The S. Maria di Collemaggio basilica: from the vulnerability assessment to the first results of SHM

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ABSTRACT
The basilica of Santa Maria di Collemaggio in L'Aquila (Italy) represents an important case of study. The recent history of the basilica is briefly outlined: from the damage occurred during the 2009 earthquake in L'Aquila, to the rehabilitation works and the main findings of the Structural Health Monitoring (SHM) carried out in 2018-2019, which represents the core and purpose of the current paper. The restoration aimed at increasing the global structural response by reducing the local collapse mechanisms (the facade, the apse and the nave walls etc.) highlighted by the actual seismic response. The vulnerability analysis, assessing the seismic capacity of the masonry structure before and after the restoration, is reported. A structural health monitoring program is currently operational, supported by a permanent monitoring system installed in 2017. First results concerning the observed basilica dynamics under both operational conditions and seismic events are detailed and commented: the maximum accelerations engaging each masonry are further reported.

1 INTRODUCTION

The restoration of historical buildings is a complex task, as it must be founded over a thoughtful analysis, capable of designing effective as well reversible structural interventions. The restoration design is based on numerical models able to approximately simulate the structural response (FEM, local mechanisms analysis etc. However, the effectiveness of numerical predictions should be validated by the results from experimental dynamic analysis, possibly obtained by monitoring systems. Theoretically, many items could be considered in order to promote the installation of a monitoring system (Gentile et al. 2016). Nevertheless, the use of dynamic-based methods to assess the damage is an attractive, but complex tool to apply to this type of structures (Ramos et al. 2010). After the restoration works of historical buildings, a continuous monitoring of the building-environment interaction (wind, people, inside walking, cars running nearby the building,...) may represent a valuable source of information (De Stefano et al. 2016); It could give the chance to appreciate the overall pattern of the structural response over time; A mindful interpretation of either unusual dynamic behaviours or regular structural responses may be used to plan extraordinary as well as ordinary maintenance interventions. The assessment of structural conditions could fruitfully benefit from continuous dynamic tests: data, continuously streaming from the acquisition unit, do provide a continuous glance over the structural health conditions, thus allowing the Administration responsible for the monument's maintenance, to effectively design a maintenance plan over time.

In the current paper the recent history of the basilica of Santa Maria di Collemaggio is described: from the 2009 earthquake in L'Aquila and its structural consequences on the basilica to the rehabilitation works (2016-2017) and the installation of a complex monitoring system (2017). The first results from the monitoring activity are lastly reported and commented.

2 THE BASILICA OF SANTA MARIA DI COLLEMAGGIO

The basilica, a world-famous medieval church, is a Romanesque masterpiece characterized by a dense, fascinating history and construction faces, which began at the end of the 13th century, with many stratified interventions occurred across the time.

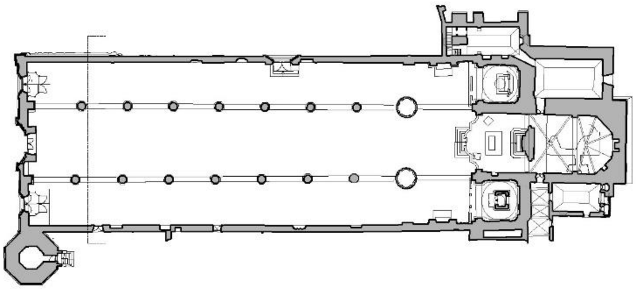


Figure 1: Plan of the basilica.

The basilica still represents an important case of study. It is characterized by a hall with three naves punctuated on each side by seven octagonal pillars supporting pointed arches, Fig.1 (Bartolomucci 2004, Redi 2006). Different and numerous are the observed masonry textures, consequence of the earthquakes, which injured L'Aquila from the 13th century.

2.1 Observed damage

The damages suffered by the basilica (Fig.2) may be classified in collapses and cracks (Antonacci et al. 2012 Antonacci et al. 2013).

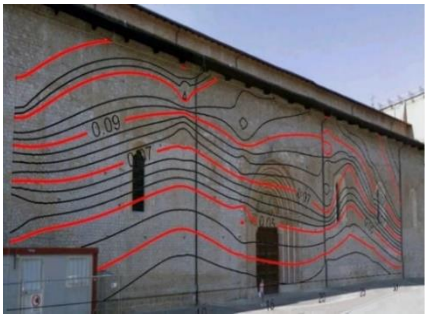
The collapse of the transept (the great multi-lobed pillars, the triumphal arch and the wall above, the barrel vaults, the dome and the roofing structures) was due to the recurrence of multiple causes, among which, in addition to the poor quality of the pillars material, to the presence of concentrated actions from the nave walls, which induced the lability of the triumphal arch itself (Antonacci et al. 2010). Out-of-plane mechanisms, exhibited by the masonry macro elements, caused instead significant cracks: the overturning of the façade from the lateral walls, the flexural behaviour of the Holy Door wall in remarked by the verticality analysis in Fig.2a.

Further signs of structural suffering emerge from the crushing of the square ashlar masonry of the nave columns, due to the heavy compression induced by the vertical and horizontal acceleration components of the 2009 earthquake.

In general, the damage mechanisms observed in façades (Fig.2b, 2c) and lateral walls could be ascribed to flexural behaviour due to out-of-plane mechanisms, inadequate interconnections among the resistant structural parts as well as to poor mechanical properties of historical mortars.

2.2 Structural rehabilitation

Both retrofitting and traditional interventions were carried out by ENI, under the direction of the Politecnico di Milano and Università degli Studi dell'Aquila. In addition, an advanced HBIM based on the laser scanner and photogrammetric survey was used to address the complex decision-making process that involved several aspects (Brumana et al. 2017).



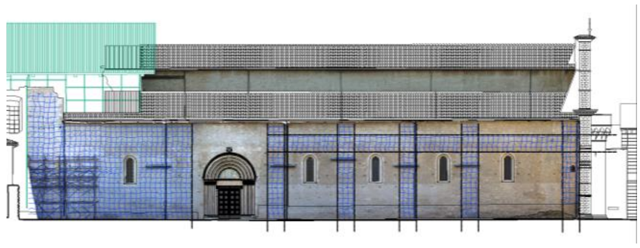
(a)

(b)

(c)

Figure 2: Evidence of damage suffered by the basilica during the 2009 earthquake: Results from the verticality analysis of the Holy Door wall (a); Detachment of the facade wall from the pilaster (b) and from nave wall (c).

The rehabilitation works, characterized by high reversibility and low invasiveness, aimed at rebuilding and repairing the damaged parts, reducing the out-of-plane mechanisms, while preserving and enhancing the historical cultural and architectural value of the basilica. Both traditional materials (Natural Hydraulic Lime mortars and stainless steel e.g.) and innovative ones (high-strength carbon fibre mesh e.g.) were used. The nave walls were strengthened by applying repointing mortar and high strength stainless steel wires. The reinforcement system, similar to the "Reticulatus" (Corradi et al. 2016) allows the reinforcement of regular and irregular-shape masonry walls, when the fair-faced aspect must be kept. It consisted in the insertion in the mortar joints, stripped to a depth of 50/70 mm, of a continuous mesh made from high strength steel wires (Fig. 3), whose nodes were anchored to both sides of the wall by means of transverse steel bars. The high strength steel wires, reinforcing both sides of the Holy Door wall, were anchored to the ground by diagonal helical bars inserted in grouted holes, while the ones, applied to the central naves, were anchored over the pillars heads. To prevent the main facade from overturning, the existing brick masonry spur was reinforced by high strength steel strips, steel retaining ties and twisted stainless steel reinforcements were added (Fig.4) . The masonry vaults were strengthened at their extrados by carbon FRP strips (Fig.5).



(a)

(b)

Figure 3: Holy Door wall: areas affected by structural reinforcement similar to the "Reticulatus" system (a); particular (b).

The masonry pillars supporting the triumphal arch, which collapsed during the main shock, were entirely rebuilt (Franchi et al. 2018, Zucca et al. 2018): they are characterized by an inner hollow RC column covered with the former outer layer of stone blocks recovered from the collapsed main pillars.

Figure 4: Steel retaining ties to prevent the main facade from overturning.

Figure 5: masonry vault strengthened at their extrados by carbon FRP strips.

The masonry pillars of the nave walls were strengthened and confined by helical stainless steel reinforcing bars (Fig. 6).

Timber and steel roof structures replaced the previous ones: traditional timber trusses covering

the nave walls were topped by Cross-Lam Timber (CLT) panels (Longarini et al. 2018), a steel frame structure replaced the RC roofs over the apse, the chapels and the transept (Fig. 7). The interventions enhanced the box-like behaviour. In order to prevent the out-of-plane mechanism of the apse retaining steel ties were further added.

2.3 Ante operam seismic vulnerability analysis

The damage observed after the 2009 earthquake is a confirmation of the vulnerabilities repeatedly shown by the basilica over the centuries, especially after the numerous telluric events, which periodically stroke the city of L'Aquila.

The evaluation of historical buildings vulnerability requires an accurate historical documentation as well as an extensive investigation phase.


(a)

(b)
Figure 6: Reinforcement of masonry columns by means of helical steel bars: core hole (a); inserting the bar (b).

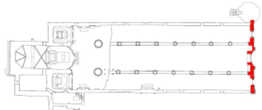
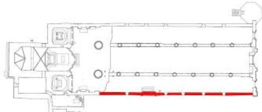
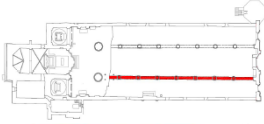
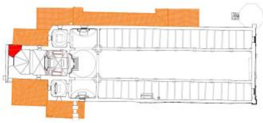

(a)

(b)
Figure 7: Steel roof structure designed to replace the existing RC structure over the apse, the chapels and sacristies (in orange (a); Installation of the steel structure (b).

The results of such investigations span from damage the structural components, to the geotechnical characterization of the soil foundation.

Dynamic analysis of irregular and structurally complex buildings show the activation of multiple vibration modes, characterized by low participation factors. For this reason and not least from the observation of recurrent seismic response patterns, global numerical models of highly complex masonry structures, like the S. Maria di Collemaggio basilica, may be not very reliable in predicting the overall structural response. In existing masonry buildings (Lourenco 2004; Lourenco et al. 2011; Angelillo et al. 2014) partial collapses often occur due to seismic action, generally, with the loss of equilibrium of rigid bodies. In the current paper limit analysis using macro-blocks is carried out for the seismic performance assessment. Synthetic results from the seismic vulnerability assessment before and after the restoration are reported, according to the Italian Seismic Code, D.M.14/01/2008.
The collapse multiplier α_0, which indicates the energy requested for the activation of each collapse mechanism, is further estimated. The vulnerability of masonry macroelements, shown in Table 1, is detailed before and after the restoration.

Table 1: Description of the masonry macroelements, chosen for the seismic vulnerability assessment

Façade		Table 1
Holy Door wall		Table 2
Nave wall		Table 3
Apse		Table 4

The analysis, following different kinematic mechanisms, has led to the results shown in 1-5, where the facade, the Holy Door wall, the nave walls and the apse are respectively considered. Due to the masonry texture complexity, it is not possible to reliably evaluate the real and precise connections between the different wall layers.
Hence various possible conditions have been examined, e.g. single or double-curtain walls.

Table 2: Short report from the facade vulnerability assessment under different out-of-plane mechanisms before the restoration, where α0 is the collapse multiplier and fa,SLV is the acceleration factor

Mechanism	α_0	Approach	$f_{a,SLV}$
Façade overturning as a monolithic element	0.07	Linear	0.36
		Nonlinear	0.94
Façade overturning as a double curtain wall	0.04	Linear	0.22
		Nonlinear	0.64
Façade with back buttresses overturning as a monolithic element	0.07	Linear	0.37
		Nonlinear	1.10
Façade with back buttresses overturning as a double curtain wall	0.07	Linear	0.27
		Nonlinear	0.80

Table 3: Short report from the Holy Door wall vulnerability assessment under different out-of-plane mechanisms before the restoration, where α0 is the collapse multiplier and $f_{a,SLV}$ is the acceleration factor.

Mechanism	Constraint	α_0	Approach	$f_{a,SLV}$
Wall overturning as a monolithic element	With roof and braces	0.08	Linear	0.44
			Nonlinear	0.91
	Without roof and braces	0.10	Linear	0.51
			Nonlinear	0.91
Wall overturning as a double curtain wall	With roof and braces	0.01	Linear	0.06
			Nonlinear	0.12
	Without roof and braces	0.03	Linear	0.13
			Nonlinear	0.25

Concerning the façade both the cases with collaborating reinforced spurs and the simple façade without them are considered.

Table 4: Short report from the nave walls vulnerability assessment under different out-of-plane mechanisms before the restoration, where α_0 is the collapse multiplier and $f_{a,SLV}$ is the acceleration factor.

Mechanism	Direction	α_0	Approach	$f_{a,SLV}$
Nave wall overturning as a monolithic element without summit constraint	North	0.03	Linear	0.12
			Nonlinear	0.41
	South	0.02	Linear	0.12
			Nonlinear	0.41
Nave wall overturning as a double curtain wall with summit constaint	North	0.13	Linear	0.67
			Nonlinear	1.00
	South	0.120	Linear	0.640
			Nonlinear	0.880

For the apse the effectiveness of pre-existing chains is investigated. In all cases, the results from calculations are compatible with the observed damage pattern; As expected, the non-linear analysis provide an acceleration factor higher than the corresponding one from linear analysis. Greater vulnerability is observed when a double curtain wall is considered, rather than when the wall is assumed as a monolithic block.

Table 5: Short report from the apse vulnerability assessment under different out-of-plane mechanisms before the restoration, where α0 is the collapse multiplier and fa,SLV is the acceleration factor.

Mechanism	Constraint	α_0	Approach	$f_{a,SLV}$
Apse corner overturning as a monolithic element with externally aligned hinge	With tie rods	0.142	Linear	0.63
			Nonlinear	1.27
	Without tie rods	0.168	Linear	0.74
			Nonlinear	2.62
Apse corner overturning as a monolithic element with backward aligned hinge	With tie rods	0.131	Linear	0.58
			Nonlinear	2.43
	Without tie rods	0.158	Linear	0.70
			Nonlinear	3.20
Apse corner overturning as a monolithic element with externally aligned hinge	Without tie rods	0.126	Linear	0.56
			Nonlinear	1.17
Apse corner overturning as a monolithic element with backward aligned hinge	Without tie rods	0.070	Linear	0.31
			Nonlinear	0.99

2.4 *Post operam seismic vulnerability analysis*

Vulnerability assessment after the restoration takes into account all the performed interventions. In some cases, different collapse mechanisms are analysed: for instance, the Holy Door wall vulnerability is assessed from the vertical flexural mechanism rather than from the simple overturning, as in the pre-interventions analysis;

The CLT roofing, installed after the earthquake, prevent the walls from simple overturning as monolithic elements. The results, reported in Table 7, evidence a significant improvement of the macroelements seismic capacity. Noteworthy is that the most vulnerable macroelement before the earthquake was the Holy Door wall, while, after the restoration works, is the facade (according to the Linear analysis) and the Holy Door wall (according to the nonlinear one).

Table 6: Short report from the vulnerability assessment under different out-of-plane mechanisms after the restoration, where α_0 is the collapse multiplier and $f_{a,SLV}$ is the acceleration factor.

Macroel.	Mechanism	α_0	Approach	$f_{a,SLV}$
Façade	Simple overturning	0.16	Linear	0.78
			Nonlinear	3.30
Apse	Corner overturning	0.24	Linear	0.87
			Nonlinear	1.60
Holy door wall	Vertical bending	2.50	Linear	2.81
			Nonlinear	1.40

3 THE MONITORING SYSTEM

Structural Health Monitoring (SHM) may have three principal objectives, (i) detection of structural properties variations, due to slow and low-intensity phenomena, which may have long-term significant effects; (ii) detection of damage, due to earthquakes or other extreme events and (iii) use in the maintenance plan in order to plan the necessary interventions.

The monitoring system continuously acquires data from sensors (Galeota et al. 2017), according to two possible recording patterns: (i) following a well-defined time schedule or (ii) when a certain acceleration threshold is exceeded. The first (i) is called "Operational Condition Monitoring", that is monitoring without interrupting the ordinary use of the buildings. The structural dynamic properties, excited from ambient vibrations under operational conditions, are periodically acquired and interpreted. The SHM last purposes consist in regularly reporting to the Administration in charge with the building maintenance about: (i) the evaluation of structural modifications with respect to a reference conditions and (ii) time evolution of the identified modal parameters, mode shapes, natural frequencies and damping factors. Such analysis do not give explicit information about the structural vulnerability but may detect modifications in the low-level dynamic response, preluding further specific analyses. On the other hand the analysis of the structural behaviour under seismic events may give information about the effectiveness of numerical/analytical models in predicting the actual response. Concerning the real time alarm system, two different alarm levels corresponding to two different acceleration thresholds, are defined: (i) an acceleration threshold corresponding to the recording of the seismic events for the purposes of analysing the basilica response; (ii) a higher acceleration threshold which triggers the event recording and the sending of a SMS to the University Staff, in charge with the Monitoring System Management and to the Stakeholders themselves.

Table 7: Monitoring system architecture.

Field station	Measurement and digitalization devices, analysis software in situ installed: accelerometers, auxiliary transducers and storage communication-acquisition hardware.
Communication interface	Middleware layer, enabling communication and management of data between the field station and the data center. In particular, a monitoring network software for management acquisition and diffusion of the measured data.
Data centre	Hardware and software for the network control, the analysis of the results and the alarms management.

The system consists of a network of sensors supported by a service platform, which synchronizes, interrogates, transmits ad manages the alarms from the different sensors. From a topological as well as functional perspective, its components are described in Table 7, exhibiting a pyramidal architecture, three levels based: Each of the three topological-functional levels are summarily described.

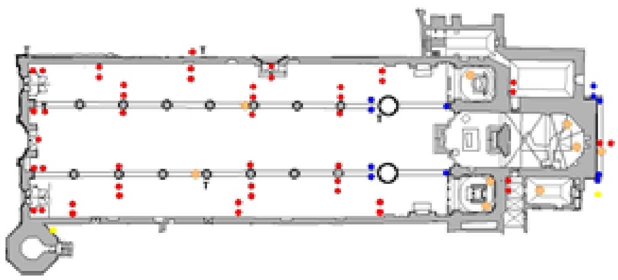


Figure 8: Monitoring system layout: monoaxial FBA(red), biaxial FBA(blue), triaxial FBA (yellow), crack monitoring device (orange), temperature and humidity sensor (T).

3.1 Field station

The installed sensors are of two types: the ones for the measurement of dynamic quantities, the accelerometers, the others for the measurement of static quantities, Fig.8. The dynamic sensors are characterized by a high dynamic range to allow the appreciation of both low and high intensity signals, expected from the structure being investigated. Crack monitoring devices are placed over historically relevant cracks, by the apse, the chapel vaults and the nave arches. From the accelerometric recordings, suitably filtered and integrated, it is also possible to obtain the inclination of the structural element, wall or pillar, on which the sensor is anchored. The accelerometers, connected to five data logger recorders placed inside and outside the Basilica, through an Ethernet cable, communicate with a central server, located in the sacristy. Data are digitized through devices with a 24 bit dynamics: both tremors with PGA of 10^{-5}g and strong-motion earthquakes can be effectively acquired. The permanent data recording in the local unit is triggered when a pre-established acceleration threshold (0.001g and 0.002g for ground and structure sensors, respectively) is exceeded. Sample rate for digitizing is 250 Hz. All system configuration parameters can be changed remotely.

3.2 Communication interface

The software, installed in the communication interface, allows the interactive control of the equipment in situ, the report of system malfunctions, the restoring of the information consistency after unexpected interruptions of the acquisitions. The middleware, with automatic pre-processing capability, manages the real time data acquisition on a non-volatile ring-buffer; the acquired time series are archived, temporally referenced and made available in a format that can be defined, at any time, by the analysis manager.

3.3 Data centre

The instrumentation consists of communication devices, workstations for hosting and managing databases as well as storage and diffusion systems. Enabled users have access to the database for the management as well as for the analysis and publication of the results. The data centre has, among the others, the following functions: it acquires near-real-time data on non-volatile disk ring-buffers, monitors in-situ equipment interactively checking the system's functionality, provides automatic data processing, analyses and displays system status information, manages messaging communication, stores data in a relational database system. It provides the operator with diagnostic, command and control tools, realizes the dissemination (in a VPN) of data to other authorized analysis centres. In particular all recorded data are sent to a second workstation hosted by the Università degli Studi dell'Aquila.

4 FIRST RESULTS

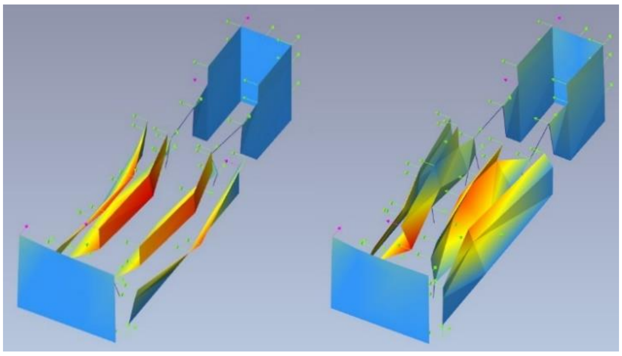
Acquired data from accelerometers are processed by means of the ARTEMIS package (Artemis Modal, 2017), in which numerous identification algorithms are implemented. In particular two well-known identification algorithms have been used, the Enhanced Frequency Domain Decomposition (EFDD) and the Covariance-Driven Stochastic Subspace Identification (SSI) (Peeters et al. 1999). The first is a frequency-domain technique, based on the singular value decomposition (SVD) of the spectral density matrix built from acquired data. The latter is a well-known time-domain technique. The SSI technique, though more time consuming than the EFDD, has been adopted with the twofold objectives of validating the results obtained from the first quicker technique and to have a more accurate identification in case of nearly coincident mode shapes. The correspondence between the frequency domain and the time domain eigenvectors is checked by means of the MAC (Modal Assurance Criterion) index (Ewins 1984), which measures the alignment between the two different mode shapes, identified from the EFDD and the SSI techniques and the complexity (Peeters and De Roeck 1999): complexity, expressed as a percentage, will be 0 when the mode is real, 1 when is imaginary. So, when the mode is real, complexity should always be around 0.) For perfectly correlated modes MAC is unity, and for completely uncorrelated shapes is zero. The Complexity value, which gives approximate estimate of identified mode shapes reliability, is further estimated (Aloisio et al. 2019) attempted to implement an online Bayesian updating procedure, for the continuous assessment of the basilica status over time. Nevertheless the procedure, calibrated on ambient vibration data acquired by the façade accelerometers, is not fully operational. In the first identification step all accelerometer outputs are processed as a whole, in a further step a macroelement analysis is driven. In the latter, the identification algorithm processes the sole data coming separately from each macroelement, the facade, the nave walls and the apse.

4.1 Modal analysis driven by the global model

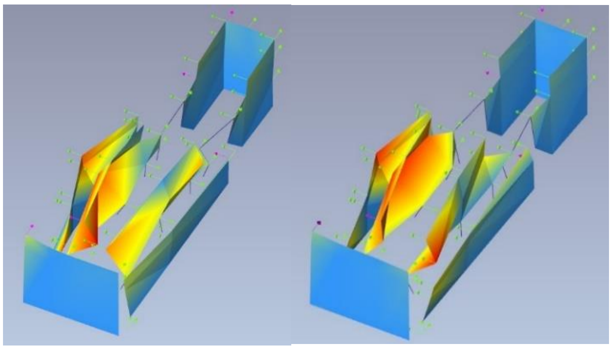
A general agreement between the results from EFDD and SSI does emerge, Table 8, as confirmed by the MAC index performed between mode shapes from the two mentioned techniques. Besides, the mode shapes from EFDD show a higher complexity than those from SSI.

Table 8: Comparison between modal parameters identified from the EFDD and the SSI techniques, obtained from all measuring points, ξ and C are the estimated damping ratios and complexity values.

EFDD			SSI		
f[Hz]	ξ[%]	C[%]	f[Hz]	ξ[%]	C[%]
2.09	1.16	1.29	2.09	1.23	0.55
3.11	0.72	18.07	3.11	1.28	3.98
3.37	0.98	3.83	3.27	1.10	1.01
3.82	0.71	44.34	3.82	1.12	3.43
4.45	0.53	9.32	4.45	1.35	0.55
4.79	0.53	11.50	4.79	0.76	1.62
5.03	0.51	1.46	5.04	0.71	1.23
5.25	0.27	5.99	5.23	0.73	5.76

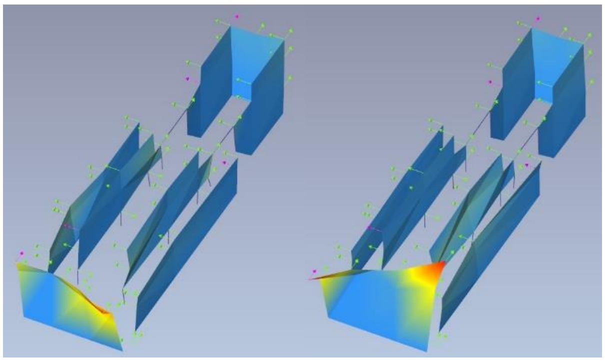


f = 2.09 Hz, MAC = 0.99; f = 3.11 Hz, MAC = 0.87

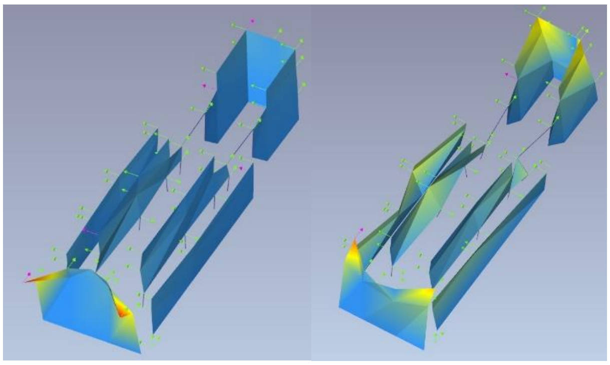


f = 3.27 Hz, MAC = 0.99; f = 3.82 Hz, MAC = 0.82

Figure 9: Modal shapes for complete model: lower frequency mode shapes, principally engaging the out-of-plane wall dynamics.



f = 4.45Hz, MAC = 0.84; f = 4.79Hz, MAC = 0.95



f = 5.03Hz, MAC = 0.99; f = 5.25Hz, MAC = 0.91

Figure 10: Mode shapes from the complete model: higher frequency mode shapes, engaging the façade and other parts of the basilica.

Mode shapes complexity could be related to multiple causes, such as non-proportional damping, bad measurements or poor modal parameter estimation, inconsistent data due to e.g. time variant conditions.

Lower frequency mode shapes principally arise from the nave wall dynamics Fig.9, while higher ones from the façade dynamics Fig.10, though the 5.2Hz mode shape involves nave walls too.

4.2 Modal analysis driven by the macroelement model

Dynamic identifications have been performed from reduced data set coming from each macroelement, Table 9. For the apse no stable mode shapes were identified in the frequency range 0-6.25Hz.

4.3 Seismic Data

So far, the monitoring system was triggered 5 times by seismic events. In Fig.11 the histogram representation of the maximum accelerations recorded by the monitoring system for each macroelement: the facade, the nave walls (progressively labelled N.W. A-D starting from the Holy Door Wall), the apse and the transept. The façade was surely the macro-block mostly invested by the earthquakes; the nave walls almost have the same maximum accelerations, while the apse and the transept show the least recorded values. The seismic response of the basilica has been further investigated by means of Input-Output identification techniques by (Aloisio et al. 2019) showing a non-stationary behaviour under earthquake due to possible rocking phenomena (Di Egidio et al. 2019). The facade natural frequency at 4.4Hz, may shift towards 4.2Hz with higher amplitude of the excitation.

Table 9: Modal parameters identified by considering both all measurement points on the entire structure and the sole accelerometers placed on each macro-element.

Global			Holy door wall		
f[Hz]	ξ[%]	C[%]	f[Hz]	ξ[%]	C[%]
2.09	1.23	0.55	2.08	1.22	0.06
3.11	1.28	3.98	3.12	1.46	0.89
3.27	1.10	1.01	3.27	1.07	0.75
3.82	1.12	3.43	3.82	1.2	0.26
4.45	1.35	0.55	4.45	1.03	0.45
4.79	0.76	1.62			
5.04	0.71	1.23			
5.23	0.73	5.76	5.23	1.64	1.68
Nave wall			Facade		
f[Hz]	ξ[%]	C[%]	f[Hz]	ξ[%]	C[%]
2.09	1.27	0.06			
3.11	1.11	4.04			
3.27	1.17	0.58			
3.95	1.44	0.08			
4.44	2.58	0.56	4.46	1.01	0.97
4.85	1.60	0.57	4.79	0.56	1.38
			5.03	0.65	2.46
5.23	1.33	1.04			

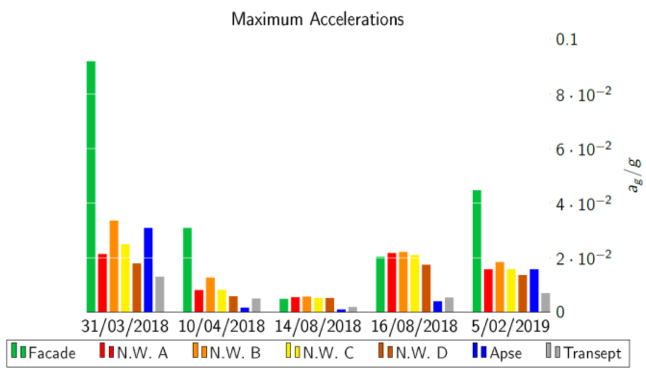


Figure 11: Histogram representation of the maximum accelerations recorded by the monitoring system for each macroelement.

5 CONCLUSIONS

The restoration works, following the 2009 earthquake, aimed at reducing the effects of local collapse mechanisms, attempting to enhance the box-like behaviour. FEM analysis, carried out on several structural models representative of both the basilica conditions before and after its rehabilitation, confirmed the benefits from the performed structural interventions. The basilica represents an important case of study, where a complex monitoring system provides continuous information on the structural behaviour over time. From the analysis of the system eigenstructure, the current status of the basilica can be assessed and compared with precedent structural conditions. The results of dynamic monitoring under operational condition and during seismic events are briefly reported. In particular, the maximum accelerations reached during the seismic events occurred in the last year are detailed for each macroelement. The authors will aim at implementing more sophisticated non-parametric damage detection algorithms capable of tracking the structural status over time. The correlation between the results of damage detection tests and the occurrence of certain damage patterns, based on numerical or experimental calibrations, could be effectively used for the on-line assessment of the basilica structural safety from data driven analysis. The recurrence of several damage patterns, observed in the basilica after the 2009 earthquake, highlighted the issue of the seismic vulnerability of architectural heritage. Actually, the lack of reliable correlations between the results of structural monitoring, based on continuous dynamic tests, and the occurrence of damage in masonry like structures, determines a non-extensive use of dynamic monitoring. If further investigations are carried out in the future, dynamic monitoring will represent a current and effective strategy for the direct and continuous assessment of structural safety of masonry-like structures.

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