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THE PERMANENT POST-EARTHQUAKE MONITORING OF THE BASILICA ST. NICHOLAS OF TOLENTINO IN CENTRAL ITALY

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ABSTRACT

During recent earthquakes that hit Central Italy in August and October 2016 several historical constructions of foremost artistic importance collapsed or experienced serious damages. The earthquake swarm dramatically confirmed the high vulnerability of monumental masonry structures to seismic actions. Among others, the Basilica St. Nicholas of Tolentino, one of the most important monument in the area, experienced several damages in its main components, which indicate a possible reduction of the seismic capacity. A permanent dynamic monitoring system was installed in April 2017, aimed at investigating the dynamic properties of the structure, observing the evolution of the damages and forecasting their effects on the structural stability. In addition, a numerical model was made, based on geometrical data coming from advanced geomatics technique solutions and updated based on the experimental dynamic results. The procedure will be focused on updating the parameters more sensitive to observed damages, able to highlight the critical aspects such as the activation of local mechanism or the integrity of connections between the structural elements.

Keywords: 2016 Central Italy earthquake, cultural heritage, seismic monitoring, dynamic identification, model updating

1. INTRODUCTION

Historical buildings are prone to experience severe damages or even collapses under the effects of seismic actions. The peculiarities of these structures make difficult designing and realizing effective consolidation works needed to minimize the risk of collapse. An important role can be played by monitoring over the time the evolution of the damages and to deepen how they can influence the structural stability of the whole building or of its parts. There are several examples of seismic monitoring of historic and monumental buildings (Lancellotta et al., 2013). In most of these, the monitoring systems are mainly designed to acquire the structural vibrations in ordinary conditions or when a pre-established trigger level is overcame (Sabia et al., 2015, Ceravolo et al., 2017). The extraction over the time of meaningful parameters from the acquired data allows observing the changes of the

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global structural response or the evolution of the damages induced by past events (Farrar and Worden, 2012). The knowledge coming from monitoring system can also be used to improve the reliability of the FE model, which can be used to optimize the design of strengthening intervention and to define warning threshold levels.

The earthquake swarm, which hit central Italy in 2016, confirmed the weakness and vulnerability of the historical buildings that characterize these territories. Between others, the religious architectures experienced severe damages or even collapses of their main structural components. This work describes the activities related to the seismic monitoring of the Basilica of St. Nicholas of Tolentino (MC), installed in April 2017. It is part of a strategic cooperation between the Politecnico di Torino, the University of Nagoya City and the local municipalities, aimed at monitoring historic structures located in Marche region. The signals acquired along the first months have been analysed and the modal properties estimated.

FE model has been built on the basis of the accurate geometric information provided by advanced geomatic survey, taking into account the damages observed on the Basilica, and updated using the identified modal properties. The non-linear analyses results have also been performed and the results compared to the existing crack pattern.

2. THE BASILICA OF ST. NICHOLAS OF TOLENTINO

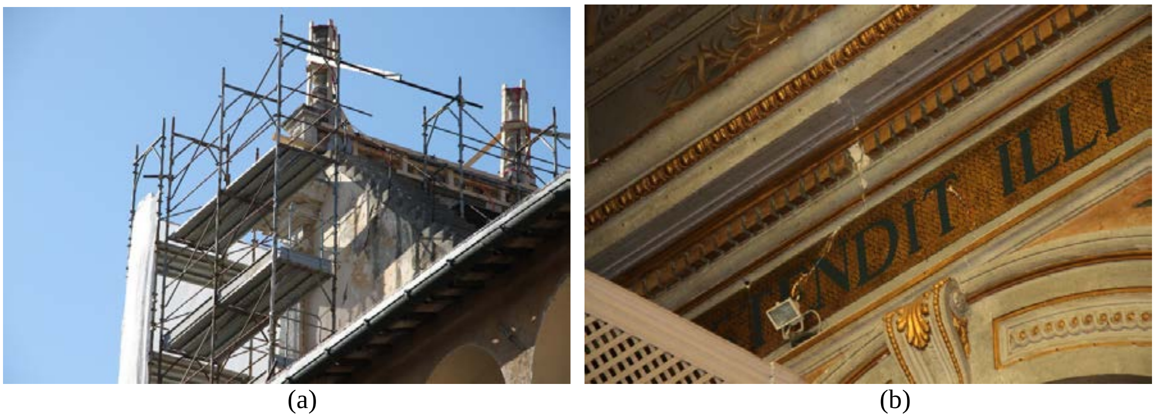
The Basilica of Saint Nicholas of Tolentino is one of the most important church of central Italy. St. Nicholas of Tolentino (1245-1305) was an Augustinian friar, miracle worker and great preacher. He lived in the convent of the Hermit Friars of St. Augustine in the city, from 1275 until his death. The basilica was consecrated in 1465. The interior is a rectangular nave with a polygonal apse. The coffered wooden ceiling and eight chapels date back to the 17th century. The Basilica preserves important works of art (S. Anna by Guercino, S. Tommaso da Villanova by G. Ghezzi). The great 17th century chapel of the Holy Sacrament, topped by a cupola, is on the left of the altar.

Of particular value is the Cappellone (Chapel) whose frescoes painted by artists from Pietro da Rimini and Giuliano and Giovanni Baronzio, of the Giotto school, are the highest examples of 14th century painting in the Marche. The chapel floor plan is rectangular and has a cross vault. A Renaissance marble arch, with the statue of St. Nicholas above it, is in the centre of the Chapel.

The imposing marble facade of the church was constructed over the centuries, and was completed in the 17th century by Nanni Di Bartolo. On the façade there are some sculptures representing St. Augustine, the Virgin Mary with child and St. Nicholas do Tolentino.

2.1 Survey of the damages

The main structural components of the Basilica have been severely damaged during the 2016 earthquake swarm. The relevant cracks opened on the buttresses of the façade, located in the upper part of the roof (Figure 1a), suggested to realize a provisional safety intervention to avoid the fall of cracked stone slabs and decorations. Cracks also opened in both the longitudinal walls of the nave at the decorative cornices (Figure 1b).



(a) (b)

Figure 1. (a) Crack opened on the buttress of the façade; (b) Cracks on the nave wall near the façade

Severe damages occurred in the structures near the altar, in particular on the dome (Figure 2a) and on the longitudinal walls, probably causing the detachment of the apse from the nave (Figure 2b). Relevant cracks also opened on the lateral chapels, in particular on the Chapel of the Blessed Sacrament, whereas moderate cracks opened on the frescos of the Cappellone of St Nicholas.

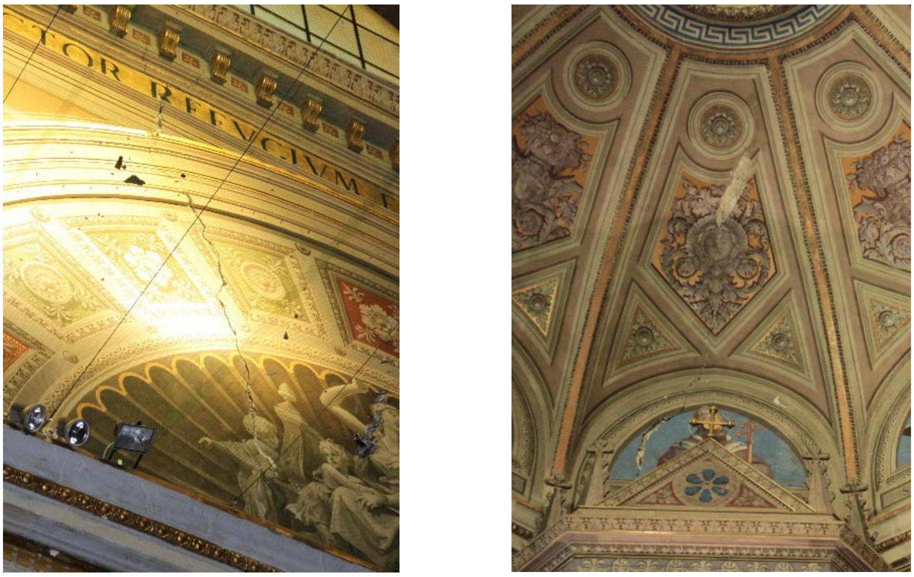


Figure 2. Details of cracks on the apse

3. THE MONITORING SYSTEM

The permanent monitoring system is composed of 20 uniaxial capacitive accelerometers, 17 of which are mounted at different heights and oriented along the principal axes of the structure, and three at the base of the bell-tower, to measure the vibrations at the foundation level (Figure 3). Moreover, the air temperatures on the nave, on the roof and on the outside are measured by three thermocouples. All the accelerometers are simultaneously acquired at a sampling frequency of 100 Hz, whereas the temperatures are acquired one sample per second.

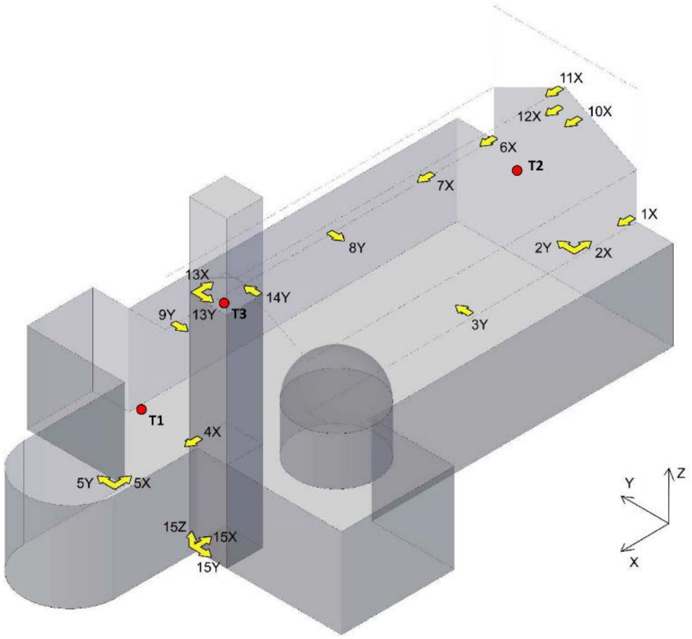


Figure 3. The monitoring system layout. Accelerometers (yellow arrows) and thermocouples (red dots)

The system layout have been designed taking into account the damage survey previously depicted. In fact, the sensors are installed to capture both the global structural dynamics and the evolution of the damage mechanisms activated in the past earthquakes. In this sense, two pairs of sensors (1X e 2X, 6X e 7X in Figure 3) have been placed on the façade and on longitudinal walls observing their relative movements to eventually evaluate the effectiveness of the wall interconnection. Similar purpose is demanded to the pairs 4X and 5X, placed along one of the main cracks of the apse.

3.1 Data analysis

Vibration data have been pre-processed to roughly define the energy content interval in frequency domain and then conditioning by subsampling, mean removal and detrending through a polynomial fitting. The main structural modes have been identified applying the Stochastic Subspace Identification (SSI) (Van Overschee e De Moor, 1996) to the ambiental vibration signals. The model order was progressively increased and the modes have been selected applying the following stabilization criteria:

- frequency variation < 1%;
- damping variation < 10%;
- damping comprised between 0% and 10%;
- MAC (Modal Assurance Criterion) > 95%.

Figure 4 reports the stabilization and frequency-damping clustering diagrams.

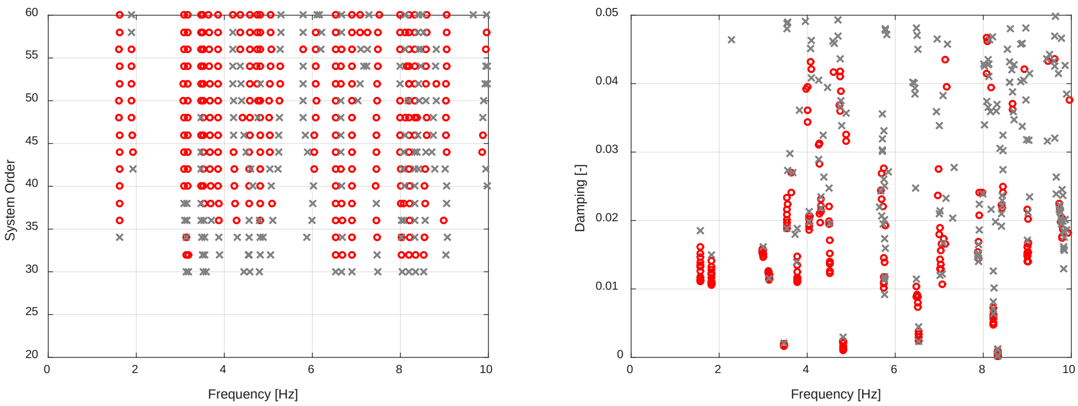


Figure 4. (left) Stabilization diagram (“x”: unstable mode; “o”: stable mode); (right) Frequency-damping clustering diagram

The identification algorithm was applied to 9 signals 1 hour long, selected one for week from April to June 2017 in order to take into account different excitation and temperature conditions.

Table 1 reports the mean values and variances of the identified modal frequencies and damping ratios, whereas Figure 5a-5e show the corresponding modal shapes.

Table 1 Modal frequencies and damping ratios identified through SSI

Mode	**Frequency [Hz]**	**ζ [%]**	**Description**
1	1.581 (0.018)	1.16 (0.22)	Tower X
2	1.849 (0.021)	1.39 (0.84)	Tower Y
3	2.997 (0.046)	1.45 (0.29)	Nave Y
4	3.124 (0.026)	1.18 (0.20)	Nave X
5	3.791 (0.043)	1.73 (0.42)	Façade

The first modal shapes are the flexural modes of the bell tower, the third and the fourth involves the nave along the main axes of the Basilica, whereas the fifth mode appears as a local mode of the façade (Figure 5).

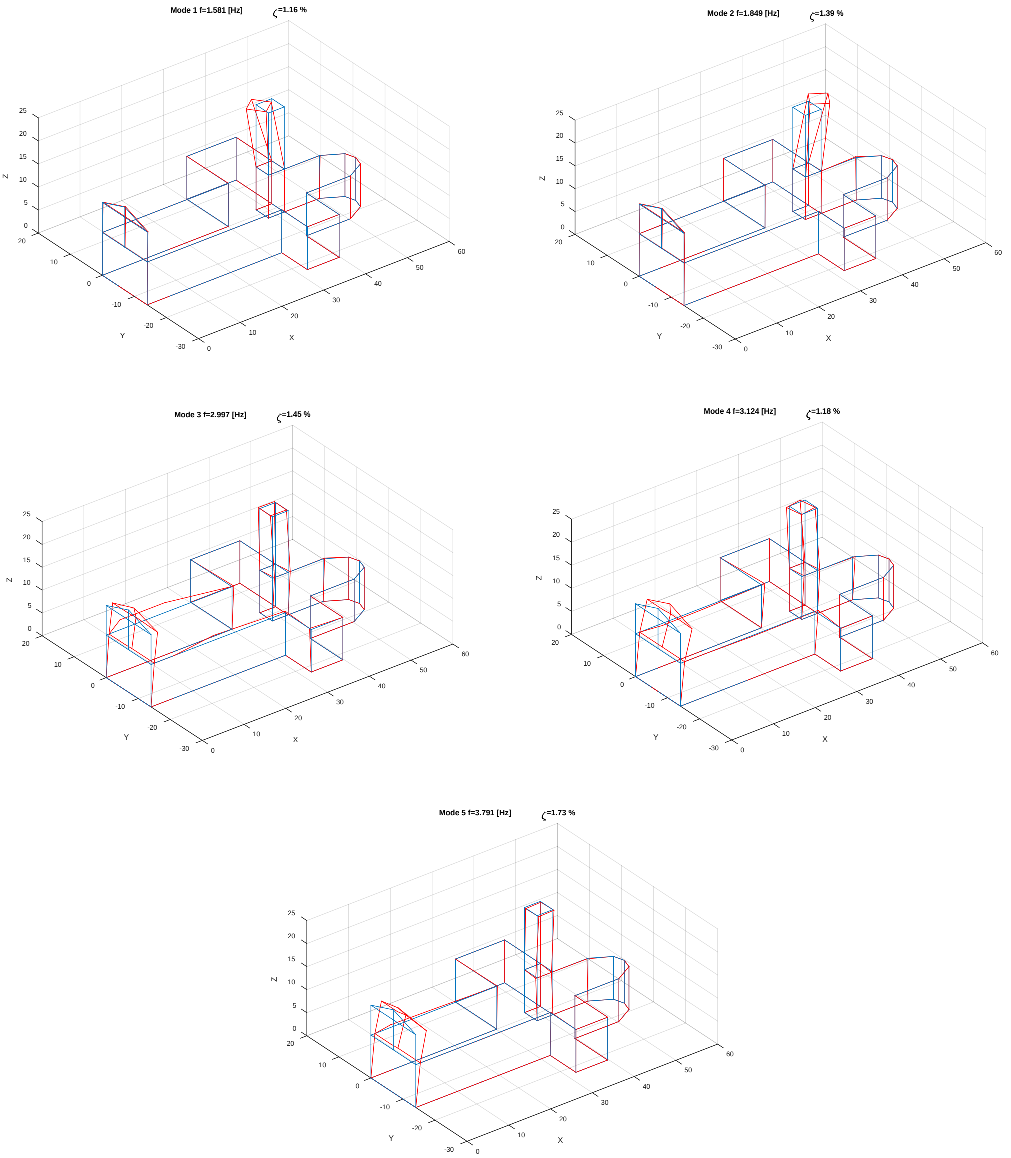


Figure 5 (a-e). Identified modal shapes. Identified mode-shapes: (a) mode 1 - 1st flexural along X bell tower; (b) mode 2 - mode 1 - 1st flexural along Y bell tower; (c) mode 3 - 1st mode of nave dir. Y; (d) mode 4 - 1st mode of nave dir. X; (e) mode 5 – Mode of the façade.

3.2 Dynamic monitoring

The data continuously acquired during the time allow to detect anomalies and singular events when the structural response deviates from a standard pattern. For example, Figure 6 shows the root mean square (RMS) of the accelerations during the investigated nine weeks both on the structure and at ground level. It is clearly visible the occurrence of an event involving at 180 h since the monitoring start. It is a sequence of three seismic event occurred the 24th of April having epicentre at Visso (MC). Figure 7 shows the recorded time histories. The RMS is an effective parameter to quickly check the responses during the time and to define reference threshold levels.

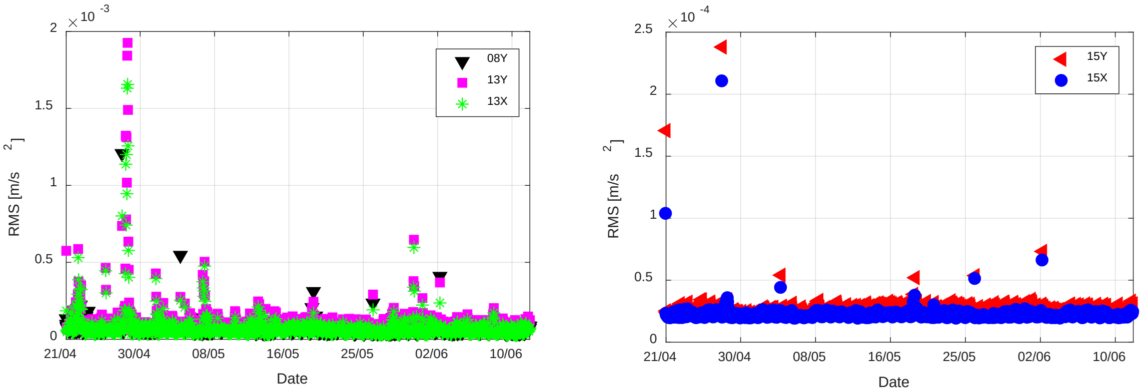


Figure 6. Data analysis. 1h RMS of the signals recorded on the structure (left) and on the foundation (right)

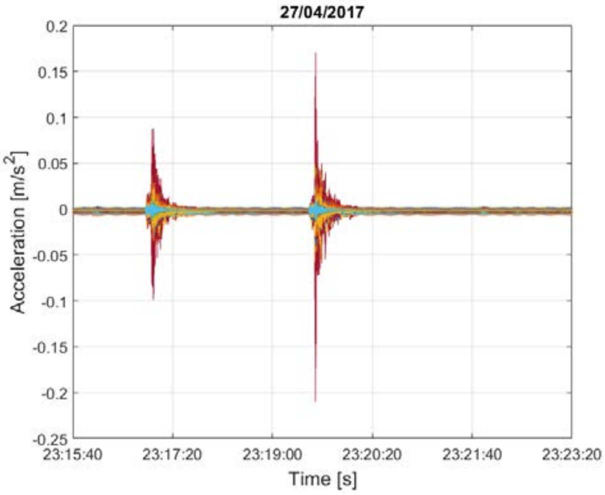


Figure 7. 27/04/2017 seismic events

4. THE NUMERICAL MODEL

The analyses of the actual condition of the Basilica require to build a numerical model that take into account the damages observed on the main structural components.

The model is based on the accurate geometric information provided by advanced geomatic survey procedures. It starts from the acquisition of a dense cloud of points. Photogrammetric methods were used for the external part of the church and “lidar” for the interior (Costanzo et al., 2017). The first step was the realization of the topographical framing network, to allow the mutual reference of all acquisitions and to geo-reference all derivative products. For the interior a topographic refinement was performed with vertices measured through Total Station. As a result, from the dense cloud of points were extracted plants, longitudinal and transversal sections, successively used to identify the median surfaces representative of the structural components.

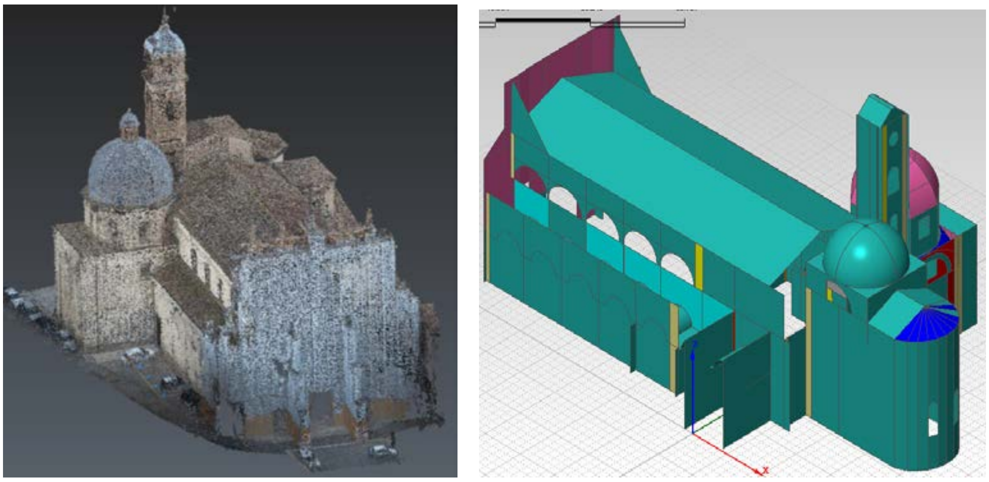


Figure 8. Dense cloud of points and the geometrical model

The FE model is constituted by about 10000 shell elements. Material properties were assumed on the basis of visual inspections, adopting the mechanical parameters suggested by current Italian codes. The elastic modulus E was adopted uniform for all the masonry components, assuming an initial value of 1.47 GPa. To take into account the connection of the Basilica with the cloister and the convent wing, linear springs were introduced to simulate the mutual interaction.

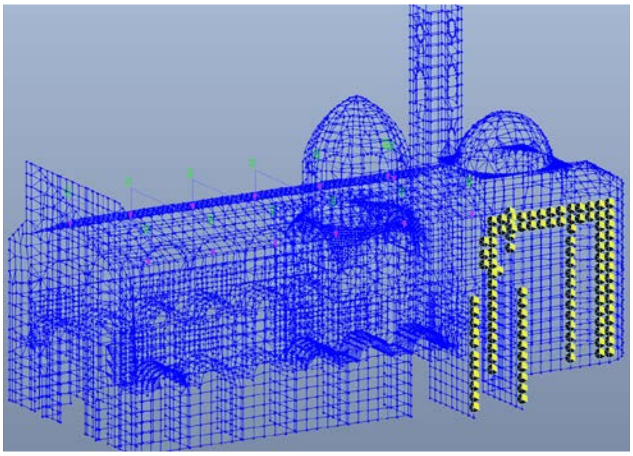


Figure 9. The FE model (in yellow, the lateral springs simulating the mutual interaction)

A non-linear static analysis was carried out on the integral model to evaluate the capacity of the structure with respect to horizontal action. This analysis allow also to highlight the criticises in the structural components and eventually to identify the activation of possible damages mechanisms. The constitutive law supposes the infinite compression resistance and null tensile resistance. The horizontal incremental load was assumed proportional to the mass. Figure 10 shows the capacity curve obtained for Y direction, where the last step corresponds to an horizontal acceleration of 0.6 g.

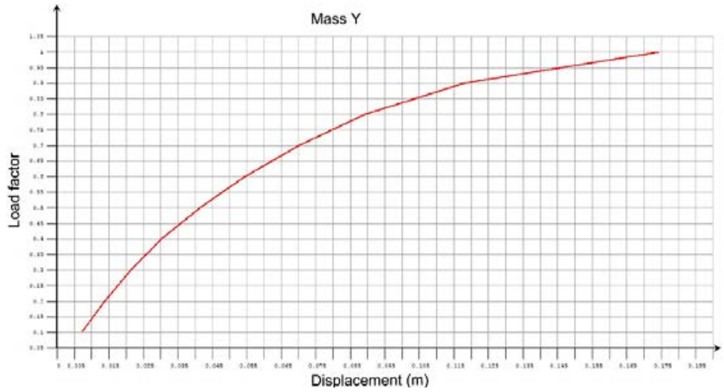


Figure 10. Capacity curve for load increment along Y direction.

Figure 11 shows the model output for the ultimate condition. The black crosses indicate the elements where the principal stress is tensile. As highlighted in Figure 11, these positions are congruent to the damage survey described in §2.1. It worth noticing that the horizontal acceleration 0.4g on the structure is compatible with the ground motion level reached in October 2016 earthquakes, circa 0.11g, and with the mean amplification factor estimated from the data acquired by the installed monitoring system during the April, 2017 earthquakes depicted in Figure 7. In this sense, the model seems to be able to reliably reproduce the real damage scenario.

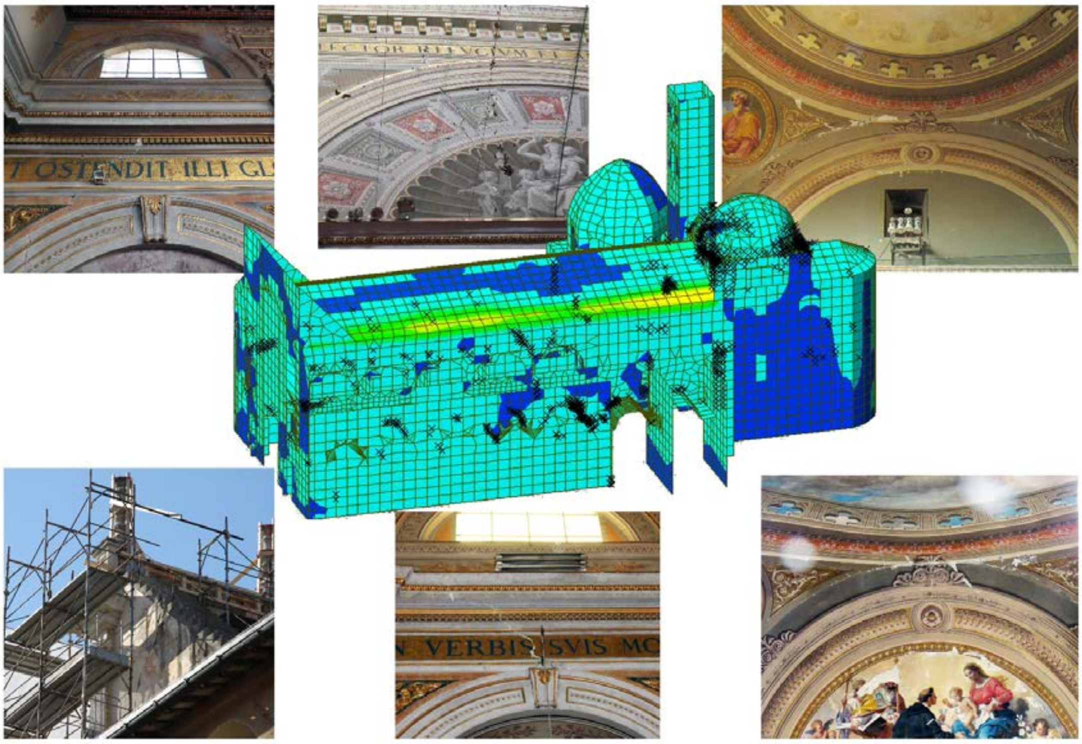


Figure 11. Comparison between the crack patterns of non-linear FE model and the observed damages

Basing on the results of the non-linear analysis, an elastic damaged model was built at the aim to compare the experimental modal components to the theoretical ones. The cracked areas were simulated reducing the stiffness of the corresponding elements tenfold. A roughly calibration of the so obtained damaged model was done on the basis of the experimental parameters reported in Table 1. In particular, the FE model was firstly discretized in homogeneous zones, i.e. the bell tower, and the elastic modulus reduced by reconciling the analytical modal responses with the corresponding identified ones.

The elastic modulus was reduced to 1.1 GPa. Table 2 reports the updated frequency values, the relative errors respect to the identified ones and the modal assurance criterion (MAC). Differences can be justified by the assumptions made in the modeling of the interactions with the other structures, not explicitly modeled, and in the use of a global elastic modulus for the masonry elements.

Table 2 Comparison between experimental and theoretical modes

Mode	FE frequency [Hz]	Identified frequency [%]	Relative error [%]	MAC [%]
1	1.64	1.581	3.73%	99%
2	1.85	1.849	0.05%	78%
3	2.97	2.997	-0.90%	69%
4	3.70	3.124	-	-
5	3.77	3.791	-0.50%	74%

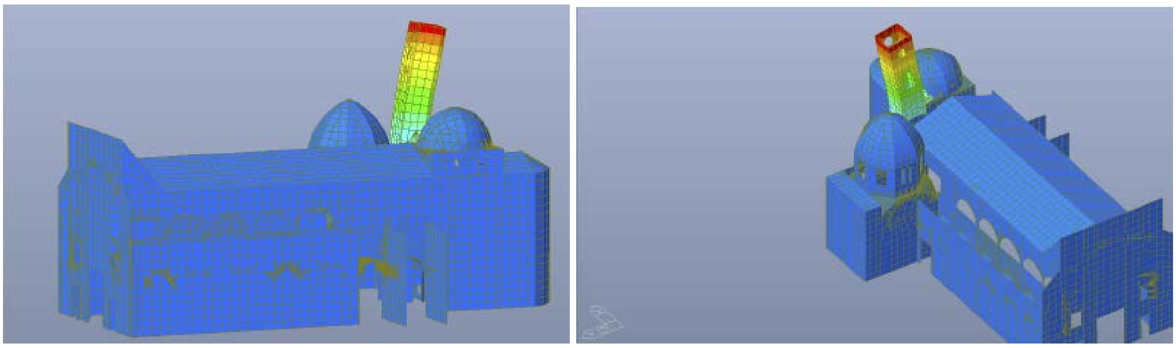


Figure 12. Bell tower FE mode shapes (mode 1 and 2 in Table 2)

5. CONCLUSION

This work presents the first results of the study conducted on the Basilica of St Nicholas of Tolentino, damaged by the 2016 Central Italy earthquake. The vibration data acquired throughout the first months were used to identify the dynamic properties of the structure. A FE model was built based on advanced geomatic techniques and both linear and non-linear analyses were performed. The non-linear static analysis demonstrates its reliability in forecasting the damage scenario induced by the past seismic events. The linear elastic model, derived from the previous non-linear analyses, was compared with the experimental data. Although the simplification made, a good matching between the FE and experimental modes has been reached. Improvement in the identification results, throughout the analysis of the incoming data, and the implementing of a deeper procedure for updating will lead to a more reliable model, able to predict the real response of the Basilica. The data coming from the permanent monitoring system will allow continuously improve the knowledge of the structural behavior and characterize the evolution of existing damages.

6. ACKNOWLEDGEMENT

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