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Energy Procedia 75 (2015) 1325 – 1334

Energy Procedia

The 7th International Conference on Applied Energy – ICAE2015

Multidisciplinary approach to structural/energy diagnosis of historical buildings: a case study

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Abstract

A synergic approach for the investigations of the historical building performances - with reference to both the structural behavior and the energy performances for the space heating and cooling - is presented. The historical masonry building “Palazzo Bosco Lucarelli”, located in Benevento, has been chosen as case study.

The structural and energy analyses are carried out in parallel, especially during the identification of the building characteristics through tests and surveys in-situ. For the structural analysis - beyond examinations on materials - some dynamic tests have been used for better assessing a numerical Finite Element model necessary for the verification of the structure safety. Moreover, being necessary a structural refurbishment, also an energy retrofit could be realized. A rigorous evaluation procedure - aimed to guarantee the necessary reliability of numerical predictions - is performed in order to verify the technical and economical convenience of various energy retrofit solutions.

Peer-review under responsibility of Applied Energy Innovation Institute

Keywords: *Building analyses; Seismic Engineering; Structural Modeling; Performance Based Design; Energy retrofit*

1. Introduction: multidisciplinary approach to structural/energetic behaviors of buildings

The renovation of existing buildings, especially with reference to historical ones, requires a multidisciplinary approach, since different performance aspects are involved. A detailed knowledge of the architecture with integrated tools [1], even if it could appear expensive and time-consuming, allows optimizing the repairing design with sustainable costs. Papadopoulos *et al*. [2] carry out an extensive analysis on the Greek existing buildings. Ramos and Lourenço [3] investigated the vulnerability of city centers in seismic areas and [4] underlines the necessity of rehabilitation of the historical centers.

Instead, the proposed paper suggests an integrated approach to the diagnosis of the monumental buildings, by finding several points of interaction between structural and energy issues, with the aim to identify a valid basis for the design of integrated interventions of structural reinforcement (in areas characterized by seismic risks) and energy refurbishment. The case study of Palazzo dell' Aquila Bosco-

Peer-review under responsibility of Applied Energy Innovation Institute
doi:10.1016/j.egypro.2015.07.201

Lucarelli is proposed. This is an administrative and teaching building of the Department of Engineering of the University of Sannio, already interested by detailed structural studies [5] and investigations of energy performances [6]. All parameters necessary to estimate the dynamic behavior of the building, the structural safety the energy building performance and are investigated.

The investigations to achieve a complete knowledge of an existing masonry building, may be grouped in various categories. Firstly, historical inquiries allows to analyze the origin of the structure. Successively, detailed surveys of geometry and materials allow detection of characteristic features. The assessment of the mechanical properties of the various constituent materials and of their assembling has to be investigated by means of detailed *in-situ* testing [7]. About it, the dynamic tests conducted with low intensity dynamic sources, often represented by 'ambient vibrations', are an innovative and useful instrument [8-9]. Obviously, only the linear response of the structure may be detected. In-situ tests with extraction of core samples from the masonry walls are, indeed, not always allowed for heritage buildings or should be limited to few points. Dynamic *in-situ* tests can allow the definition of reliable model of the structure, in terms of elastic constants and density of the material, masses and restraints.

2. Case-study: Palazzo dell' Aquila Bosco Lucarelli

Palazzo Bosco Lucarelli is a masonry building in historic center of Benevento (Italy), (figure 1).

Fig.1. Palazzo dell'Aquila Bosco Lucarelli: Façade, perspective and aero-photo

The original nucleus of the present building goes back to the XI century; after the strong earthquake of 1702, the existing building was rebuilt and amplified assuming the current structure with an internal closed court. The building has a rectangular holed floor plan, with an underground floor, not completely accessible, a ground floor, two upper levels, and an attic under the pitched roof. The largest dimensions in plan are 33.5 m and 26.3 m and the total height is 18.2 m. The floors are made of standard 'I' shaped steel profiles with height of 200 mm spaced of 0.85 m, interspersed with brick tiles and covered by a concrete deck 100 mm thick. The ceilings of the staircase and of some rooms at the ground floor are made of masonry vaults. The roof is made by a steel truss with a profiled steel sheeting externally and covered with brick tiles. The thermal-physical properties of the opaque envelope are described in the following sections. Windows have double-glazing 3-6-3 uncoated, air filled, with clear glasses. The ruined wood frame doesn't allow air-tight. The thermal transmittance of the whole window is around 3.1 W/(m^2 K).

3. In-situ investigations

3.1 Mechanical properties of masonry

The *in-situ* relieves revealed a masonry structure with different materials, thickness (between 0.6 m – 1.0 m) and textures. In particular, at the underground and ground floors the walls are made of irregular blocks of limestone and conglomerates with some inclusions of tuff or clay bricks. At the underground level, the blocks are greater and more irregular. At the first and second floors, walls are made of clay bricks with regular texture. These levels were probably realized when the building was expanded at the end of XIX century. All walls are covered on both sides by a reinforced plaster (thickness 50 mm) with a steel grid (bars with diameter 4 mm and spaced of 150 mm).

Endoscopic relieves evidenced a continuous texture also the wall thickness. Penetrometer tests on the mortar gave a compressive strength of about 0.65 MPa for the ground floor and 0.40 for the superior levels; these results allow classifying the mortar as a poor typology [10]. Lacking experimental data about the mechanical properties of the whole masonry, the compressive strength, f_m, the shear strength without vertical loads, $\tau_{o,m}$, and the tensile strength ($f_{tm}=1.5\ \tau_{o,m}$) have been assessed basing on [10-12]. In particular, the masonry of the ground and underground floors has been identified in the category "*masonry chopped stones with a good texture*", while for the higher levels of the building, the masonry has been identified as made of "*clay bricks with lime mortar*". Considering an amplifying factor of 1.5, due to the beneficial effect of the reinforced plaster and a reduction factor of 1.2 for the strength due to intermediate level of knowledge with the *in-situ* surveys, f_m results 2.2 and 2.9 MPa, $\tau_{o,m}$ 0.08 and 0.095 MPa, f_{tm} 0.122 and 0.143 MPa, for the masonry of ground floor and for the upper levels, respectively.

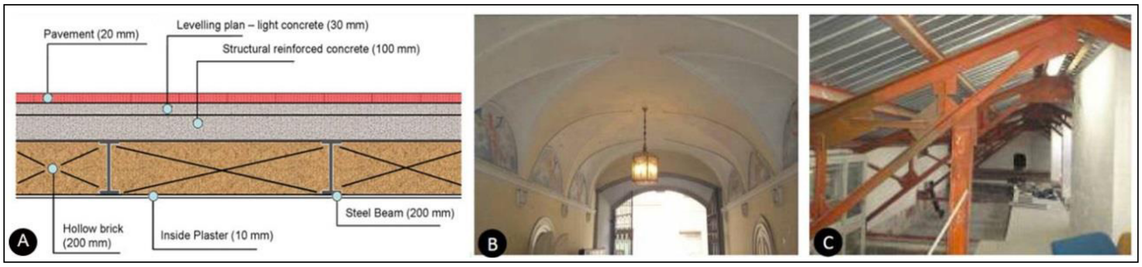

Fig.2. a) Schematic draw of the floor; b) Vaults of the closed court at the ground floor; c) Steel truss on the roof

Basing on the unit weights of the single materials (26 kN/m^3, 17 kN/m^3 and 19 kN/m^3 for the calcareous stone, the bricks and the mortar, respectively, and 24 kN/m^3 for the reinforced plaster) and on the masonry texture, the unit weight has been estimated about 20 kN/m^3 for clay bricks masonry and 24 kN/m^3 for the masonry of the ground floor. No experimental evaluation of the Young's modulus, E, has been still developed and, thus, the ranges suggested by codes [12], as function of masonry type, have been examined. For the Young's modulus, an amplifying factor of 1.5, due to the presence of the reinforced plaster, can be used. Thus, considering an intermediate level of knowledge that leads to assume the mean values of the ranges, the values of 2610 MPa and 2250 MPa have been assessed for the masonry of the ground floor and for the upper floors, respectively. These values refer to an un-cracked condition.

3.2 Dynamic identification of the building

Besides the mechanical tests, some information on the structure has been obtained by using the outcomes of a dynamic monitoring system, installed since 2008 by the Italian Department of Civil Protection. The system is aimed to provide real-time information both on the entity of eventual seismic actions and on the response of the structure to usual and less severe dynamic actions.

Among different methods proposed in recent years for identifying natural vibration modes of a structure, one of the most diffuse signal processing is the Operational Modal Analysis (OMA, [13]). The most known and simple is the Peak Picking Method (PPM), where Fourier transforms of the recordings are carried out and frequency peaks are detected on the Fourier amplitude spectra. If the mode shapes of the structure are well separated, the PPM may provide reliable indication on the dynamic parameters [5].

The monitoring system is made of nine couples of mono-axial accelerometers placed at the three floors of the building and a three-axial accelerometer located at the ground floor. Each couple of sensors is able to measure separately along the X and Y direction. The sensors have been aligned as accurately as possible along three vertical lines: one corresponds approximately to the centre of the building and the other two to the external edges. Different sets of accelerometric recordings were done both under ambient noise and impulsive sources [5]. The eingenfrequencies have been identified basing on the graphs of the FFT functions of the accelerations, by considering the relevant peaks in acceleration (Peak Peaking Method). In Table 1 the mean values and the Coefficient of Variation (CoV) of the first two frequencies identified considering the registrations obtained under both environmental noise and the impulsive source produced by the fall of a concrete block close to the building are listed.

The first frequencies identified in both X and Y direction are low scattered among the measures of the nine sensors and under different dynamic sources (CoV < 3%). The first frequency in X direction is lightly greater (10%) than in Y direction (4.7 vs. 4.2 Hz) that is indicative of a greater deformability of the building in the direction Y. To corroborate this assumption it can be observed that along the shorter side (Y) the building has a lower amount of masonry walls, often not continuous, compared with the longer side (X, see plans in figures 5). The values of second frequency in X and Y direction have been identified only by few sensors and are characterized by higher scatter since they probably refer to not significant modal shapes with low mass participating ratio, as it will be evidenced by the numerical analyses too.

Table 1 - Values of the first eingenfrequencies identified by in situ dynamic tests

Mode	Direction	Environmental noise	Concrete block	Mean (Hz)	CoV (−)
I	*Y*	4.2	4.2	<u>4.3</u>	3%
II	*X*	4.6	4.7	<u>4.7</u>	2%

3.3 The “energy” characterization

The studies of the mechanical properties allows the determination of thermal behaviours, because some characteristics have effects both on mechanical and energy properties. More in detail, endoscopies and core samplings allowed the knowledge of the stratigraphy and the analytical evaluation of thermal parameters, according to ISO EN 6946 [14]. These values was compared with measures by means heat flow meters, according to ISO 9869 [15]. In the following lines, the analyses on the external wall (figures 3) of the first/second floor (a) and on the floor between the second floor and the attic (b) are described. Also IR thermography was performed to identify the thermal bridges and the main heat losses.

3.3.1 Experimental investigations by means of the heat flux meter

The measured data have been evaluated according to “Average Methodology" proposed by [15]:

a) <u>Vertical wall</u>: a monitoring period equal to 5 days has been established, with acquisition time-step of 30 minutes. The measured thermal conductance is around 0.92 W/(m^{2}K). Considering the surface heat transfer coefficients suggested by [14] - inner surface conductance 7.7 W/(m^{2}K); outer surface conductance 25 W/(m^{2}K) - the value of thermal transmittance is 0.80 W/(m^{2}K).
b) <u>Floor Structure:</u> the measured thermal conductance was around 3.15 W/(m^{2}K). According to [14], with reference to indoor environments significantly ventilated, a surface conductance equal to 12 W/(m^{2}K) should be used. Finally, the thermal transmittance is around 1.90 W/(m^{2}K).

3.3.2 Analytical investigations through data derived by endoscopy and core sampling

The experimental measures have been compared with the analytical calculations.

a) Vertical wall: three layers compose the wall (*1. indoor reinforced plaster, 2. clay-made main masonry, 3. outdoor reinforced plaster*), with known thicknesses. Thermal conductivities were determined according to material typology and thus the thermal transmittance was 0.87 W/(m^{2}K).
b) Floor Structure: a rigorous calculation of the mixed layer "hollow bricks - steel beams" was done, by contemplating a wide portion of structural element and dividing the mixed layer into 3 different ideal sub-structures. Finally, the achieved thermal transmittance of the roof, analytically evaluated and inclusive of internal and external heat transfer coefficients, is equal to 1.83 W/(m^{2}K).

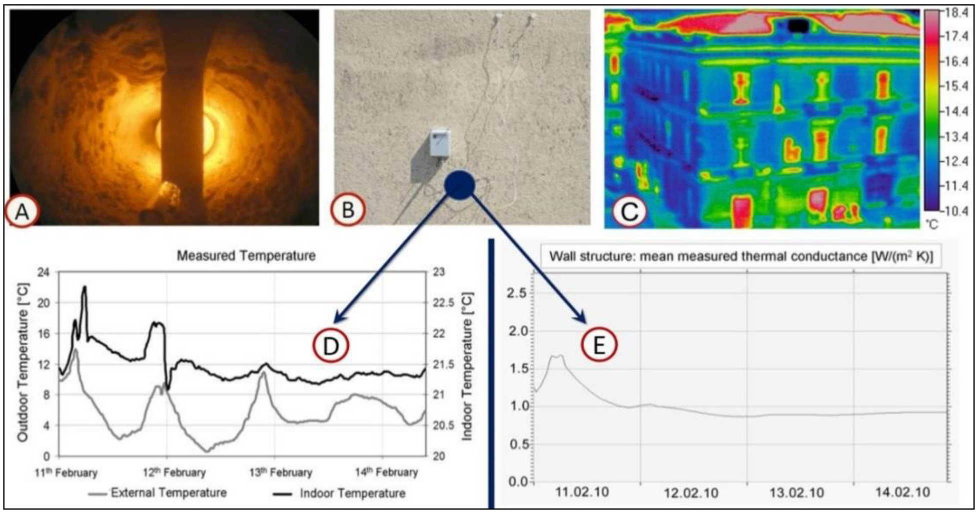

Fig.3. a) Corse sampling, b) Installation of flow meters, c) thermography, d) monitoring temperatures, e) conductance measures

3.3.3 *Comparisons between experimental and analytical investigations*

For both the examined envelope structures, a very satisfactory crossed comparison has been obtained:

a) Vertical wall: The analytical calculation provided a U_{value} around 0.87 W/(m^{2}K), while the in-field analysis by heat flow meter gave a U_{value} equal to 0.80 W/(m^{2}K). It means a gap around 9%.
b) Floor Structure: The calculated U_{value} is equal to 1.83 W/(m^{2}K), while the in-field analysis by heat flow meter provided a thermal transmittance around 1.80 W/(m^{2}K), with a difference around 4%.

4. The models of the building

4.1 The structural modeling of building

Based on the geometrical relief of the building, three-dimensional models (figure 4) - made of 3-Dimensional 'brick' elements with rectangular shape in the (X, Y) plane – have been realized with the software DIANA TNO. The structural modelling of the structure is aimed to: 1) identify numerically the dinamic behaviour of the building and compare it with the experimental outcomes, 2) estimate the safety level of building, and 3) check if an improvement of the global performance is reasonable in the framework of both energetic and structural behaviour.

In the plane (X, Y) the elements have a mean dimension of 0.3 m, while along the Z axis the greatest dimension is 1.0 m. The three floors are modelled by introducing a stiff constraint at each level and applying the loads of the floors directly on the walls. According to a simplified approach, the presence of the underground level has been neglected in this phase, because of the uncertainness about the dimensions of the walls, the foundation types and the interaction with the lateral soil and the sub-soil; thus, the building has been considered restrained at the ground floor level. Under gravity loads, the self-weight was considered in addition to the dead $G_k = 5.40$ kN/m^2 and the accidental loads $Q_k = 3.00$ kN/m^2 (estimated for eventually crowded ambient, [10]) for the intermediate floors and $Q_k = 0.50$ kN/m^2 for the level under the roof. The weight of the steel roof has been estimated as 1.75 kN/m^2. The reduction factor of accidental loads under the seismic loading condition is $\psi_2 = 0.6$ [10, 16].

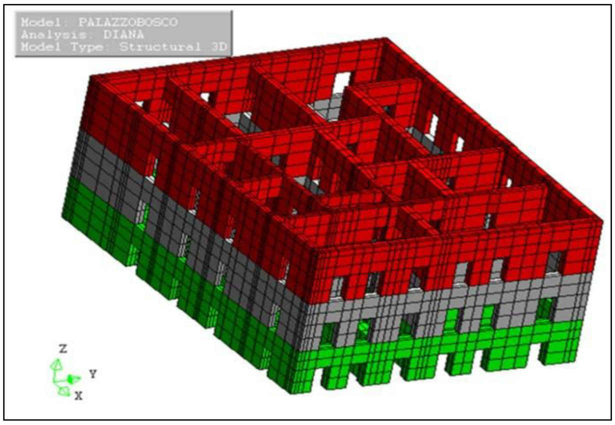

Fig.4. 3D FE Model of Palazzo dell'Aquila Bosco Lucarelli (*no modeling of the underground floor*)

About the seismic actions, the Eurocode 8 [16] allows choosing different occurrence periods. The current Italian code [10] for existing structures provides that it is sufficient to satisfy verifications at the Limit State of Life Protection (LS LP). The LS LP has a return period of 475 years, corresponding to a probability of exceedance for the peak ground acceleration, a_g, of 10% in 50 years. For the examined site $a_{g,475}$ is 0.262 g [17]. The reference life of the building has been assumed $V_R = 50$ years for a building with a using class II [10] (i.e., the building has not any strategic role during an earthquake).

4.2 Model for the energy performances

All the real available data have been coupled to the analyses of the historical energy requests in order to calibrate a reliable energy model of the building. More in detail, the historical energy consumptions have been analyzed, by averaging the monthly billings of electric energy and the natural gas for the period 2005-2009. Referring to the electric energy demand, a quite constant monthly demand around 9000 - 9500 kWh has been noted. Using a conventional lower calorific value for the natural gas equal to 9.59 kWh/m^3, the calculated specific heating demand, in terms of primary energy, is around 9.84 kWh/m^3 (with a net heated volume of 11'150 m^3). A building analogous, as regards geometrical dimensions and shape (Surface to Volume Ratio around 0.3 m^{-1}), should require no more than 7.15 kWh/m^3 according to the Italian Law. Thus, the actual energy performance is not very poor.

Moreover, typical load profiles crossed with singular census have been used for modeling the various hosted uses (i.e., offices, classrooms, laboratories etc). The energy demand of the ground floor, simulated only in order to define proper boundary conditions, have been neglected, being used not by the University. Typical air-change rate around 6 l/s per person has been considered, beyond the undesired infiltration through windows and doors, around 1.2 ACH. In summer time, during the HVAC running, air temperatures around 24 °C have been monitored, while, in winter, the air temperatures are equal to 22 °C during the diurnal hours of week days. Therefore, these set-points have been considered in the energy calculations. About the length of the heating season, the conventional heating period has been considered, while the cooling season was determined according to University policy (i.e., 1st June - 31th September).

5. Predictive reliability of the models

5.1 Analysis of the dynamic behavior of the building

The linear modal analyses have been developed considering all the vibration modes characterized by a participating mass ratio greater than 5%. The analyses show that the first and second eingenfrequencies are related to modal shapes completely translational in direction Y and X (figure 5), with a participating mass ratio of about 75% in both cases.

Several numerical analyses have been carried out by changing the values of masonry Young's modulus. The theoretical frequencies that are more similar to the mean experimental ones (differences about 3%) result 4.36 Hz and 4.87 Hz for the first frequency in Y and X direction, respectively, obtained assuming $E_G = 1450$ MPa for the ground floor and $E_{1\text{-}2} = 1250$ MPa for the upper levels. These values are about 55% of the uncracked mean values (2610 and 2250 MPa). Usually, code suggests [12] to reduce the Young' modulus of masonry of 50% to consider the cracking. Also the numerical results evidence that the first eingenfrequency (direction Y) is about 10% lower than the second (direction X), thus evidencing a tendency of the building to be more deformable in the direction Y. Conversely, the third, fourth and fifth modes are local and are characterized by mass participant ratios lower than 5-10% and, thus this confirms the difficulties into identify them by means of ambient dynamic in-situ tests.

Several dynamic analyses have been carried out also in [5] by a different Finite Element model made of 2D shell elements and taking into account the interaction with subsoil by means of concentrated springs. These analyses led to estimate little higher values of E (1875 and 1680 MPa for the ground floor and for the upper ones, respectively) for achieving numerical frequencies close to the experimental ones; this is due to the different type of Finite Element used and to the additional deformability introduced by the springs simulating the subsoil effect.

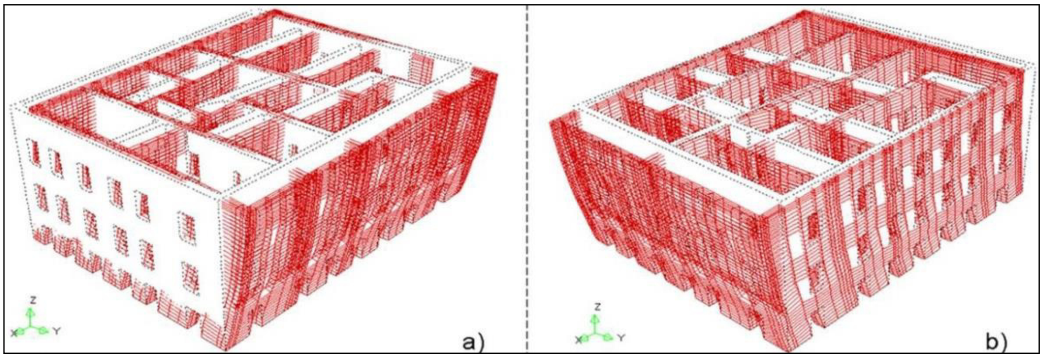

Fig. 5. Modal shapes given by the 3D model for the building: a) Mode I (Y direction), b) Mode II (X direction)

5.2 The structural safety of the building

The evaluation of the structural safety level under seismic actions has been carried out by a linear dynamic approach, according to the spectrum analysis provided by European and Italian codes [10, 16].

The building has been modelled without the underground level and the total seismic mass is about 68700 kN. The values of unit weight assessed in section 3.1 and the values of Young's modulus assessed in section 5.1 ($E_G = 1450$ MPa and and $E_{1\text{-}2} = 1250$ MPa, see) have been used. The analysis of the building under only gravity loads has estimated average compressive stresses equal to about 0.45 MPa, 0.35 MPa and 0.20 MPa for the ground, the first and the second floor respectively. These values are in good agreement with the ones calculated by simply dividing the mass to the area of the resistant walls at each floor (0.41 MPa, 0.26 MPa and 0.14 MPa). These values are sensibly lower than the strength (2.2 and 2.9 MPa), that, in case of linear analysis, has to be further divided by the safety factor (for masonry structure is at least 2 [10]). This means that, under gravity loads, the minimum safety factor (achieved at the ground floor) is about 2.5.

The dynamic linear analysis under seismic actions has been developed adopting the 32 loading conditions indicated by [10] taking into account the accidental eccentricity in both directions X and Y. The q-factor has been assumed equal to 3.6, as suggested for masonry structures [10]. This value is safe compared with the results obtained in detailed not linear analysis [5].

The most stressed masonry walls are signed in figures 6 for each floor; furthermore, the overall values of shear and flexural moment, M_s and V_s, have been calculated by integrating the correspondent stresses. The worst loading condition is represented by the combination of the minimum axial load, P_s, with the greatest values of shear, V_s, and bending moment, M_s. The resistant values of shear, V_t, and flexural moment, M_u, are calculated according to the code provisions of [10,12].

The lowest safety factors, γ_s, (minimum between M_s/M_u and V_s/V_u) are in general related to the shear failure and are achieved in some walls parallel to the direction Y. In particular, the walls result all verified with exception of some panels at the 2nd floor (four panels with $\gamma_s = V_s/V_u = 0.8$-0.9 and a panel with $\gamma_s = M_s/M_u = 0.6$); this is probably due to the lower axial load at such a floor. However, with exception of these few cases, the safety factors are meanly in the range 1.3-1.8 for the shear and 1.3-2.4 for the flexural failure. These results evidence that under the expected seismic action the building is safe, if some local interventions for strengthening the deficient panels and for improving the global behaviour in terms of both strength and ductility will be realized. It is worth to note that the structural safety has been evaluated by means of a linear dynamic approach that is a safe approach.

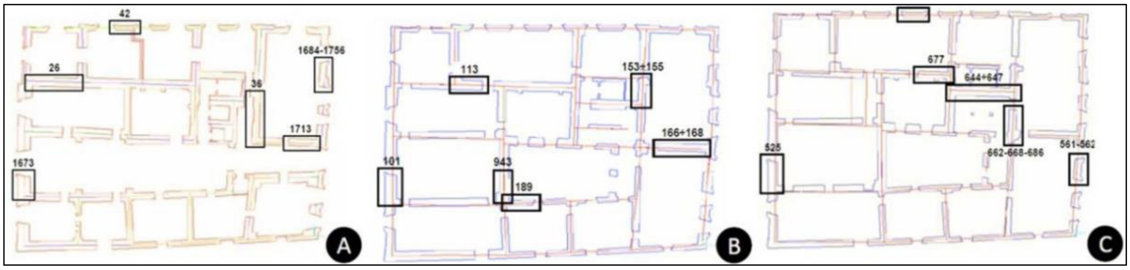

Fig. 6. Masonry walls subjected to verify: a) Ground floor, b) First floor, c) Second floor.

5.3 *Energy analysis: comparison between results of the dynamic simulation and measured requests*

EnergyPlus [18] was used for simulating the energy performance of the building, by adopting transient heat transfer algorithms. The real building and the modeled one are represented in figures 7a and 7b.

The outcomes of the energy simulation, compared to the consumptions derived by the energy billings, are reported in figure 7c and 7d, with reference to the primary energy need for the space heating and the electric energy requests, respectively. Similar trends can be noted. As regard the heating demand, the highest discrepancy is registered in November. However, the high percentage gap is due to the quite low absolute requests. Globally considering the whole year, the distance between the calculated gas use and the measured one is very low (figure 7c), around 3%. The specific primary energy demand for the space heating is not so excessive, being the EP_i = around 9.84 kWh/m^3 by simulation, 10.13 by gas billing. The achieved energy indicators are not particularly high, because of the significant thicknesses of the envelope structures, so that also in presence of conductive materials, the overall thermal resistance is quite satisfactory. Very similar trends are shown in figure 7d as regards the electricity demand. The slightly higher percentage gap (around 4%) is due to the several uses of electricity that imply higher simulation errors.

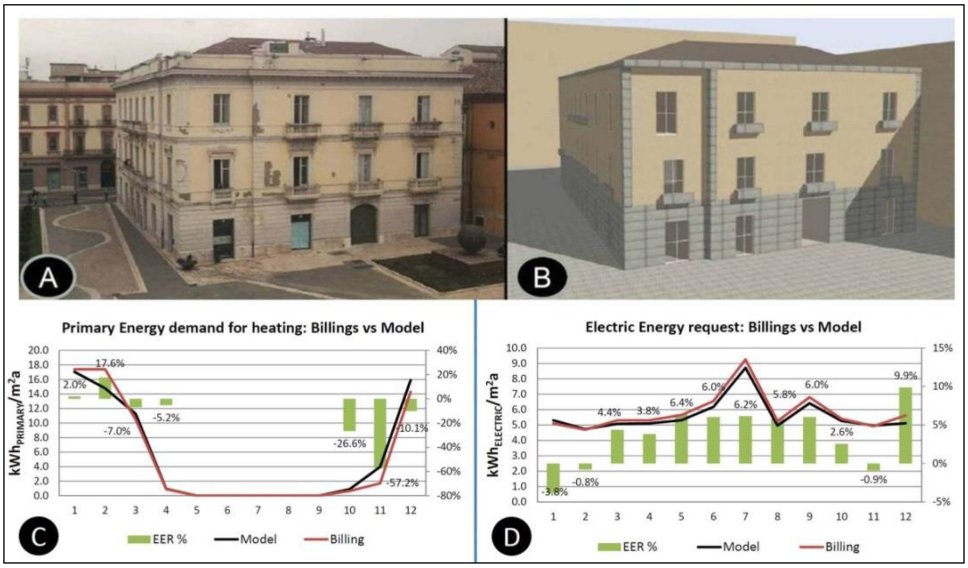

Fig. 7. Building model for the energy audit (A and B) and comparisons between measured data and the simulation results (C and D)

6. Conclusions

This study refers the main outcomes of multidisciplinary investigations performed on an historical building located in Benevento, aimed to evaluate both structural and energy performances, by coupling several diagnostic methodologies, and recurring to in-field investigations. Indeed, the joint of the diagnostic activities is an opportunity useful especially for optimizing costs and execution time of the *in-situ* tests, and for performing synergic evaluation of various outcomes - in terms of structural safety and energy performances.

The structural FE model, well-assessed especially on the results of dynamic *in-situ* tests, allowed checking the safety level as regards the vertical loads and the seismic actions indicated by the national codes. In particular, the verifications of the structural elements carried out under the expected seismic actions according to a linear approach (linear dynamic analysis), which does not consider the reserve of the building in the not linear field, are not safe for few panels. However, the structural safety can be guaranteed by few local interventions aimed to strengthen the deficient panels.

As regards the energy diagnosis, core-sampling, endoscopy, measurements by heat flow meters allowed a proper energy characterization of the building envelope. Analogues investigations have been performed with reference to the HVAC systems. The acquired knowledge allowed the building modelling and simulation, then calibrated on the basis of the energy billings. The building energy performances are strongly improvable.

Finally, the results obtained as regards the two analysed topics, indicate a satisfactory global performance of the building and that a plan of rehabilitation of the building is surely promising for improving the global building performance integrating both energetic and structural items.

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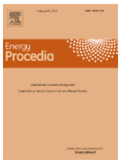

Biography

Rosa Francesca De Masi - Ph.D. in Engineering of Mechanical Systems - is Grant Researcher at the University of Naples Federico II, for the Italian Scientific Sector ING-IND/11 (Applied Thermodynamics and Heat Transfer for Buildings). She develops researches about the evaluation of energy performances of the system building-HVAC devices through analytic and numerical approaches.