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Seismic vulnerability of building heritage of the University of Bologna: methodology and analysis

G. Cimino(1), I. Ricci(2), G. Gasparini(3), T. Trombetti(4)

(1) Ph.D. Student, University of Bologna - Department of Civil Engineering, Chemical, Environmental and Materials, Captain of Italian Army Engineers Corps, gianluigi.cimino2@unibo.it
(2) Research Assistant, University of Bologna - Department of Civil Engineering, Chemical, Environmental and Materials, ilaria.ricci6@unibo.it
(3) Assistant Professor, University of Bologna - Department of Civil Engineering, Chemical, Environmental and Materials, giada.gasparini4@unibo.it
(4) Associate Professor, University of Bologna - Department of Civil Engineering, Chemical, Environmental and Materials, tomaso.trombetti@unibo.it

Abstract

Due to the recent code developments and of the growing attention given to the seismic safety of structures, especially after the last Italian earthquakes, the analysis and verification of existing building heritage have become a fundamental tool to assess the seismic vulnerability, to safeguard human lives and to plan structural interventions.

The Italian building heritage is characterized by high complexity and heterogeneity, both from architectural and structural points of view. It consists in structures built in various ages, placed both in the city center and in the outskirts, erected by different structural techniques and with different uses. For all these reasons, it is important to define a methodology to obtain comparable results to plan the future activities of risk analysis, assessment and management.

Before providing a result about the level of safety of an existing building, it is essential to acquire the right knowledge and to choose a method of analysis that can capture as much as possible its actual behaviour under seismic action. This study must take into account the conventional limits imposed by the codes and those of the analytical instruments.

Nowadays it is fundamental that any building manager or owner correctly knows these data to implement a strategy of prevention that can cope with the seismic hazard of the territory and to optimize the economic resources in order to eliminate vulnerabilities through specific intervention programs.

The purpose of this research is to identify a methodology of verification easily manageable and adaptable to many different buildings, but at the same time able to determine the actual state of structure in terms of critical steps and structural deficiencies. In order to develop this methodology, the building heritage of the University of Bologna was taken as a reference. In particular the present building heritage has an overall floor area of approximately 470,000 m^2 and consists in 59 buildings (composed by 104 Structural Units “US”), placed in the municipalities of Bologna and Ozzano Emilia.

Keywords: building heritage, seismic vulnerability, methodology, capacity, ranking.

1. Introduction

In the last years the issue of assessment and verification of seismic vulnerability of existing building has become a key point of analysis and study to improve the performances of structures subject to earthquake and to safeguard human lives.
In this framework, the Technical Office of the University of Bologna (AUTC) and the Department of Civil, Chemical, Environmental and Materials Engineering of the University of Bologna (DICAM) signed, in February 2012, an agreement to develop a scientific study to evaluate the seismic vulnerability of the building heritage of the Athenaeum.

1.1 The building heritage of the Athenaeum

The scientific study was confined to the municipalities of Bologna and Ozzano Emilia and covered 59 buildings, composed of 104 Structural Units "US"[1], having an overall floor area of 471.145 m^2 and an overall volume of 1.928.402 m^3.
The original data were obtained from specific forms requests by the Italian Prime Minister's Office - Department of Civil Protection and fill in by AUTC.
The analyzed US can be divided into the categories identified in the following figures, depending on the structural type (figure 1) and the year of construction (figure 2).

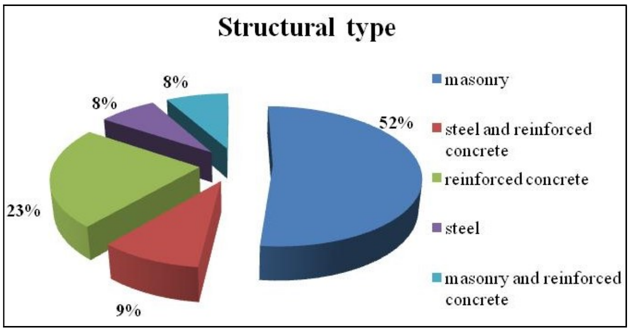

Fig. 1 - Chart of the distribution of the US depending on the structural type

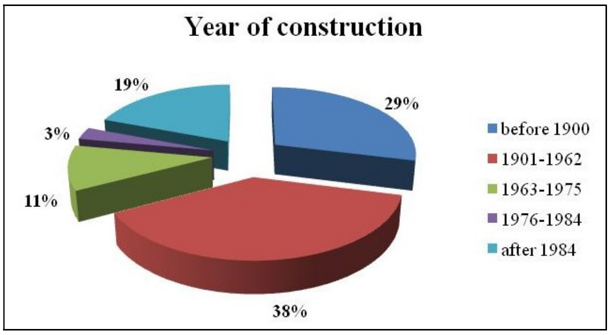

Fig. 2 - Chart of the distribution of the US depending on the year of construction

Over the years, the building heritage of the Athenaeum has developed following the transformation of Bologna. It is composed of existing historical buildings (retrofitted historical palaces) and existing recent structures built mainly during the twentieth century. In this period, the Athenaeum experienced the most important phase of renewal and development of its building heritage which was enlarged realizing new structures for the various Departments on the free areas immediately around the inner city This development around the inner city never diminished the peculiar and valuable existing heritage which has continued to grow and to characterize the buildings in use at the University.
After the Second World War, as other Italian Universities, Bologna had to deal with the increase in students, which induced the need of reorganizing rooms, raising teaching areas, retrofitting existing structures (especially the ones placed in the inner city). In this phase, programmed management and rational exploitation of structures became fundamental to guarantee adequate maintenance both ordinary and extraordinary [1].
This management could not ignore the evaluation of the static and seismic stability of structures, AUTC concluded a multi-annual agreement with the DICAM Department to develop technical verification of the seismic vulnerability of the structures which compose the building heritage of the Athenaeum, in order to plan the future activities of risk analysis, assessment and management on the base of the results of activities carried out by the DICAM.

[1] The Structural Unit (US) is a portion of a building aggregate characterized by uniform and homogeneous structural behaviour (see paragraph 8.7.1 in D.M. of 14/01/2008).

1.2 The conditions and operational constraints

The development of a methodology for structural analysis able to assess the level of security is essential to verify that the building meets the structural requirements for which it was designed during its useful life. This check is carried out by comparing the demand (or the effect of actions applied to the structure) and the capacity (or performance that the structure is able to offer against the stresses induced by the actions).
However, for existing buildings, the identification of demand and capacity is affected by many uncertainties both due to the reliability of the available engineering tools of investigation and due to materials, architectural, structural and topographic peculiarities that, over the years, inevitably undergo slow processes of change. Therefore, in order to develop a reliable and realistic vulnerability assessment, it is necessary to adopt an approach that integrates the specific scientific studies with those devoted to the knowledge of the building. With these requirements it was inevitable to define a methodology capable of providing results comparable and uniform to facilitate future activities of management and planning of the technical interventions for all buildings of the Athenaeum, starting from: (a) the knowledge of a complex and heterogeneous building heritage, (b) the availability of limited information, (c) the objectives of the analysis to be performed and (d) the economic and time resources devoted to the research.
However, any method or approach should be referred to a precise framework of laws, which, in this case, is composed by the following Italian Codes:

- O.P.C.M. of 20/03/2003 n. 3274 (shown below as *Ordinanza*);
- D.M. of 14/01/2008 (shown below as *NTC*);
- C.M. of 02/02/2009 n. 617 (shown below as *Circolare*).

Finally, the fundamental element of the scientific studies was the comparison of structural analysis with the knowledge gained from the survey conducted with the objective data available, in other words, the match between the analytical results and the direct confrontation with the state of the structures. This concept was developed in the early '900, at the Royal School for Engineers in Bologna, where Prof. Ing. Silvio Canevazzi, master of Pier Luigi Nervi, provided a "Physical" approach to engineering problems in his course of mechanics applied to buildings. He never failed to point out that the results obtained with the application of theoretical formulations must be united and completed with the experimental observation of reality and the intuitive understanding of the static behaviour of the structures [2].

2. The methodology developed

2.1 The process of knowledge

The process of knowledge of each building was connected to a series of activities. First, (a) all the documentation available was analyzed, such as the original structural and architectural projects, historical photos, more recent technical documentation regarding structural interventions etc. Second, (b) an historical-critical analysis was performed for a correct identification of the existing structural system, through the reconstruction of the manufacturing process and of the changes undergo by the construction over time, as well as all events (fires, earthquakes, wars, etc.) that characterized the history of buildings. Later (c) a structural-geometric survey was performed to check the documentation available and to identify the structural elements, also taking into account both (d) the quality and the condition of the materials and of the components and (e) the presence of cracks, damages or disrepair. Finally, (f) the mechanical characterization of materials was carried out to achieve an adequate knowledge of the characteristics of the materials through visual inspections in situ, demolition and experimental tests.

2.2 Numerical modeling

Thanks to the knowledge acquired, a three-dimensional geometric CAD model composed of vertical and horizontal structural elements was developed for each structure. From this CAD was derived the three-dimensional numerical finite element (FEM) model by using the software "SAP 2000" developed by “CIS Berkeley”.

The walls (masonry walls or r.c. walls) were generally modeled with 50 cm x 50 cm 2D finite element, called "shell", having three or four nodes and variable thickness depending on the case. These elements took into account both the flexural and the membrane behavior. The effects of the shear were evaluated by the formulation of Reissner-Mindlin [3].
Beams and columns were modeled with "beams", having the dimensions of the structural components. These elements took into account the biaxial bending moment, torsion, axial strain, the shear deformation [4] and were characterized by six degrees of freedom in each end.
The horizontal structural elements were modeled with "shell none" (without thickness) that allowed to transfer the loads applied to the vertical elements of competence. The horizontal elements were considered infinitely rigid, where a slab in reinforced concrete or other similar conditions were present.
The behaviour of the materials was modeled with the typical constitutive laws, using the mechanical characteristics provided by the experimental campaign or by the codes, depending on the level of knowledge reached for the specific building.

2.3 The benchmarks for the vulnerability assessment

According to the Italian Codes, the buildings of the present study were classified as constructions of Type 2 (ordinary buildings) and Class of Use III (significant crowding). To these parameters correspond a nominal life V_N=*50years* and a Coefficient of Use C_U=1,50. These values provide a reference period V_R for the seismic action by Eq. (1):

$$V_R = V_N \times C_U = 50 \times 1{,}5 = 75\,years \tag{1}$$

To define the reference seismic action it was chosen to consider the same reference spectrum for all buildings to be able to compare the obtained results and that the Italian Codes considers for the Limit State SLV[2].
The characterization of the geomorphology of the site was carried out in an univocal way on the basis of data of the surrounding area, which indicates that the category of the soil for seismic analysis is the C, according to the Italian Codes.

2.4 Analysis of the structures

In order to determine the actions on the structures, linear dynamic analysis (or modal) with response spectrum and behaviour factor "q"[3] was performed on all the FEM models. In order to allow a global interpretation of the results the following values were considered:

- q=1,00 for the verification of the local mechanisms of the masonry elements;
- q=2,25 for the verification of the global mechanisms of the masonry structures (considered irregular);
- q=3,00 for the verification of the mechanisms of the ductile elements in reinforced concrete and steel;
- q=1,50 for the verification of the mechanisms of the fragile elements in reinforced concrete and steel.

The overall response of the system was expressed as a superposition of the effects related to the individual modes considered in according to the combination of the square root of the sum of the squares (SRSS), wherein for the maximum response is meant the square root of the sum of the squares of the effects related to the modes considered.
The average strength of the materials was assumed: (1) equal to the average values f_m obtained from the experimental tests, when a proper experimental campaign was performed, (2) equal to the values provided by the Italian Codes, in all the other cases. The design strength of the materials, f_d, used in the evaluation of the capacity of the structural elements was evaluated reducing the average strength with two coefficients Eq. (2): (a) a

[2] The SLV is the condition such that following the earthquake the building undergoes breakage of non-structural components and installations and significant damage of structural components which is associated with a significant loss of rigidity against horizontal actions; the construction retains a part of the strength and stiffness for vertical actions and a safety margin with regard to the collapse for horizontal seismic actions.
[3] The behaviour factor is indicated as "q" in European practice and as "R" in United States practice. This is used to derive the design acceleration response spectrum from its linear elastic equivalent.

Confidence Factor "FC", corresponding to the Level of Knowledge "LC"[4], and (b) a safety coefficient, γ, depending on the material (according to Italian Codes: γ=3,00 for masonry, γ=1,50 for concrete, γ=1,15 for steel in r.c. elements and γ=1,05 for the steel of carpentry):

$$f_d = \frac{f_m}{FC \cdot \gamma} \tag{2}$$

2.5 Criteria for static analysis of structural elements

The evaluation of the safety of masonry walls towards vertical static loads at the Limit State SLU[5], was performed taking into account only the normal stress, in order to get an indication on the generalized state of stress. This safety coefficient was defined in Eq. (3):

$$FS_{masonry} = \frac{f_d}{\left(N_{SLU} / A_m\right)} \tag{3}$$

In the previous equation: (a) N_{SLU} is the normal stress at the SLU; (b) A_m is the area of the masonry wall.
For r.c. beams, the evaluation of the safety towards vertical static loads at SLU was performed calculating the safety factor corresponding to bending and shear stresses Eq. (4):

$$FS_{beam} = \min\left(\frac{M_{Rd}}{M_{Sd}};\frac{V_{Rd}}{V_{Sd}}\right) \tag{4}$$

In the previous equation: (a) M_{Rd} and V_{Rd} are the bending and shear strenght of the beam evaluated according to the formulations given in Italian Codes (paragraph 4.1.2.1 of the *NTC*); (b) M_{Sd} and V_{Sd} are the bending and shear stress of the beam.
For r.c. columns, the evaluation of the safety towards vertical static loads at SLU was performed calculating the safety factor corresponding to combined axial and bending stress and shear stress Eq. (5):

$$FS_{column} = \min\left(\frac{M_{Rd,x}}{M_{Sd,x} \cdot 1,3};\frac{M_{Rd,y}}{M_{Sd,y} \cdot 1,3};\frac{V_{Rd}}{V_{Sd}}\right) \tag{5}$$

In the previous equation: (a) $M_{Rd,x}$ and $M_{Rd,y}$ are the combined axial and bending strength obtained from the axial stress N_{SLU} in the domain of interaction M-N; (b) $M_{Sd,x}$ and $M_{Sd,y}$ are the combined axial and bending stresses in the two principal directions of the column at the SLU; (c) V_{Rd} is the shear strength evaluated according to the formulations given in Italian Codes (paragraph 4.1.2.1 of the *NTC*); (d) V_{Sd} is the shear stress; (e) 1,3 is a factor that amplifies the bending moment both in x and y direction and allows to simplify the verification in presence of combined axial and biaxial-bending stress (paragraph 7.4.4.2.2.1 of the *NTC*).

2.6 Criteria for structural seismic analysis

The seismic verifications were performed evaluating by different Safety Factors (FS), as the ratio between capacity and demand. This operation was conducted for all the various crisis mechanisms, depending on the type of the structural element considered, in order to evaluate the minimum value of the safety factor, FS_{min}, that rule the collapse of the structure.
In the case of masonry, the local mechanisms of collapse were first evaluated through a limit analysis of rigid bodies by means of kinematic linear approaches. This procedure is generally more conservative with respect to the non-linear one, because the resources in the non-linear phase of the motion cannot be adequately taken into consideration [5]. To do this the Excel application CINE (Conditions of instability in buildings) provided by RELUIS (version 1.0.4, September 2009) was used.

[4] The Italian Codes establishes that the level of knowledge acquired LC of a building is determined by the combination of quality surveys and investigations carried out: at every level (1, 2 or 3) corresponds a value of confidence value FC (1.35, 1.20, 1.00). It is evident the prize, in terms of increased resistance, which comes from the most complete knowledge.
[5] The SLU is the condition such that, by effect of only the vertical actions, the structural elements collapse for the achievement of its tensile resistance.

Instead, for the global behavior, the collapse mechanisms of shear, in plane and out of plane press-bending were evaluated for each masonry wall. The demand (shear V_d, in plane press-bending M_d and out of plane press-bending $M_{d,fp}$) were evaluated from the results of the dynamic analyses with response spectrum, through the function "section cut" of the software SAP2000.
In particular, as far as the effect of the out of plane actions are concerned, the value of the eccentricity request to the wall e_d was used as a parameter to identify the level of stress Eq. (6). This eccentricity was evaluated as the ratio between the out of the plane moment induced by the seismic action, $M_{d,fp}$, and the normal stress in the wall N:

$$e_d = \frac{M_{d,fp}}{N} \tag{6}$$

As far as the shear capacity is concerned, the masonry was verified towards in plane diagonal traction, which, in existing buildings, is generally more unfavorable respect to the collapse for horizontal scrolling (usually performed for new buildings). This condition occurs when the principal tensile stress of traction at the center of the panel reaches the traction strength of calculation of the masonry f_{td}. Instead, the resistance of the masonry panel was evaluated in accordance with Italian Codes (paragraph C8.7.1.5 of the *Circolare*).
Then, also the in plane press-bending capacity was calculated in accordance with Italian Codes (paragraph 7.8.2.2.1 of the *NTC*) and was based on the following assumptions: (a) not-reactive traction masonry; (b) preservation of plane sections; (c) uniform distribution of compression stresses on a section of reduced dimensions (also known as "stress block" theory).
Given that the collapse of masonry buildings is mainly due to the stability against overturning rather than due to local insufficiency of the material in compression or traction, the capacity of the wall to resist to an earthquake depends primarily on its geometry and on the level of structural connections with the perpendicular elements [6]. For this reason, it was given particular attention to the mechanism of out-of-plane collapse, which can be treated the same as a local mechanism of collapse. However, instead of considering only the "rocking effect" [7], it was decided to evaluate this mechanism also as a criterion for global monitoring. To do this, the ultimate eccentricity, e_u, was introduced as a parameter to identify the capacity of the wall. The ultimate eccentricity is the minimum between the following two quantities Eq. (7):

$$e_u = \min\left(e_{u1} = \frac{t}{6} ; e_{u2} = \frac{M_{u,fp}}{N} \right) \tag{7}$$

where: (a) t represents the thickness of the masonry wall; (b) e_{u1} is the eccentricity of the beginning of damage (that occurs when part of the section is in traction and the axial stress is outside the central core of inertia); (c) e_{u2} is the eccentricity of crushing of the masonry (that occurs when the axial load has an eccentricity that produces a bending moment equal to the ultimate out-of-plane bending moment); (d) $M_{u,fp}$ is the out-of-plane bending moment valued in accordance with Italian Codes (paragraph 7.8.2.2.3 of *NTC*).
As far as the shear capacity of reinforced concrete elements (columns and beams) is concerned the following minimum safety factor was evaluated by Eq. (8):

$$FS = \min\left(\frac{M_{Rd,beam} - M_{static,beam}}{M_{seismic,beam}} ; \frac{M_{Rd,column}(N) - M_{static,column}}{M_{seismic,column}} ; \frac{V_{Rd} - V_{static}}{V_{seismic}} \right) \tag{8}$$

where: (a) $M_{Rd,beam}$ is the bending moment strength rated as the minimum value between the strength at the midpoint and the strength at the ends; (b) $M_{static,beam}$ and $M_{static,column}$ are the bending moment due to the only static vertical loads; (c) $M_{seismic,beam}$ and $M_{seismic,column}$ are the bending moments due to the seismic action obtained from the design spectrum reduced with q=3,00; (d) $M_{Rd,column}$ *(N)* is the combined compressive and bending strength rated from the domain of interaction M-N for the given axial force; (e) V_{Rd} is the shear strength rated as the minimum between the shear strength of the steel and the shear strength of the concrete; (f) V_{static} is the shear due to the only static vertical loads; (g) $V_{seismic}$ is the shear stress due to seismic action obtained from the design spectrum reduced with q=1,50. As for the static criteria, also in this case, the calculation of the safety factor to biaxial press-bending was conducted in a simplified manner, increasing the bending stress of the 30%.
In the end, for each US, all the values of FS_{min} were organized in increasing order, from the minimum value to maximum one, to provide the sequence in which the crisis of the structure developed Eq. (9):

$$PGA_{C,SLV} \geq FS_{\min} \cdot PGA_{D,SLV} \quad (9)$$

In the previous equation: (a) $PGA_{C,SLV}$ is the capacity at SLV in terms of peak ground acceleration; (b) $PGA_{D,SLV}$ is the demand at SLV in terms of peak ground acceleration for the city of Bologna ($PGA_{D,SLV}$ = 0,27g with a return period $TR_{D,SLV}$ of 712 years).

3. Analysis and comparison of the results

3.1 "First collapse" capacity of the building heritage

With reference to the "first collapse" (i.e. the first structural element which reaches the crisis due to the seismic action), for each building were calculated:

- the capacity expressed in terms of peak ground acceleration ($PGA_{C,SLV}$) from the results of seismic analysis;
- the vulnerability index α_V as indicated in the Eq. (10):

$$\alpha_V = \frac{PGA_{C,SLV}}{PGA_{D,SLV}} \quad (10)$$

- the capacity in terms of return period ($TR_{C,SLV}$) using the formula of *NTC* Annex A;
- the vulnerability index α_V* as indicated in the Eq. (11):

$$\alpha_V{}^* = \left(\frac{TR_{C,SLV}}{TR_{D,SLV}}\right)^{0,41} \quad (11)$$

In the case of different buildings with equal index, in order to establish a vulnerability ranking for all the US, the exposure was taken into consideration. The exposure (also known as density of occupation) is the ratio between the number of occupant and the surface of the building.

For the graphical representation of the results (figure 3), were considered the following levels of criticality:

- "HC - High Criticality" (red color), for the US characterized by values of PGA less than 40% of the $PGA_{D,SLV}$ and $TR_{D,SLV}$ less than 75 years;
- "MC - Moderate Criticality" (yellow), for the US characterized by values of PGA included between 40% and 60% of the $PGA_{D,SLV}$ and $TR_{D,SLV}$ included between 76 and 150 years;
- "OC - Ordinary Criticality" (green color), for the US characterized by values of PGA included between 60% and 100% of the $PGA_{D,SLV}$ and $TR_{D,SLV}$included between 151 and 712 years;
- "NC - Not Criticality" (blue color), for the US characterized by values of PGA greater than 100% of $PGA_{D,SLV}$ and $TR_{D,SLV}$greater than 712 years.

The results obtained show that only the 8% of the building heritage of the University of Bologna is characterized by NC level (figure 3). Moreover, the vulnerability ranking obtained show that up to the 84th position (80%) the US are in HC level and, in case of α_V, the first 43 US show a relationship between capacity and demand less than 0,10 (10% of the $PGA_{D,SLV}$).

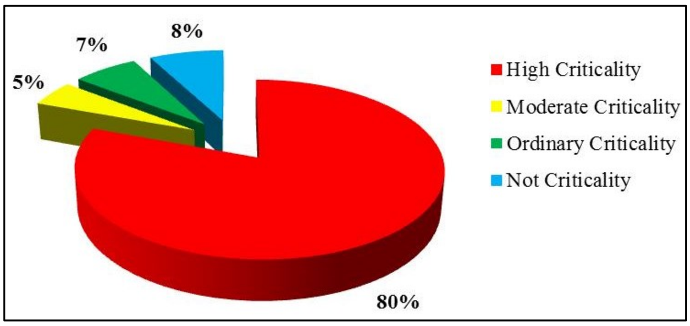

Fig. 3 - Percentages of the US studied according to the levels of criticality defined.

3.2 Probabilistic and deterministic analysis

However the situation that emerge from this study do not correspond to the *status quo* in which the US were observed during the inspections. Hence the need to conduct comparisons between results obtained and probabilistic and deterministic analysis.

The probabilistic analysis was performed by means of a "process of Poisson" [8], a stochastic process which simulates the occurrence of events that are independent from each other and that happen continually over the time. In detail, for each building, was evaluated the probability of having already suffered in the past, throughout its useful life, one or more seismic events characterized by a return period equal to $TR_{D,SLV}$. To do this, it used the Eq. (12):

$$P(TR = TR_{C,SLV}) = 1 - e^{-\frac{t}{TR_{C,SLV}}} \tag{12}$$

where: (a) $t=2016-n$, whit n the year of construction of each US; (b) $e=2,718$ is the number of Neplero.

The results obtained shows that most of the US included in the HC level presents high value of $P(TR=TR_{C,SLV})$ (figure 4). However, the fact that the overall building heritage is still intact despite the occurred seismic events (no relevant damages were observed during the inspections and no significant structural intervention have been performed along the years) indicates that the result obtained from this study in terms of capacity expressed in return period ($TR_{C,SLV}$) are rather conservative.

N. U.S.	Property name		Hosted department	Structural type	Year of construction	Employment density [people / sq]	$TR_{C,SLV}$	t [years]	t / $TR_{C,SLV}$	Probability of occurrence
68	Edificio ex Sirani		CIRAM	masonry	1400	0,041	30	615	20,500	100%
20	Palazzo Malvezzi-Campeggi		Centro Interdipartimentale CIRSFID	masonry	1500	0,092	30	515	17,167	100%
101	Casa Non Grande dei Bentivoglio		Istituto Botanico	masonry	1500	0,023	30	515	17,167	100%
102	Edificio Piazza Verdi 3		APOS	masonry	1500	0,039	30	515	17,167	100%
99	Palazzo Gaudenzi		CIRSFID	masonry	1529	0,041	30	486	16,200	100%
8	San Giovanni in Monte	DiSCi ed EDU "G. M. Bertini"	Dipartimenti DiSCi ed EDU " G.M. Bertin"	masonry	1549	0,031	30	466	15,533	100%
9	San Giovanni in Monte	Cilta	Sede centrale del CILTA	masonry	1549	0,031	30	466	15,533	100%
10	San Giovanni in Monte	Collegio Erasmus	Collegio Erasmus	masonry	1549	0,035	30	466	15,533	100%
12	Palazzo Poggi	Edificio storico e Torre libraria	Rettorato, Museo, Torre Libraria(Nucleo storico)	masonry	1550	0,045	30	465	15,500	100%
13	Palazzo Poggi	Cà Grande Malvezzi	Amministrazione (Ca' Grande Malvezzi)	masonry	1550	0,026	30	465	15,500	100%

Fig. 4 – Values of $P(TR=TR_{C,SLV})$ corresponding to some of the US studied.

In order to deeply analyze the results obtained in the previous section, a Deterministic Seismic Hazard Analysis (DSHA) was performed with the aim of identifying (1) how many times, during their lives, the buildings had faced earthquakes and (2) which values of peak ground acceleration characterized these events.

According to the Italian Seismic Zonation (ZS9, figure 5), 386 earthquakes (out of 1400) were selected from CPTI11 catalog of National Institute of Geophysics and Volcanology (INGV). Moreover, 2 events of the Emilia earthquake of 2012 were added to the previous events. Subsequently, the PGA in the city center of Bologna (Latitude 44,4949° and Longitude 11,3426°) was computed by the attenuation law of "Sabetta-Pugliese" [9], reported in Eq. (13):

$$\log_{10} PGA = -1{,}845 + 0{,}363M - \log_{10}\sqrt{D^2 + 5^2} + 0{,}195S \tag{13}$$

where: (a) M is the moment magnitude given by CPTI11 catalog; (b) D is the distance in kilometers from the epicenter and the city center of Bologna; (c) $S=0$ for deep and hard soils; (d) $S=1$ for soft soils, similar to those of the US studied. The values of PGA in Bologna for some of the most significant historical earthquakes having M>5,5 are showed in table 1.

For the Emilia earthquake of 2012 (not included in the CPTI11 catalog yet), the PGA registered in the city center of Bologna was obtained directly from the shake maps produced by INGV and the seismic station ZPP (Latitude 44,5240° and Longitude 11,2040°).

The results of the 388 earthquakes selected for DSHA where presented with red dots in figure 6. The distribution of the red dots shows that all the events recorded in the city of Bologna in about 700 years are characterized by values of PGA lower than the value of $PGA_{D,SLV}=0.27g$ considered as reference in this study.

On the same figure, were added squared dots which indicates the values of the capacity in terms of $PGA_{C,SLV}$ of each US analyzed. In this way it is possible to observe (1) the number of earthquakes that the US faced from their construction to now and (2) the corresponding values of peak ground acceleration which characterized these events.

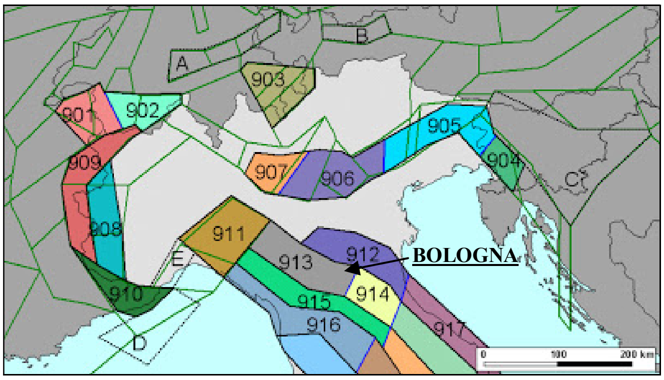

Fig. 5 – ZS9 in the north of Italy and localization of the city of Bologna.

Table 1 – Values of PGA in Bologna for some of the most significant historical earthquakes having M>5,5.

Nr. in CPT11	Year	Affected Area	Latitude	Longitude	PGA [g]
176	1470	Bologna Apennines	44.162	11.037	0.047
211	1501	Modena Apennines	44.519	10.844	0.059
256	1542	Mugello	44.006	11.385	0.058
408	1688	Romagna	44.390	11.942	0.041
738	1796	Oriental Emilia	44.615	11.670	0.062
842	1831	Reggiano	44.752	10.544	0.025
1562	1909	Padania	44.579	11.688	0.057
1803	1920	Garfagnana	44.185	10.278	0.041
2426	1971	Parmense	44.814	10.345	0.022
--	2012	Emilia	44.895	11.263	0.020

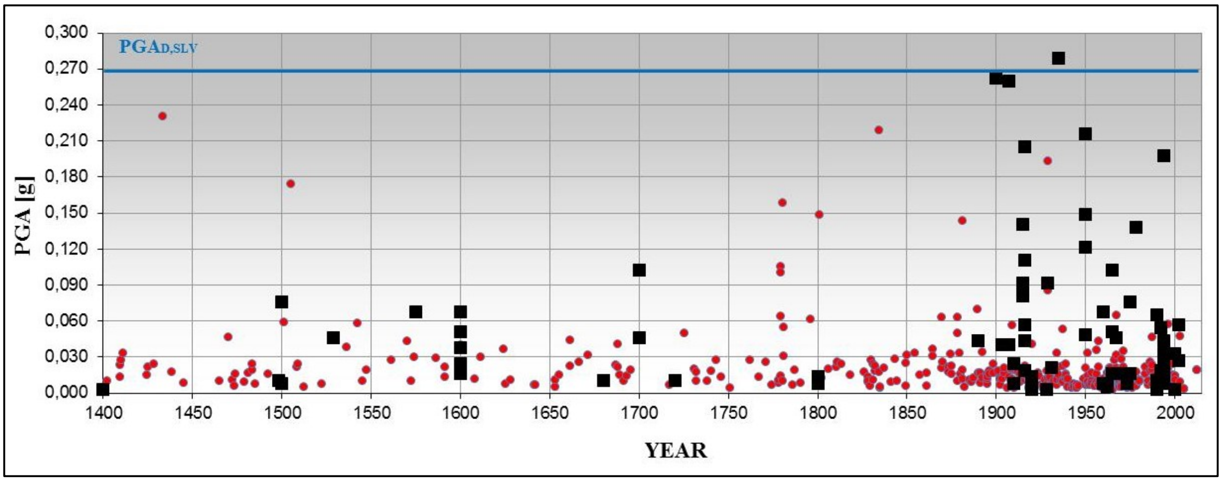

Fig. 6 – Results of DSHA for 388 earthquakes compared with $PGA_{C,SLV}$ of 104 US.

Figure 6 shows that the most part of the building analyzed suffered, throughout its useful life, various seismic events characterized by peak ground accelerations much greater than their “first collapse" capacity, without any damages, documented or observed during the inspections.

3.3 A possible alternative in the definition of the capacity of the building heritage

The comparison between $PGA_{C,SLV}$ and probabilistic/DSHA analysis clearly showed that some buildings characterized by HC level have, actually, a higher security level. Therefore, it was decided to define in an alternative way the capacity of the 104 US analyzed, in order to obtain a reliable vulnerability ranking, which represents the starting point for an accurate planning and management of the structural interventions on the buildings of the Athenaeum. To do this, follow these steps for each individual US were conducted: (a) observation of the activation curves of the most significant elements; (b) identification of the percentage of elements that belongs to the different level of criticality; (c) evaluation of an average value of acceleration $PGA_{C,SLV,med}$ for each level of criticality; (d) determination of the minimum value of $PGA_{C,SLV,med}$; (e) calculation of a new index of vulnerability with the Eq. (14):

$$\beta_V = \frac{\min \mathrm{PGA}_{\mathrm{C,SLV,med}}}{PGA_{D,SLV}} \tag{14}$$

In figure 7 $PGA_{C,SLV}$ is compared with $PGA_{C,SLV,med}$ and it is clear the increase of the structural capacity for all the US analyzed.

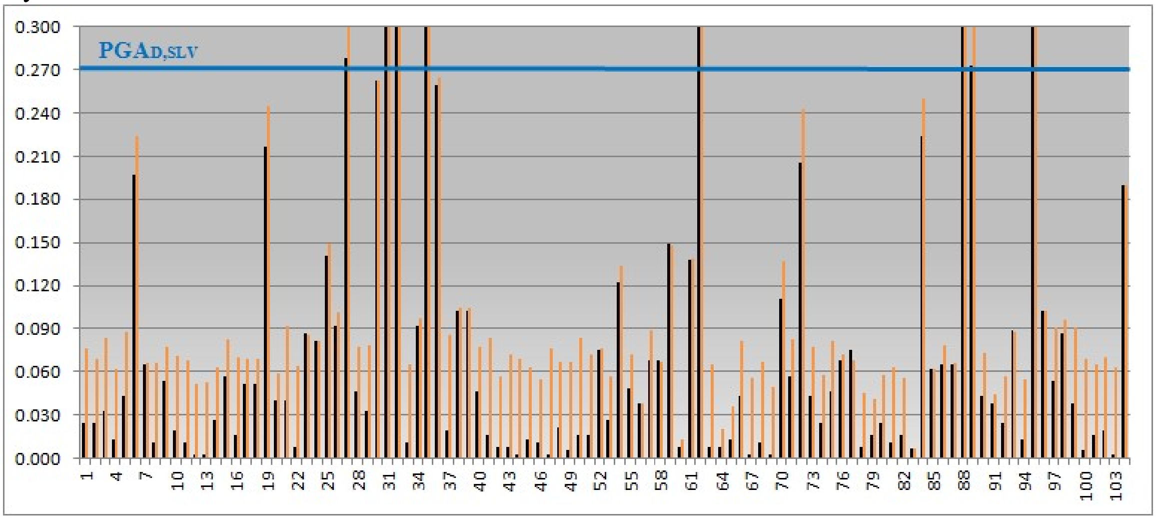

Fig. 7 – Comparison between $PGA_{C,SLV}$ (black line) and $PGA_{C,SLV,med}$ (orange line) for each US analyzed.

It was performed a further comparison adding to the graph of figure 6 the values of $PGA_{C,SLV,med}$ (orange triangles): in figure 8 note how the US allegedly suffered throughout its useful life seismic events characterized by peak ground acceleration to higher their "first collapse" capacity are decidedly less in number.

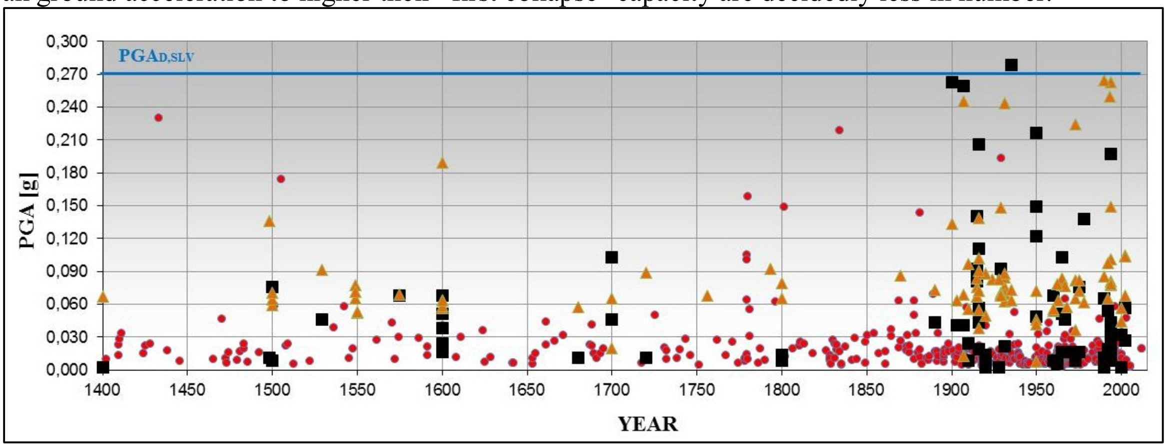

Fig. 8 – Results of DSHA for 388 earthquakes compared with $PGA_{C,SLV}$ and $PGA_{C,SLV,med}$ of the 104 US analyzed.

Finally figure 9 shows the final vulnerability ranking and, once again, until the 84th position, the building heritage is characterized by a HC level, with the difference however that only the first 3 US (and not more 43) have a ratio between capacity and demand lower to 0,10.

Ranking	N. U.S.	Property name		Hosted department	Structural type	Year of construction	Employment density [people / sq]	β_V	$PGA_{C,SLV}$ med
1°	83	Palazzina "ex scuderie"		DICAM - Laboratori didattici	steel	1950	0,208	0,027	0,007
2°	60	Medicina Legale		Istituto di Medicina Legale	masonry	1907	0,068	0,047	0,013
3°	64	Palazzo Giolo Golfarelli		Scuola di Lettere e Beni culturali	masonry + r.c.	1700	0,139	0,075	0,020
4°	65	Biblioteca "Walter Bigiavi"		Biblioteca di Discipline Economiche "W. Bigiavi"	r.c.	1973	0,087	0,134	0,036
5°	56	Edificio Viale Filopanti 9		CIRDCE	masonry	1916	0,089	0,140	0,038
6°	79	Ex Bodoniana	Corpo 2	Scuola di Farmacia, Biotecnologia e Scienze Motorie	r.c.	1950	0,127	0,154	0,042
7°	91	Ex Macello Comunale	Blocco M	Dipartimento di Musica e Spettacolo	steel	2000	0,309	0,165	0,045
8°	78	Ex Bodoniana	Corpo 1	Scuola di Farmacia, Biotecnologia e Scienze Motorie	r.c.	1950	0,127	0,169	0,046
9°	69	Edificio Via Belmeloro 6		FaBiT - Dipartimento di Farmacia e Biotecnologie	masonry	1920	0,029	0,182	0,049
10°	12	Palazzo Poggi	Edificio storico e Torre libraria	Rettorato, Museo, Torre Libraria(Nucleo storico)	masonry	1550	0,045	0,192	0,052
81°	26	Ex Morassutti	Blocco C	Dipartimento di Fisica e Astronomia (DiFA)	r.c.	1994	0,062	0,376	0,101
82°	96	Via Belmeloro 10-12	Blocco C	Scuola di giurisprudenza	masonry	1916	0,078	0,378	0,102
83°	39	Veterinaria	Clinica veterinaria	clinica veterinaria	r.c.	2002	0,100	0,385	0,104
84°	38	Veterinaria	Casa del custode e asilo nido	Casa del custode e asilo nido	r.c.	2002	0,100	0,387	0,104
85°	54	Ex Scuole Ercolani		AFORM - CeSIA	masonry	1900	0,079	0,493	0,133
86°	70	Palazzina della Viola		DIRI - Area Relazioni Internazionali	masonry	1498	0,034	0,505	0,136
87°	61	Botanica		Erbario, Museo Botanico, Orto Botanico	masonry	1916	0,048	0,513	0,138
88°	59	Edificio Via San Giacomo 12		Dipartimento di Scienze Mediche e Chirurgiche	masonry	1929	0,038	0,550	0,149
89°	25	Ex Morassutti	Blocco B	Dipartimento di Fisica e Astronomia (DiFA)	r.c.	1994	0,061	0,553	0,149
90°	104	Via San Vitale 114-116	Corpo 2	Fondazione per le Scienze Religiose	masonry	1600	0,100	0,701	0,189
91°	6	Facoltà di Ingegneria	Aule Nuove - Mensa	Mensa	steel	1973	0,025	0,829	0,224
92°	72	Edificio Via Filippo Re 8		Dip. di Lingue, Letterature e Culture Moderne	masonry	1931	0,034	0,901	0,243
93°	19	Farmacologia/Anatomia Umana		Dipartimento di Farmacia e Biotecnologie FABIT	masonry	1907	0,028	0,908	0,245
94°	84	CUS "Terrapieno"	Piscina	Centro Sportivo Universitario "Terrapieno"	steel	1993	0,053	0,925	0,250
95°	30	Ex Morassutti	Blocco G	Dipartimento di Fisica e Astronomia (DiFA)	r.c.	1994	0,062	0,973	0,263
96°	36	Veterinaria	Plesso H	Medicina veterinaria e biotecnologie animali	r.c.	1990	0,041	0,979	0,264
97°	95	Via Belmeloro 10-12	Blocco B	Scuola di giurisprudenza	masonry	1916	0,069	1,589	0,429
98°	89	Ex Mulino Tamburi	Blocco D	Dipartimento di Filosofia e Comunicazione	masonry	1800	0,071	2,603	0,703
99°	62	Auletta prefabbricata		Aula didattica di Botanica	steel	1965	0,255	2,616	0,706
100°	88	Ex Mulino Tamburi	Blocco C	Dipartimento di Filosofia e Comunicazione	masonry	1800	0,154	3,830	1,034
101°	27	Ex Morassutti	Blocco D	Dipartimento di Fisica e Astronomia (DiFA)	r.c.	1994	0,062	10,852	2,930
102°	35	Veterinaria	Plesso G	Medicina veterinaria e biotecnologie animali	r.c.	1990	0,082	12,573	3,395
103°	32	Veterinaria	Plesso 80/B "stecca"	Medicina veterinaria e biotecnologie animali	steel	1990	0,020	21,954	5,927
104°	31	Veterinaria	Plesso 80/A	Medicina veterinaria e biotecnologie animali	steel	1990	0,044	27,323	7,377

Fig. 9 – Extract of the vulnerability ranking of the 104 US analyzed.

4. Conclusions

In the last years the issue of assessment and verification of seismic vulnerability of existing buildings has become a key point of analysis and study to improve the performances of structures subjected to earthquake and to safeguard human lives, especially after the last seismic events registered in Italy.

This activities, however, has to faced with a building heritage characterized by high complexity and heterogeneity, both from architectural and structural point of view. It consists in structures (a) built in various ages, (b) placed both in the city center and in the outskirt, (c) realized with different structural techniques, (d) characterized by several intended uses. The purpose of this research was to identify a methodology of verification easily manageable and adaptable to many different buildings, but, at the same time, able to determine the actual state of the structure in terms of critical steps and structural deficiencies. The method was designed taking as a reference the building heritage of the University of Bologna (composed of 59 buildings and 104 Structural Units "US"). Taking into account the particular conditions and the operational constraints, it provided comparable and uniform results that facilitate the future activities of management and planning of the technical interventions.

The results obtained from the analysis were presented both in terms of peak ground acceleration ($PGA_{C,SLV}$) and return period ($TR_{C,SLV}$) corresponding to the "first collapse" capacity and in terms of vulnerability indexes, α_V and α_V*, taking account the demand given by the seismic action considered as reference.

From these results, four level of criticality were defined (high, moderate, ordinary and not-critical). The analyses performed shown that the building heritage of the Athenaeum is highly vulnerable: only 8% of the US is

characterized by vulnerability indexes higher than 1 (Not Criticality “NC”). However, the comparison between the above mentioned results and probabilistic and deterministic analyses, indicated that some of these buildings are actually characterized by higher level of safety.
For this reason, it was introduced an alternative definition of the capacity of the US studied in order to obtain a more reliable assessment. The vulnerability ranking obtained with this new definition of capacity, shown that until the 84th position the building heritage is characterized by High Criticality “HC” level and by the first three US having a ratio (in terms of acceleration) between capacity and demand less than 0,10.
However, the fundamental aspect of the studies was always been the "Physical" engineering approach by Prof. Canevazzi. In fact, the comparison between what emerged from the analysis and the outcome observed during the inspections, shows that the results obtained are very conservative, especially considering that it was necessary to employ high safety factors and advance cautionary hypothesis because of available resources and heterogeneity of the structures to be analyzed.
The first development of the present research could be to increase the number of experimental investigations on highly critical buildings. This allows to express a more detailed judgement on their seismic vulnerability and to find more effective actions taking account the settlement and development strategies that the Athenaeum intends to take over.
Subsequently, the developed methodology could be applied on other real estate assets, for example Italian Army building heritage, in order to experimentally validate its reliability. This way permits to optimize human and economic resources and, not least, to obtained a seismic vulnerability assessment of strategic buildings that today still need to be audited.
At the end of this research it is possible to affirm that the process of knowledge of an existing building is a long and complex process and the reliability of the results of a seismic vulnerability assessment is the expression of the level of detail achieved. The role of experts, in fact, should be to provide their own assessments on the basis of knowledge gained, highlighting clearly the degree of uncertainty and, therefore, the reliability. The final decision is up to the authorities, and it can only be in the direction of maximum prudence, informing users on risks and actions taken out.

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