 10–13.
[37] Penna A, Lagomarsino S, Galasco A. A nonlinear macroelement model for the seismic analysis of masonry buildings. Earthq Eng Struct Dyn 2013. http://dx.doi.org/10.1002/eqe.2335.
[38] ASCE/SEI 41/06. Seismic rehabilitation of existing buildings. Reston (VA): American Society of Civil Engineers; 2007.
[39] Eurocode 8 - EN 1998-3. Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings. Brussels (Belgium): CEN; 2005.
[40] Ministerial Circular (M. C., 02/02/2009). Instructions for the application of the "New Technical Code for Constructions" (M. D: 14/01/2008). Official Gazette of the Italian Republic published on February 26th; 2009.