The dynamic behavior of the Basilica S. Maria di Collemaggio before and after the 2009 L’Aquila earthquake

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ABSTRACT: The dynamic behavior of the Basilica S. Maria di Collemaggio has been studied since 1988 by researchers of University of L’Aquila by both experimental testing and numerical models. This activity has allowed to build reliable numerical models of the structure in the undamaged configuration before the 2009 earthquake and to interpret the collapse of the transept during the seismic action. Nowadays, the actual configuration of the monument, with a series of temporary scaffolding inserted in the seriously damaged structures, is the object of permanent monitoring, both for the static and dynamic behavior. The paper summarizes the results of numerical investigations devoted to the simulation of the dynamic behavior of the Basilica under the 2009 seismic actions. Results of static and dynamic analyses conducted by FE element models with increasing complexity and computational efforts are used to evaluate the role played by the three-dimensional mechanical behavior, especially in the collapsed transept area. Main seismic vulnerabilities of the monument are discussed. The investigations are integrated with the results obtained through the permanent structural monitoring installed after the earthquake and continuously enhanced.

KEY WORDS: Monumental structures, masonry, seismic analysis, dynamic testing, nonlinear analyses, finite element modeling.

1 INTRODUCTION

Monumental churches have an inestimable value for the world cultural heritage. For this reason, continuous efforts are devoted to understanding their structural behavior. Nonetheless, monumental churches quite often undergo structural failures due to both static conditions and dynamic forces, sometimes with catastrophic consequences; and lengthy reconstruction works often leave the seismic vulnerability issues unsolved [1].

Because of this, great efforts are being made to comprehend the structural behavior of historical and monumental churches, mainly in Europe, with attention both to understanding cases of structural damage resulting from specific static conditions [2,3] and to evaluating vulnerability to earthquake-induced activity, if located in an active seismic area [4,5]. Due to the inherent complexity of tackling the physical problems, whether this means reconstructing the reasons for structural damage appearing under usual conditions (caused by permanent loads, boundary displacements slowly evolving in time and seasonal thermal changes) or predicting dynamical behavior under rare circumstances, such as those caused by earthquakes, the structural analysis of monumental churches is often accompanied by the extensive use of modern non-destructive testing techniques which permit the acquisition of valuable information at both the local (material) and the global (structural) levels.

Notwithstanding the common framework used by researchers in this specific field, the unique features of each church require the adoption of specially-tailored and well-balanced approaches in all the aspects and activities of the analysis. This includes historical investigations devoted to understanding the specifics of how the structure has evolved over the course of past interventions (such as are often performed on ancient monuments), the geometrical and material characterization, the information received by global dynamic testing (if performed), and the structural analysis and modeling. Moreover, the structural assessment and retrofitting of important churches in L’Aquila, some of them having a monumental value, has been a deeply studied theme. Results concerning the seismic behavior of masonry churches have shown that the dynamic excitation due to the seismic ground motion activates many vibration modes of the structure, though all of them are characterized by small participation factors. Therefore, churches are not behaving through a superposition of global modes but more as dynamic interaction of localized modes. Consequently, in many examined case studies, the ratio between the total base shear and the church total weight ranged between 20% and 30% evidencing a reduction with respect the plateau value of the spectral acceleration provided by Italian Code. Therefore, appropriate choices of the force reduction factor should be adopted for these monumental buildings different from the case of traditional residential buildings characterized by shear type behavior. Further, the activation of many local modes also calls for retrofit interventions, which should “tie up” the building, thus avoiding the local failure modes that are often observed [6]. Consequently, the process of preserving historical monuments today still remains a challenging issue [7-11]. The occurrences of failures are particularly important to study, in order to bring the critical points in the overall process into focus, and to clarify how to enhance optimal and sustainable paths of investigation and intervention [1, 12, 13].

The present study collects most of the information available on the structural behavior of the church in order to present a possible scenario of its seismic behavior. Several finite element models have been developed for this purpose, many
of which take advantage of information acquired during different dynamic test campaigns. Seismic actions are described either through the national Italian code or using the registered main shock of the April 6, 2009 earthquake recorded at the station, AQK, located close to the Basilica.

The effects of the modeling of masonry elements, such as the vaults and the dome in the transept by bi-dimensional or three-dimensional finite elements are discussed.

Spectrum linear analysis and nonlinear static analysis have been performed to determine the range of possible values for the seismic response of the masonry structures, considering all the limiting hypotheses typical of this kind of analysis. The results obtained have been used, in combination with the authors’ engineering expertise, to explain the observed partial collapse of the Basilica that occurred during the earthquake.

2 THE BASILICA S. MARIA DI COLLEMAGGIO

The Basilica is placed on top of a hill in the city of L’Aquila, just outside the ancient walls. The Basilica’s facade, made of pink and clear stones arranged to form crosses, is beautifully adorned with three rosettes and three Romanesque-Gothic portals (Figure 1).

The hall of the Basilica has a nave and two side aisles. The nave measures 61×11.3 m. The two side aisles measure 61×8 m (to the right of the nave) and 61×7.8 m (to the left). The maximum height of the nave is 18.3 m; each of the aisles is 12.5 m. The octagonal columns separating the three naves are seven on each side. The columns have an approximate relative distance of 7.5 m, a height of 5.25 m, and are about 1.0 m in diameter. They hold a total of 16 ogival arches. The double layer walls are for the most part constructed of two distinct external panels, well-spaced, with a filling of small stones realizing the so-called rubble masonry.

The transept of the Basilica, the area set crosswise to the nave, does not completely produce the so-called “Latina cross shape” because the transversal parts, or “arms,” are not extended across the external longitudinal walls. Thus the main part of the church presents a long and slender longitudinal shape before reaching the sanctuary area. The construction of the Basilica began in 1287. In later centuries, the structure has undergone many alterations and continuous renovations [14], in part as a result of the frequent earthquakes that have affected the territory of L’Aquila. In the seventeenth century the church, of medieval origin, underwent renovation in the baroque style in its inner parts. After the earthquake of 1703, the longitudinal walls had partially collapsed and were lowered. The earthquake in Fucino, in 1915, significantly damaged only the upper left corner of the facade. In subsequent years, interventions were made to improve out-of-plane stiffness and resistance: a reinforced concrete grid, a leaf of brick masonry at the rear of the facade, and two reinforced concrete spurs on longitudinal walls.

In the early 60s, the demolition of the dome above the transept, heavily damaged by the previous earthquake, was concluded, and a new one, made in reinforced concrete, was rebuilt with a similar shape. Between 1970 and 1972 the church was restored to its medieval appearance, laying bare the ancient octagonal columns and pointed arches, and increasing the height of the longitudinal walls (for the lateral wall Δh = 3.15 m; for the central wall Δh = 3.65 m).

2.1 The structural system and its modal characteristics

The structural system of the Basilica is the result of its peculiar geometry and the material characteristics of the masonry. The monument was composed by three main subsystems: (i) the nave which are closed transversally by the facade and covered by a light wooden roof; (ii) the transept composed by the main arch, two barrel vaults, a central main dome and four pendentives; and (iii) the chancel, the sacristy and the apse, the dome, and the main arch closing the nave. The masonry walls, which delimit the space of the naves and the transept, are quite slender. In the masonry it is possible to recognize different patterns due to successive reconstruction phases, which generally occurred after partial collapses caused by earthquakes (Figure 2). The structural dynamics of the Basilica has been the object of studies in recent years. In particular, dynamic testing has been conducted, at small oscillation amplitudes, to characterize the dynamic behavior of the slender macro-elements, the nave walls and the facade, in order to extract the main modal signature of the system.
Vibration tests were performed using an instrumented hammer and a vibrodyne [14]. In a updating model, identified modal characteristics have been compared to those obtained by finite element models to produce a reliable representation of the whole church dynamics for seismic behavior prediction.

Along these lines, FE models of the whole church, in different environments (SAP2000, MIDAS), have been developed taking into account the most relevant macro-elements of the Basilica with different levels of description either in the geometric assumptions or the constitutive laws characterizing the material behavior of the masonry. Two-dimensional and three-dimensional finite element models were used in the linear and non linear structural analyses. In constructing the refined models, attention has been devoted to fit the dynamic features identified from the experimental measurements. In particular, at low frequencies, the local modes involving mainly the nave walls have been considered, at higher frequencies, the local modes representing the out-of-plane facade oscillations were obtained.

The mechanical parameters of these models have been updated by minimizing the errors between experimentally-determined modes and frequencies and the numerical ones. The results obtained are shown through Figure 4 in which two selected modes of the global model of the church involving mainly the walls of the nave are in good agreement with the ones identified during the dynamic test campaign [14]. In the updated model, modes involving mainly the facade can also be recognized, starting from the tenth one. Table 2 shows a direct comparison between the main natural frequencies of the model and those detected during the dynamic test campaign. The errors are very small in all the available identified modes captured during the testing and therefore the selected model can be considered representative in the range of small oscillations. However, this updating process may be significant in the construction of a reliable nonlinear finite element model able to represent the dynamic behavior of the Basilica under strong earthquakes, such as the one occurring in 2009. Two nonlinear models have constructed, the first one uses mainly bi-dimensional elements (Fig. 3b), while the second one is made of three-dimensional solid elements (Fig. 3c), both preserve the modal characteristics of the previous linear one (Fig.3a).

<table>
<thead>
<tr>
<th>Mode</th>
<th>I mode</th>
<th>II mode</th>
<th>III mode</th>
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<td>2.12</td>
<td>2.60</td>
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<tr>
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<td>2.47</td>
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<tr>
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<td>2.46</td>
</tr>
<tr>
<td>3D NL-FEM</td>
<td>1.45</td>
<td>2.33</td>
<td>2.65</td>
</tr>
</tbody>
</table>

3 STRUCTURAL ASSESSMENT

The structural assessment of historic monumental buildings in seismic area is a challenging matter. Even though continuous efforts are done in this thematic area we are far from possessing a full and well categorized procedure, retained valid by all the different specialists involved in the problem. Furthermore, in Italy, as in other seismic countries, the seismic code has evolved continuously in response to the dramatic events that have occurred in recent years. Due to the extensive damage caused to the Italian monumental heritage in all the cases mentioned, increasing attention has been devoted specifically to the issue of the seismic vulnerability of monuments. Within this complicated matter, the main findings of the conducted structural analyses are here summarized.

3.1 Static and design spectrum response analyses

Static analysis of an ancient monument may appear a minor task in the structural assessment of monumental building in seismic area where the main concerns are related to the horizontal loads, which are detrimental for masonry structures. However, the prediction of the stress state in ancient masonry elements, generally induced or removed and induced again along the years, may be a useful indicator of the successive seismic vulnerability. In the studied case, particular attention is due to the transept and specially to the pillars sustaining the vaults and the dome. Figure 5 reports, synthetically, the results of two stress analyses induced by live loads conducted by using three-dimensional finite elements (Fig. 5a,b) or bi-dimensional ones (Fig. 5c,d). It is evident as the stress flux is more clearly defined in the arches and in the pillars by the more refined model, even if the level of stress amplitude is comparable in the two cases. At the same time the analysis evidences that in the pillars is present an important flexural deformation induced by the differences of thrusts in the longitudinal direction between the arches belonging to the nave walls and the arches of the transept on which are laying the dome and the vaults.
The 2009 L’Aquila earthquake is an event deeply studied by the scientific community due to the extensive amount of available data [16]. The event caused an up-dip slip movement with the main shaking registered by 55 stations of the National Accelerometric Network. The highest accelerations were recorded at the AQK station, with values of $\text{PGA}_{\text{EW}} = 0.67\, \text{g}$, $\text{PGA}_{\text{NS}} = 0.56\, \text{g}$ and $\text{PGA}_{Z} = 0.52\, \text{g}$. At AQK station a higher value of the vertical acceleration ($\text{PGA}_{Z} = 0.35\, \text{g}$) relative to the horizontal ones ($\text{PGA}_{\text{EW,NS}} = 0.34\, \text{g}$) was found. In order to analyze the characteristics of the earthquake, reported in Figure 6 in relation to its effects on the Basilica, the acceleration motion trajectory (Fig. 6a) and the elastic response spectrum (Fig. 6b,c) of the AQK natural records were calculated. It should be noted that the recording station has a distance from the church approximately equal to 380 m. This proximity permits the possibility to relate the observed damage to the registered accelerations.

At AQK station, located in stiff soil, the maximum recorded peak ground acceleration in the NS component is 0.35 g. It must be emphasised that L’Aquila was located in the vicinity of the normal fault, and because of that the recorded vertical acceleration (UP) of 0.37 g is slightly bigger than the horizontal one in the NS component. It is interesting to compare the recorded accelerations with those provided by the Italian code for buildings at L’Aquila city. Fig. 6b represents the elastic response spectra for the two horizontal components of the accelerations recorded at AQK (5% damping ratio), compared with elastic spectra provided by the Italian code, while Figure 6c is related to the vertical (UP) component. The response spectra of the horizontal EW time history presents the highest values at the period $T=0.18\, \text{s}$, with acceleration $1.17\, \text{g}$, while the NS and Z (UP) components have a significant peak at the period $T=0.07\, \text{s}$, with acceleration 1.00g and 0.95g, respectively. The Italian code, using the return period of $T_r=475\, \text{years}$ and B-type soil, underestimates the spectral values of the recorded signals. Finally, the reported data evidence that the prevailing direction close to the site of the main shock was oriented in such a way that prevailing longitudinal actions in the church may have occurred (Fig. 6a) and the modes of the Basilica are in the frequency range where the earthquake had concentrated the most part of the energy (Fig. 6b,c).

Figure 5. Principal stresses due to static loads in the transept: 3D FE a) radial b) vertical; 2D FE c) radial d) vertical;

Figure 6. AQK registration of L’Aquila earthquake: a) acceleration trajectory; b) elastic response spectra (horizontal components) vs Italian Code provision; c) elastic response spectra (vertical component) vs Italian Code provision.

### 3.2 Comparison between code and registered earthquake response spectrum

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Notwithstanding the absence of earthquake data recorded at the Basilica basement, the available AQK measurements have been used to perform a design spectrum analysis taking into account the effective direction of the excitation with respect to the Basilica. Indeed, it should be noted that the inclination of the longitudinal axis with respect to the E-W direction is equal to 17° while the polarized direction of AQK is inclined of about 45° with respect to the same direction. Therefore, it can be estimated in 28° degree the inclination of the main shock with respect to the longitudinal Basilica axes. Consequently, E-W and N-S AQK spectra components are combined to take into account the earthquake direction with respect to the Basilica axis.

Details on the comparison of responses obtained by using the spectra combination provided by the code and the AQK case, are, here, omitted for sake of brevity; however the results show clearly that the previously selected prevailing longitudinal combination of the Italian code furnishes a seismic response similar to the one obtained by using the AQK spectra. Even if the seismic adequacy related to the prevailing longitudinal combination is greater than the one obtained considering the transversal action, the calculated adequacy levels are small, and therefore compatible with the damage observed at the site immediately after the event.

### 3.3 Seismic nonlinear static analyses

Thanks to peculiar aspects developed in the modelling environment used for the nonlinear analyses (MIDAS), a preliminary study has been conducted to evaluate the effects of orthotropic behavior of the masonry treated as homogenized media. An equivalent homogenized material based on a strain energy concept has been obtained [17]. A sensitivity analysis of moduli ratios characterizing the orthotropic behavior of the masonry, with respect to the mechanical and geometrical parameters of the material, has been conducted around reasonable values for the case of the Basilica. The analysis has shown the occurrence of small modifications with respect to the isotropic behavior. Therefore, with the aim of preserving the results obtained by the model updating process through the experimental dynamic, an isotropic behavior has been assumed for the nonlinear analysis.

In modelling the internal progressive damage during the inelastic behaviour induced by the earthquake two approaches (STRUctural MASonry, named STRUMAS [18] and Total Strain Crack, named TSC [19]) have been compared by a plane stress model for the Basilica nave wall containing the lateral entrance, the so-called “Porta Santa” - named macro-element D (Fig.3b) in the global model [14]. Moreover the STRUMAS approach permits a good evaluation of the macro-element limit shear force V, which is coherent with the ones evaluated in the TSC cases. However, the estimated limit value is reached without considering stiffness-degradation, which should be direct consequence of the cracking progression. This occurrence does not permit the direct evaluation of the ultimate displacement of the macro-element. The TSC computational effort is greater than the STRUMAS one, in which the obtained results are however comparable, therefore the nonlinear analyses for the global model of the Basilica (Fig. 7) have been conducted using this last approach and the main results are here synthetically described.

**Figure 7.** Varied configurations obtained by longitudinal pushover analysis with force distribution proportional to masses (+x dir).

Because of the adopted modelling approach, which is able to simulate both the in-plane and out-of-plane nonlinear behavior of the masonry macro-elements, it was possible to perform incremental static analyses either in both longitudinal and transversal directions. As in [4,6], through a spectrum modal analysis, the global model has also been used to evaluate the seismic demand in each macro-element. In the analysis, a large number of modes have been retained (up to 500). Seismic demands of each macro-element are determined assuming an earthquake coming only in longitudinal (x-direction) or transversal (y-direction) in order to compare these directly with the capacities evaluated through incremental static analysis in the same directions. In doing this, both the NTC and AQK spectra have been utilized, assuming that the NS component is completely transversal while the EW component is longitudinal.

The results of the linear elastic spectrum analysis are synthesized in terms of macro-element shear resultants in Table 2. It is pertinent to notice that there are not significant differences in the results using the spectrum given by the National code or the spectrum derived by the registration AQK, even in the total shear. Moreover, the results show again significant differences when a prevailing longitudinal or transversal earthquake hit the Basilica.

**Table 2.** Seismic demand: shear force resultants for each macro-element (see Fig. 3b) expressed as ratios with respect to weight (V/W) evaluated by elastic response spectrum analysis (AQK and NTC spectra).

<table>
<thead>
<tr>
<th>Macro-element</th>
<th>Weight</th>
<th>Longitudinal seismic action</th>
<th>Transversal seismic action</th>
</tr>
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<tr>
<td></td>
<td>% of total</td>
<td>AQK EW %</td>
<td>NTC %</td>
</tr>
<tr>
<td>A</td>
<td>17</td>
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<td>34</td>
</tr>
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<td>B</td>
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</tr>
<tr>
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<tr>
<td>E</td>
<td>5</td>
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<td>-</td>
</tr>
<tr>
<td>G</td>
<td>8</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>H</td>
<td>6</td>
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</tr>
<tr>
<td>facade</td>
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<td>17</td>
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<tr>
<td>Total</td>
<td>100</td>
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</table>
Indeed, the behavior under prevailing longitudinal seismic action permits engaging several masonry panels in their in-plane behavior, while in the transversal case, especially with regard to the nave walls, the seismic loads should engage their out-of-plane carrying capacity. The shear resultants evaluated by the elastic spectrum analysis evidence that in both directions (longitudinal and transversal) the total shear demand is about 35% of the total weight of the church. However, the greatest seismic demand concerns the central nave walls (B, C) in which resistance is required to balance 27% of the longitudinal action and 17% of the transversal action. Also, the external nave walls are strongly stressed by action coming in both directions (17% longitudinal and 12% transversal), while the apse walls (G, H) are working primarily under the transversal condition (2% longitudinal and 15% transversal).

In the evaluation of the limited capacity of each macro-element through the global model, incremental static analyses have been performed considering two different load distributions: concentrated forces at the element top and mass proportional. Figures 7 shows selected deformed configuration of the church under longitudinal or transversal horizontal actions with a distribution proportional to the masses at different load amplitude. The figures permit appreciation of the modifications in the shape of the deformation for increased loads (steps). Both in the longitudinal and in the transverse cases, the area of the transept is the one in which the shape change is most evident, confirming the localization of inelastic deformation and stiffness reduction. Figures 8 depict the distribution of the minimum principal stress in the nave walls named B (see Fig. 3b), obtained for different values of the increased longitudinal action in both positive and negative x-directions. Looking at both the stress distributions and the deformed configuration, the asymmetric behavior with respect to the sign of the horizontal action evidences that the longitudinal motion from the transept towards the facade (+ x dir) creates large deformations and stresses at relatively low levels of horizontal action. Moreover, in this situation it can be noticed that the arches connecting the large pillars closing the nave with the wall of the apse are affected by large tensile stresses in the area of the voussoirs at 30° (“reni”), even for small horizontal actions (step 1). Increasing the load, the area that overcomes the tensile strength ($f_t=0.3 \text{ N/mm}^2$), which is evidenced by black crosses, becomes larger, evidencing smaller cracks also at the interconnection between the nave wall and the facade.

This picture is truly representative of the mechanism of collapse in the transept area, as evidenced by the numerous previous observations regarding the damage scenario and the seismic action. A larger increase of lateral forces (step 7) will cause damage also at the nave wall. The observation of the behavior for horizontal forces applied from the façade towards the transept again shows, at step 4, the possible opening of cracks above the pillars in the transept arches.

Table 3 summarizes the load capacities of each macro-element of the church, obtained through the capacity curves, evaluating the shear value for a conventional displacement equal to 0.001 of the macro-element height [14]. Comparing these load capacities with the seismic demand on Table 3 confirms all the observations previously presented. In particular, the low ductility of the macro-elements B and C with respect to the actual and expected seismic action becomes clear. Also evident is the low load capacity of the nave walls with respect to transversal actions; this capacity is not completely null only thanks to a mechanism, which involves the longitudinal and the transverse walls all functioning together in a way that permits to carry larger horizontal forces.

4 DAMAGE AND MONITORING

The major damage suffered by the Basilica in 2009 was in the region of the transept (Figure 9) with a mechanism of implosion of all the structures that compose it: the great multi-lobed pillars, the triumphal arch and the wall above, the barrel vaults, the dome and the roofing structures. Observing the area immediately after the earthquake, the collapse took place without involving the exterior walls, not even their top parts, while a pile of debris fell entirely within the masonry walls. In particular, no debris was observed outside the external walls. These observations of a probable collapse mode shape suggest the likely sudden structural failure of the two large pillars which supported the structural system constituted by the triumphal arches, the pendentive supporting the drum and the dome above it, the barrel vaults, and finally the wooden roof on top of everything else.

It should be pointed out that the transept is realized through a structural system in which the arch and vault thrusts, induced by permanent vertical loads, play an important role in the transverse walls.
stabilizing role. During the 2009 earthquake, due to the proximity to the epicenter, the vertical and horizontal acceleration components had similar amplitudes, producing a potentially destructive destabilizing effect induced by large vertical upward accelerations. The intrinsic vulnerability of the transept area to earthquakes such as the one that took place on 6 April 2009 is evidenced by the fact that the recent transept collapse was probably similar to a collapse that had occurred previously in the fourteenth century.

The collapse, involving the first pair of arches of the nave and the walls above the first pillars, left the remaining part quite clean and sharp. The last columns of the nave were practically undamaged because, as previously illustrated, they were completely rebuilt in the restoration of the '70s, in distinction from the collapsed pillars. These pillars, in despite of their diameter dimensions, resulted to be weaker than what it was possible to evaluate without reliable data on the material characteristics of their core.

The interconnection between the orthogonal walls proved very ineffective, mainly due to the poor quality of the masonry, especially in the area above the arch and for the wall orthogonal to the nave in the plane of the triumphal arch.

Several of the threaded rods of the bracing system under the central nave roof were broken, almost all in the vicinity of the large crack on the north side wall of the church. In the north aisle, slippage of bolts designed to anchor the wooden trusses was noticed. Both of these cases of damage show the presence of transverse deformation of the longitudinal walls mainly located in the north. In the nave columns there were deep cracks due to heavy compression, initially appearing only in the central columns of the nave and then progressively emerging, with the after-shocks, in almost all the others as well. The area of the apse was also characterized by severe damage, affected by clear cracking in the mortar joints of the hewn stone blocks of the main apse, where there were also expelled blocks forming a wedge shape above the mullioned window. Observation of the interior reveals cracks in the apse vault, which is one of the most loaded elements, a partial detachment, and the presence of permanent relative displacements, which occurred in one of the ribs. More serious damage was suffered by the lateral chapels; in particular the Celestine chapel to the right of the altar exhibits three extensive sub-vertical cracks, one central and two almost symmetrical, starting from the intersection of the vault with the vertical walls and going down to the ground level.

The assessment of the changes occurred in structural behavior, together with the analysis of the structural modifications coming out from successive consolidation and restoration interventions, has been followed by a permanent monitoring system. The system development can be broadly divided into two distinct phases: the design and deployment of an accelerometric monitoring network and the design and deployment of a second network for wall inclination and crack width local measurement. The main goal was an accurate measurement of the building dynamic response, both to environmental action and to seismic events. The monitoring systems uses 16 Imote2, wireless sensing platform with SHM-A sensor board from ISHMP and Westmote custom developed sensing solution, to acquire the measurements from 16 LIS3DSH tri-axial MEMS accelerometers for acceleration measurements, 16 Sensirion SHT11 temperature and humidity sensor; 8 OTR OG400B crackmeter and 3 OTR OG307 inclinometer. Performance characteristics of MEMS accelerometer relevant to the present analysis are resumed in Table 4.

During the months following the installation, the monitoring system has been continuously enhanced and brought to complete and automatic management to sense seismic induced vibrations. During this path, test campaigns have been conducted with different induced source of vibrations such as hammer, ambient vibrations and free-vibration tests. To date, six major events have been detected. Three are associated with the Emilia region and three are events at L’Aquila. Table 5 shows the maximum accelerations registered during each earthquake. Recorded structural responses show prevailing out-of-plane oscillations of the nave walls.

### Table 4. LIS344ALH mechanical characteristics

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<td>Bandwidth</td>
<td>1.8 KHz</td>
</tr>
<tr>
<td>Noise Density</td>
<td>50 µg/√Hz</td>
</tr>
<tr>
<td>Non linear behavior</td>
<td>±0.5 % FS</td>
</tr>
<tr>
<td>Cross Axis</td>
<td>±2 %</td>
</tr>
</tbody>
</table>
The complete understanding of the structural behavior of ancient monumental buildings, especially during seismic shaking, will require a long and difficult path. The present study, which is not exhaustive on the matter, furnishes a series of observations from different perspectives involving the use of damage observations, information derived from dynamic tests, and computational mechanics.

Starting from data available from previous researches on the monument, the present work has been conducted to comprehend the seismic damage scenario. The results of the structural analysis here presented are compatible with the on-site observations and the available seismic measurements and can be summarized as follows.

The seismic accelerations, which occurred at the base of the Basilica, were polarized with a prevailing component nearly aligned with the longitudinal direction of the Basilica. Debris from the damage was found mainly inside the church, very little outside of it. The observed cracked elements (for example the concrete beam) seem not to have conserved significant permanent displacements caused by transversal accelerations.

Linear structural analyses using the design response spectrum provided by the Italian code have evidenced the strong vulnerability of the nave walls, especially the interior ones, with respect to transversal seismic actions, even in the presence of the light steel bracing systems connecting them. The combination of AQK spectra that preserves the directionality of the occurred earthquake has produced a structural response comparable with one of the prevailing longitudinal combination provided by the code. Nonlinear static analyses have used to describe the crack propagation increasing the action intensity. A plane model of the external wall in which is located the second entrance to the Basilica, has been used to compare results obtained with the STRUMAS and TSC approach, which evidence that the first one requires less computational effort preserving a sufficient level of approximation in the evaluation of the limit horizontal force sustainable by the studied element. Based on this approach, a complete model of the Basilica has been used to evaluate the seismic capacity of all the masonry macro-elements in their global behaviour. The transnet structural systems formed by arches, pendivettes and barrel vaults transferring the vertical loads onto the multi-lobed pillars, has been demonstrated to be vulnerable with respect to both transversal and longitudinal actions. In particular, even in the case of longitudinal actions, the nonlinear static analysis evidences the presence of high tensile stress in the arches connecting the pillars to the apse walls. This mechanism, observed in the numerical simulations, and amplified by the strong vertical acceleration component registered close to the site, together with the minimal resistance of the material inside the core of the collapsed pillars, is the most probable explanation of the implosion of the transept structures in the church as observed and reported in the numerous figures enriching the present study.

REFERENCES


Table 5. Recorded response of Basilica of Collemaggio.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Time (UTC)</th>
<th>Magnitude</th>
<th>Peak Response Acceleration [g]</th>
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<tr>
<td>Main Shock</td>
<td>5/5/2012</td>
<td>2:03 AM</td>
<td>5.9</td>
<td>0.0054</td>
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<td>Emilia</td>
<td>5/5/2012</td>
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<td>5.1</td>
<td>0.0018</td>
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<td>Aftershock</td>
<td>6/6/2012</td>
<td>6:08 AM</td>
<td>4.5</td>
<td>0.0014</td>
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<tr>
<td>Main Shock</td>
<td>14/10/2012</td>
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<td>2.8</td>
<td>0.0072</td>
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<td>Ravenna</td>
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<td>3.6</td>
<td>0.0073</td>
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<td>L’Aquila</td>
<td>16/11/2012</td>
<td>3:37 AM</td>
<td>3.2</td>
<td>0.0082</td>
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