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INTEGRATED METHODOLOGIES BASED ON STRUCTURAL HEALTH MONITORING FOR THE PROTECTION OF CUTURAL HERITAGE BUILDINGS

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SUMMARY

In the last decades the need for an effective seismic protection and vulnerability reduction of strategic structures and particularly the architectural heritage determined a growing interest in Structural Health Monitoring (SHM) as a measure of passive mitigation of earthquake effects. The object of monitoring is to identify, locate and classify type and severity of damages induced by external actions or degradation phenomena and to assess their effects on the structural performance. In this way it is possible to take appropriate measures to reduce the danger of collapse and, when necessary, perform strengthening interventions to improve the structural and seismic capacity.

Motivated by the above reasons, this thesis aims at providing a contribution to the development of techniques and integrated methodologies, based on SHM, for the assessment and protection of Cultural Heritage (CH) buildings and monuments.

Firstly, after a detailed state of the art review on specific topics related to SHM of civil engineering structures, a new methodology for the implementation of monitoring techniques on historic masonry structures is proposed. Selected case studies, equipped with distributed sensors and acquisition systems, allowed the definition and successive validation of SHM as a knowledge-based assessment tool, implemented to evaluate intervention needs, following an incremental approach during their execution, and to control the damage states of buildings in a post-seismic scenario.

In order to maximize the benefits of SHM and optimize the entire process, dedicated software for static monitoring and automated algorithms for modal parameters identification have been developed, able to provide almost real time information on the health state of the monitored structure.

Finally integrated procedures based on robust statistical and numerical models have been implemented to interpret and exploit SHM outputs to assess the structural conditions of the investigated CH buildings.

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SOMMARIO

L'esigenza di una efficace protezione sismica e di riduzione della vulnerabilità del costruito storico, con particolare riferimento alle strutture strategiche o sottoposte a tutela dei Beni Culturali, ha determinato negli ultimi decenni un crescente interesse nei confronti del monitoraggio strutturale (Structural Health Monitoring - SHM) come misura di mitigazione passiva degli effetti dei terremoti. L'obiettivo del monitoraggio è identificare, localizzare e classificare tipologia e gravità del danno indotto da azioni esterne o fenomeni di degrado e di valutarne gli effetti sull'integrità e la capacità strutturale dell'edificio oggetto di indagine. In questo modo si possono adottare dei provvedimenti atti a ridurre il pericolo di collasso e, quando necessario, a eseguire interventi mirati di rinforzo strutturale e miglioramento sismico.
Sulla base di queste motivazioni il presente lavoro di tesi ha l'obiettivo di fornire un contributo allo sviluppo di tecniche e metodologie integrate di monitoraggio strutturale per la valutazione e protezione sismica dei Beni Culturali.
In seguito ad una dettagliata revisione dello stato dell'arte su specifici aspetti relativi al monitoraggio di strutture dell'ingegneria civile, è stata sviluppata e proposta una metodologia per l'implementazione di tecniche di monitoraggio su edifici storici in muratura. Casi di studio attentamente selezionati e dotati di una rete di sensori distribuiti e unità di acquisizione, hanno permesso la definizione e successiva validazione di sistemi di controllo strutturale. Il monitoraggio è stato utilizzato come strumento basato sulla conoscenza da un lato per la valutazione dei bisogni di rinforzo e miglioramento sismico, da eseguirsi procedendo con approcci incrementali e interventi sequenziali, dall'altro per il controllo dello stato di avanzamento del danno di edifici gravemente colpiti da terremoto.
Al fine di massimizzare i benefici del monitoraggio strutturale e ottimizzarne l'intero processo, sono stati sviluppati software dedicati per l'analisi dei dati statici e algoritmi per l'identificazione automatica dei parametri modali, in grado di fornire informazioni quasi in tempo reale sullo stato di salute dell'edificio.
Infine si sono implementate procedure integrate basate su modelli statistici e numerici avanzati con lo scopo ultimo di interpretare e sfruttare le informazioni estratte e i risultati ottenuti per valutare le condizioni strutturali degli edifici storici e monumenti sottoposti a controlli strutturali continui e permanenti.

DEDICATION

To my family

“Motivation is so important. In fact all human actions can be seen in terms of movement, and the mover behind all actions is one’s motivation. If you develop a pure and sincere motivation, if you are motivated by a wish to help on the basis of kindness, compassion and respect, then you can carry on any kind of work, in any field, and function more effectively with less fear or worry, not being afraid of what others think or whether you ultimately will be successful in reaching your goal.”

- The Dalai Lama -

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1 INTRODUCTION

1.1 Background and motivations

Structural Health Monitoring (SHM) represents an ancient engineering technique in its most elementary conception. Nowadays SHM systems are composed by a network of sensors connected to an external data acquisition unit. Most modern systems incorporate wireless transmitters and receivers to overcome some limitations caused by the use of cables. Those systems are capable of assessing the structure continuously, providing real time information on its health state, including identification, localization and quantification of damage.

Civil engineering structures are subjected in fact to external loads during their lifetime. Actions can be environmental (wind, earthquakes, natural hazards, etc.) or anthropogenic, as a result of the human interaction. Regardless of the causes, those events can produce damages and structural changes and may affect performance and serviceability of structures. Since it is not possible to predict and calculate the exact effect of the induced stresses, buildings need to be continuously controlled so that possible damages can be assessed and their residual life estimated. This complex process is known as structural monitoring. The objective is to identify, locate and classify type and severity of damages and estimate the effects on the structural performance. Once the damage has been identified and evaluated, it is possible to take the appropriate measures to reduce the danger of collapse and, if necessary, to perform some strengthening interventions.

Historically, structures were monitored by visual inspections and with the aid of simple devices or tests. However, traditional methods have many drawbacks. Firstly they are limited to the accessible areas of the building. Secondly, they can be performed only periodically, as they require the presence onsite of technicians and experts. Finally, this kind of inspections is highly subjective and rely on the experience and judgment of inspectors. Although these methods are still widely used, in the last decades there has been a gradual shift towards the use of automatic sensors in the field of structural monitoring.

Even though there are higher initial costs associated with the installation of a monitoring system, the long-term benefits and savings on operating and management costs are extensive. Typically, when structures are subjected to exceptional events, accidental overloads or other phenomena, it is necessary to verify their structural and safety conditions and, in most cases, they must be kept closed to public until the necessary inspections are performed. Especially in the case of natural disasters, when many buildings in a small area may be affected and damaged, this process can last for months or years. Downtimes, therefore, can be long and lead to very high associated costs. The implementation of SHM can minimize the recovery time, providing real time information on the structural performance. In the worst case a structure could be overloaded unconsciously, reaching a catastrophic collapse. In those cases a SHM system is able to detect overloads and associated damages and acts as an early warning tool.

SHM play an important role also in the field of the maintenance process of civil engineering structures. Traditionally, maintenance is performed on a scheduled basis, which can be inefficient. Thanks to SHM, maintenance can be implemented only when needed, avoiding the execution of unnecessary interventions and respecting in general both economic criteria and conservation principles when applied to cultural heritage patrimony.

In the last decades the need for an effective seismic protection and vulnerability reduction of the built environment, especially of infrastructures, strategic constructions or Cultural Heritage (CH) buildings, determined a growing interest in SHM as a measure of passive mitigation of earthquake effects.

The Italian territory, moreover, is characterized by high seismicity and Cultural Heritage is constantly at risk. Historic buildings are often designed and built with deficient construction details and the poor state of conservation of materials and components has obvious consequences on their overall performance. For this reason seismic assessment and structural rehabilitation are considered nowadays a crucial phase during the processes of urban planning and civil protection.

Given the growing attention to SHM both in the professional and scientific community, it seems that the operating procedures in the study and conservation of architectural heritage are increasingly supported by the scientific community and the advanced research, dismissing the typical coarse and sometimes unsophisticated approach that characterized until a short time ago each restoration project.

In order to design and implement a monitoring program, in fact, it is necessary to perform accurate analyses on the structural functioning of the building following a rigorous scientific process: from the study of the historical and geometrical evolution of the structure to a methodical survey of the crack pattern to interpret any active damage mechanisms and define the most important parameters to be monitored. Combining and cross correlating data from in situ or laboratory investigations and dynamic identification tests with the preliminary results of reference Finite Element (FE) models, it is possible, therefore, to design the architecture of the permanent monitoring system to be installed on the protected building. Significant parameters and characteristic properties are constantly controlled, allowing continuous inspections of the construction and giving the opportunity to schedule maintenance work and execute repair interventions in case of a sudden worsening of the structural conditions.

In some cases, when the damaging mechanisms are well known and safety thresholds can be defined, SHM becomes an efficient alternative tool to the execution of unnecessary strengthening interventions, in agreement with the conservation principles.

A typical architecture of SHM is based on distributed sensors directly connected by cables or wireless control units to a centralized data acquisition system. Data collection, analysis and interpretation are, however, always difficult to implement. Most critical aspects are related to the installation process (such as choice of optimum number and position of sensors, wires placement, selection of the best monitoring strategies, logistical complications) and to the difficulties of correlating features extracted from raw data with the actual performance of the structure or the real effectiveness of the adopted strengthening solutions.

The capability of processing and analyzing quickly all the monitored parameters, through the implementation of automatic procedures, allows a rapid assessment of health conditions. Additionally, the automation of SHM processes and the consequent availability of results within a reasonable time, give the possibility to design quickly consolidation works only if really needed and to assess the effectiveness of the implemented retrofitting. Further advantage is the possibility to set warning thresholds useful to guarantee appropriate safety conditions during the execution of interventions or in case of buildings severely damaged by earthquakes. Moreover SHM outcomes can be exploited to validate, calibrate and update reference numerical or analytical models, fundamental to assess the capacity of historic structures and perform the necessary verifications of their structural and seismic performance.

In this context SHM represents an integrated methodology within the complex knowledge-based approach for the assessment, protection and conservation of architectural heritage.

1.2 Problem statement

For several years the Department of Civil, Architecture and Environmental Engineering of the University of Padova, in line with the needs for the conservation of monuments, has been employing SHM systems on representative historical constructions and monuments. The final aim is the assessment of their structural behavior, especially with reference to the seismic performance, integrating monitoring techniques, inspections and structural analyses.

Numerous case studies have been analyzed during the research activities described within the present PhD thesis, from the initial knowledge phase, to the execution of specific diagnostic investigations and, through the construction of numerical models, until the implementation and real application of state-of-the-art monitoring systems on selected monuments.

In this framework, designated case studies cover different types of problems concerning specific objectives and applied methodologies. The different problems identified are a consequence of the meaningful differences shown by case studies with regards to construction typologies, seismicity of the sites, quality of the construction materials and techniques, damage conditions, effects of past or recent earthquakes and effectiveness of possible performed strengthening interventions.

CH buildings, currently equipped with SHM systems, are listed hereafter (in bold monitoring systems designed and installed during the research period):

- **Roman Arena of Verona** (Verona, Italy)
- Cansignorio stone tomb (Verona, Italy)
- S. Sofia church (Padova, Italy)
- **Spanish Fortress** (l'Aquila, Italy)
- **S. Marco church** (l'Aquila, Italy)
- **S. Biagio / S. Giuseppe churches** (l'Aquila, Italy)
- **Civic tower** (l'Aquila, Italy)
- **S. Agostino church** (l'Aquila, Italy)
- **S. Silvestro church** (l'Aquila, Italy)

Thanks to the implementation of a large number of monitoring systems to CH structures it was possible to define a methodology of application according to the final aim of monitoring within each selected case history. Following this approach monitoring represents an essential step in the assessment and protection processes of historical constructions, regarding, in particular, the following problems:

a) Increasing the knowledge on the structural behavior using SHM to assess strengthening needs and avoid the execution of unnecessary interventions;
b) Applying an incremental approach to the execution of strengthening interventions using SHM before, during and after the implementation, validating eventually their effectiveness;
c) Post-earthquake controls on severely damaged buildings using SHM to control the evolution of damage and verify the effectiveness of provisional strengthening measures.

In the following section monitored CH buildings are briefly presented, highlighting aims and problems that SHM tries to face and solve.

ARENA OF VERONA	
INSTALLATION PERIOD	December 2011
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Alternative to the execution of interventions

CANSIGNORIO STONE TOMB (VERONA)	
INSTALLATION PERIOD	December 2006
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Structural controls before, during and after interventions

S. SOFIA CHURCH (PADOVA)	
INSTALLATION PERIOD	1999 (1st installation); 2008 (1st upgrade); 2010 (2nd upgrade)
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Structural controls before, during and after interventions

CIVIC TOWER (L'AQUILA)

INSTALLATION PERIOD	July 2010
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

SPANISH FORTRESS (L'AQUILA)

INSTALLATION PERIOD	December 2009
SHM TYPOLOGY	Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

S. MARCO CHURCH (L'AQUILA)

INSTALLATION PERIOD	August 2009
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

S. BIAGIO & S. GIUSEPPE CHURCHES (L'AQUILA)

INSTALLATION PERIOD	December 2010
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

S. AGOSTINO CHURCH
(L'AQUILA)

INSTALLATION PERIOD	July 2010
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

S. SILVESTRO CHURCH
(L'AQUILA)

INSTALLATION PERIOD	July 2010
SHM TYPOLOGY	Static/Dynamic system
PURPOSE OF MONITORING	Post-earthquake controls

1.3 Aims and methods of research

The main aim of the present research is to provide a contribution to the development of techniques and integrated methodologies, based on Structural Health Monitoring, for the assessment and protection of the Cultural Heritage patrimony.

The first part of the work focuses on a detailed and thorough literature review on the application of structural monitoring to civil engineering structures, with particular attention to the field of historic buildings and monuments. Addressed issues include also a general description of the monitoring process and a state of the art on the available tools to automate and make more efficient data treatment and feature extraction phases. A specific section is dedicated to an introduction to the basic concepts of structural dynamics and Operational Modal Analysis, which constitute a fundamental part of SHM and are extensively applied throughout the research.

In the second part the specific application of SHM to CH buildings is introduced establishing and proposing a methodology, still missing at the state of the art, for the implementation of monitoring techniques on historic constructions. The aim is to extend methodologies, technologies and strategies of monitoring already well-establish in the application to civil and mechanical engineering structures to the field of conservation and protection of monuments and historic centers.

Following the proposed methodology some selected SHM case studies are presented with particular attention to the description of preliminary phases that are needed to design the architecture of a monitoring system, select the optimum layout and choose the best monitoring strategy. Among the large number of available case studies and previously introduced, it is decided to select two monuments in Verona (the Roman Arena and the Cansignorio stone tomb) and two in l'Aquila (the Civic Tower and the Spanish Fortress). The idea is to cover different problems related to the needs of monitoring: from the alternative to the execution of structural interventions, to the control of their effectiveness, until the diffused application of SHM on severely damaged buildings in a post-seismic scenario.

The third part of the research is aimed at the development of automatic computerized system to improve data treatment and processing and facilitate the interpretation of results in a general framework for a more efficient monitoring process. The central core of each monitoring system is in fact the capability to automatically extract information from which damage detection algorithms are able to identify structural damages. The automation of SHM is therefore of fundamental importance in order to obtain a continuous and reliable monitoring. Starting from these considerations dedicated processing software for static monitoring on the one hand and automated subroutines for modal parameters identification on the other hand are developed, able to provide almost real-time information on the monitored parameters and on the health state of the structure.

Finally the last section of the thesis develops and defines procedures based on both data-driven and model-driven approaches to interpret, post-process, detect damages and in general exploit the results of monitoring to increase the knowledge level and assess the structural conditions of a CH building. Both statistical and numerical models are constructed and fed with features extracted from monitored parameters in order to: (i) control the structural behavior of the building under operational conditions; (ii) develop and calibrate reference behavioral models; (iii) study and characterize the structural response in case of exceptional events.

In conclusion, to clarify how this work contributes to the overall advancement of SHM applied to historic building and monument, the division of statistical pattern recognition paradigm firstly proposed by Farrar & Worden 2007 is presented. Within this paradigm, the process of SHM is subdivided into a sequence of four stages which are summarized in Fig. 1.1: (i) Operational evaluation; (ii) Data acquisition; (iii) feature extraction and (iv) Statistical model development for feature discrimination. This work plays an active role within the third and partially the fourth phase of the proposed paradigm.

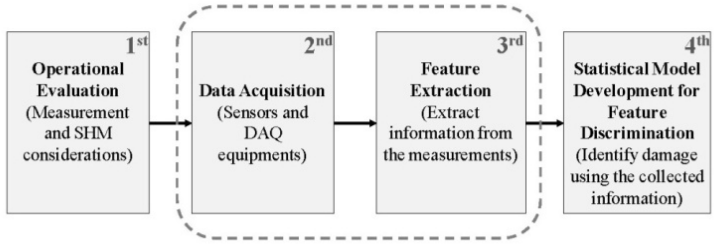


Fig. 1.1 - Phases of the statistical pattern recognition paradigm (Farrar & Worden, 2007)

1.4 Thesis organization

The thesis is organized into 9 chapters according to the logical path presented in the previous paragraph.

Chapter 1 presents an introduction to the work, focusing on background and motivations, aims and methods of the research. The thesis outline is finally described.

Chapter 2 provides a state-of-the art of Structural Health Monitoring (SHM) applied to the field of civil engineering structures. Addressed issues include a review on the application of monitoring techniques during the last century on a large variety of structures and infrastructures, description of the monitoring process with particular attention on the automated feature extraction phase. To conclude problems and limitations of SHM are reported.

Chapter 3 focuses on the basic concepts of structural dynamics together with a state-of-the-art review of some common Operational Modal Analysis techniques, successively exploited in the application to real case studies and in the development of automated algorithms for modal parameters identifications.

Chapter 4 describes the development of a sound methodology for the application of SHM to Cultural Heritage (CH) buildings and monuments. The role of monitoring within the complex knowledge-based process of conservation and protection of heritage structures is identified and discussed. Finally the application of monitoring techniques to face specific problems is presented.

Chapter 5 reports the application and validation of the proposed methodology for monitoring CH buildings to some selected real case studies, focusing in particular on the importance of the execution of detailed historical research and the implementation of inspections to increase the knowledge level on the building and define the optimum layout of the monitoring system.

Chapter 6 presents the development of specific subroutines and a dedicated Graphical User Interface for the online automated processing of data, recorded by static systems. The algorithms include the possibility to post-process the acquired signals, act as a useful tool to visualize, plot and cross correlate the results and finally integrate specific early warning tools. A validation of the developed algorithms through the application on monitored structures is also presented.

Chapter 7 describes the development of algorithms for automated modal parameters extraction from the measured response provided by dynamic monitoring. The proposed methodology uses a well-known parametric frequency domain identification technique and it is then complemented by a new procedure developed for the automatic analysis of the stabilization diagrams, based on a hierarchical clustering algorithm. A validation of the developed algorithms through the application on monitored structures is also presented.

Chapter 8 focuses on the implementation of procedures based on both data-driven and model-driven approaches to interpret, post-process, detect damages and in general exploit the results of monitoring to increase the knowledge level and assess the structural conditions of a CH building.

Chapter 9 presents the main conclusions from each chapter and a proposal for future works.

A schematic representation of the thesis outline is presented in Fig. 1.2.

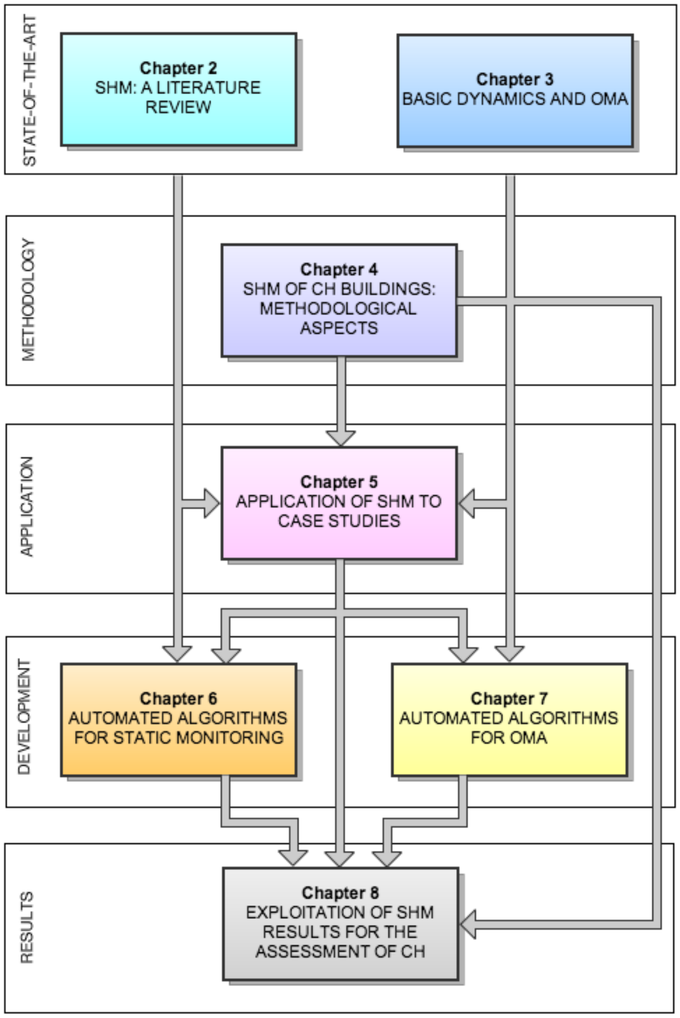


Fig. 1.2 - Thesis flowchart

2 SHM: A LITERATURE REVIEW

2.1 Introduction

The main goal of the present chapter is to review the most important developments in the field of Structural Health Monitoring (SHM). Initially a brief introduction on the application of monitoring techniques to several typologies of civil engineering structures during the last century is reported, with particular attention to its implementation on ordinary buildings and Cultural Heritage (CH) structures. Then the monitoring process is defined and its different phases described in detail. Addressed issues include sensors selection, signal processing, environmental effects and damage detection techniques. A state of the art review of automated data processing algorithms and software for SHM are also presented. Finally a summary is given with the main conclusions.

2.2 Application of SHM to civil engineering structures

Historically, the application of SHM involved several disciplines (starting from mechanical and aerospace engineering) and moved in the last decades to the field of civil engineering structures and infrastructures, including new and historic buildings, bridges, tunnels, factories, power plants, nuclear and conventional, offshore oil platforms, port facilities and geotechnical structures such as foundations and excavations.

Depending on the importance, use and risk levels, the companies that manage these structures and infrastructures have specific programs for inspections, monitoring and maintenance that may be compulsory by law and whose effectiveness is valid only if there is a good ability to establish promptly accurate performance levels.

Key sectors in the field of SHM have been the oil industry, management companies of dams and highways, whose facilities have received the greatest attention and research effort. So much recent attention in the civil SHM community has been

focused on bridges that it has overshadowed the formal application of SHM technology to other infrastructures such as dams (Ross & Matthews 1995).
Residential and commercial structures have received relatively little attention due to the lack of knowledge of the owners on the benefit and importance of structural monitoring. In these cases, it is necessary to make considerable efforts to educate owners also by means of specific legislation and insurance premiums (Chang 2000).
A major challenge in the development of a strategy for SHM for civil infrastructure is that, except for certain types of public and private housing, each structure is unique. This means that there is not a common basis repeatable in every situation. Thus, a specific feature of SHM for civil engineering structures is that an important part of the system should be oriented towards a long-term assessment of the so-called structural performance under operational or health conditions (Aktan 2001).
Ross & Mattews (1995) and Mita (1999) identified the cases where structural monitoring may be required:

- Modifications to an existing structure,
- Monitoring of structures affected by external works,
- Monitoring during demolition,
- Structures subject to long-term movement or degradation of materials,
- Feedback loop to improve future design based on experience,
- Fatigue assessment,
- Novel systems of construction,
- Assessment of post-earthquake structural integrity,
- Decline in construction and growth in maintenance needs, and
- Move towards performance-based design philosophy.

In the following paragraphs a review on the application of SHM to civil engineering structures and infrastructures firstly proposed by Brownjohn (2007), and integrated with additional parts, is reported.

2.2.1 Dams

Compulsory legislation, requiring regular inspection of dams, was born for the first time in Great Britain, due to the collapse of a 30 m embankment dam that caused the death of 254 people near Sheffield, in 1864. The legislative framework in this area has evolved and currently is in force the Reservoir Act of 1975, establishing the figure of supervising engineer for continuous surveillance of a reservoir and dams, including the interpretation of operational data (DETR 2001). Thus, dams are

historically the first class structure for the mandated application of SHM, and there is much to learn from this experience that can be applied to other structures.
In this field, the principles of SHM are stated in ANCOLD (1994) and more in detail thanks to the International Commission on Large Dams (ICOLD 2000):

- Range of tools and instrumentations to provide response data, supplemented by visual inspections;
- Need for automatic data collection;
- Intelligent interpretation of data against established behaviour patterns and identification of anomalies.

Fig. 2.1 - Collapse of the embankment dam of Malpasset in 1959

Regarding the third item in UK this role is typically entrusted to the supervising engineer, while in Italy a big scientific effort in the field of artificial intelligence (AI) was made by the ISMES, the research institute of the Italian electricity utility ENEL. Every major dam in the ENEL inventory of over 260 structures is equipped with transducers activated by central processor at regular intervals to measure static ‘structural effects’, such as:

- Relative or absolute displacements: horizontal crest displacements are the most important for concrete dams;
- Strains (for concrete dams) with temperature correction;
- Uplift pressures quantifying loads, which, for example, contributed to the failure of Malpasset Dam in 1959 (Fig. 2.1);
- Seepage rates.

Transducers are also activated to record ‘external influences’ to which the dam responds with structural effects, for example: water level, structural temperature and meteorological conditions.
In particular two AI applications were developed: DAMSAFE and MISTRAL, described by Salvaneschi *et al.* 2006. The first one assists engineers with dam

safety management procedures, whereas the latter is a real-time system that considers groups of effects with or without relation to influences.
The control of the dynamic response of dams by SHM plays an important role for two reasons. First, earthquakes are a serious threat to the safety of dams and every opportunity is used to improve the understanding of seismic performance through calibration of models and simulations (Severn *et al.* 1981). Secondly, the estimates of the dynamic characteristics obtained from ambient monitoring (Darbre & Proulx 2002) or forced vibrations (Bettinali *et al.* 1990) provide a means to monitor the structural characteristics, as indicators of structural health.

2.2.2 Offshore oil platforms

In 1970, the energy crisis and the discovery of large oil reserves in the North Sea has led to rapid developments in the field of offshore infrastructure, including production facilities of concrete and steel that operate at depths equal to or greater than 150 meters and subjected to extreme environmental loads. With the mandatory requirements (Det Norske Veritas 1977) for inspection as well as the expense and danger to the underwater inspections, began a growing interest in diagnostic systems based on vibrations (Coppolino & Rubin 1980; Shahrivar & Bouwkamp 1980; Brederode *et al.* 1986).
Around the same period of time, a large number of techniques for the dynamic identification of structures from ambient vibrations were developed by Peeters & De Roeck 2001, precursors of the modern discipline of Operational Modal Analysis (OMA). Procedures such as the maximum entropy method or the random decrement were applied to platform response data to improve the reliability and accuracy of the estimates of the dynamic parameters for the vibration-based diagnostic.
It should be noted, finally, that a particular problem for offshore installations is that the structure is a non-stationary system with constant changes in the mass properties, due to structural changes, loading and unloading of material, fluid movements in processing plants and drilling operations and therefore this complexity also affects largely the possibility to apply successfully SHM techniques and damage detection algorithms.

2.2.3 Bridges

In the last decades a large number of SHM systems have been applied to bridges and large infrastructures (Peters & De Roeck 2001; Pines & Aktan 2002; Ko & Ni 2005; Zhang & Zhou 2007; Ou & Li 2010; Magalhaes *et al.* 2012).
One of the first documented examples of systematic monitoring of bridges was conducted on the Golden Gate and the Bay Bridge in San Francisco in a complex measurement program of the various components during their construction in order to understand the dynamic behavior and the possible consequences of an earthquake.
The University of Washington in 1954 described the monitoring of the Tacoma Narrows Bridge during its short life cycle before the collapse because of wind-induced instability (Fig. 2.2).

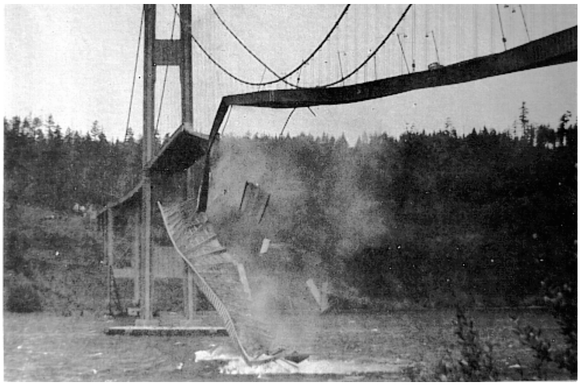


Fig. 2.2 - Collapse of the Tacoma Narrow Bridge in 1940 due to wind-induced instability

Over the past decade, permanent monitoring programs of bridges evolved into SHM systems implemented in Japan, Hong Kong, and later in North America.
In Hong Kong, the Wind and Structural Health Monitoring System (WASHMS) is a sophisticated monitoring system of bridges between the city and the airport, consisting of about 900 sensors installed by to the Hong Kong Highways Department.
In the U.S. the FHWA (*Federal Highway Administration Research*) has engaged in a research project called EAR (*Exploratory Advanced Research*) on the development of monitoring-based systems, due to the significant deterioration of the highway network. In 2002 a guideline on the *Development of a Model Health Monitoring Guide for Major Bridges*, commissioned by FHWA, was published, as a starting point to create an unambiguous national standard on bridge monitoring and to sensitize owners and management companies on these topics, standardizing the various methodologies that can be applied. The Minnesota Department of Transportation (Mn/DOT) was involved in the research project *Bridge Scour*

Monitoring Technologies: Development of Evaluation and Selection Protocols for Application on River Bridges that brings together the expertize of bridge engineers and researchers, university hydraulic and electrical engineers, field staff, and inspectors to take the first steps toward development of robust scour monitoring for Minnesota river bridges (Lueker et al. 2010).

Modern long-span suspension bridges typically have elaborate inspection and maintenance programmes, so that significant damage and deterioration of the superstructure is likely to be picked up visually, whereas an SHM system would require a high density of sensors to detect it. It is probably that only global changes such as foundation settlement, bearing failure or major defects, such as loss of main cable tension or rupture of deck element, are detectable by global SHM procedures with a minimum of optimally located sensors (Brownjohn 2007).

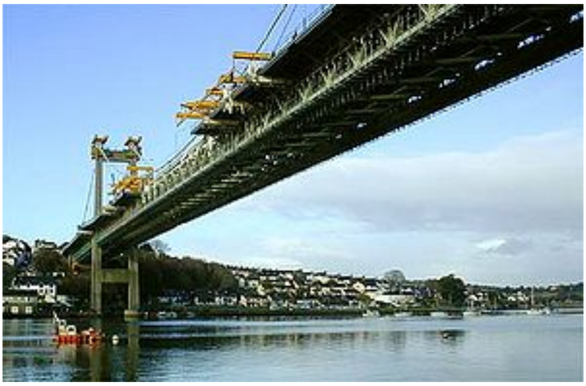


Fig. 2.3 - Tamar Bridge in the UK during the retrofitting works, 1999

Monitoring systems are also implemented in the context of management programs and retrofitting projects of existing bridges, in order to evaluate the current structural conditions and validate the effectiveness of subsequent strengthening interventions (Yanev 2003). Some examples of such use are the Severn and Wye Bridge refurbishment (Flint & Smith 1992), the Tamar Bridge in the United Kingdom (List 2004) (Fig. 2.3) and the Pioneer Bridge in Singapore (Browjohn et al. 2003). SHM in the framework of bridges construction, maintenance and management can provide adequate confidence level to the investors and information on the real health conditions of the infrastructures at the end of the concession.

Although the legislation on monitoring is still incomplete, regarding bridges is to be remembered that in 2003 the Committee ISO TC 108/SC 2 published the *ISO / DIS 14963 (E)* standard: *Mechanical vibration and shock - Guidelines for dynamic tests and investigations on bridges and viaducts* and the following year the *ISO / DIS 18649 (E)*: *Mechanical Vibration - Evaluation of measurement results from dynamic tests and investigations on bridges*, in which detailed and accurate procedures for dynamic testing of bridges and viaducts are defined. In addition in *Eurocode 2, Part 2* (*Design of concrete structures - Concrete bridges - Design and construction details*) on cable-stayed bridges the role of monitoring is mentioned especially during the installation of the stays, the inspection of the deck, the alignment of the piers. During measurements particular attention has to be paid to the record of forces and ambient temperatures.

2.2.4 Nuclear power plants

With the emergence and widespread use of nuclear energy, especially after the major nuclear disasters of Three Mile Island (USA) in 1979, Chernobyl in 1986 and Fukushima in 2011, the safety problem become crucial, imposing controls and continuous monitoring. Although power plants are designed to withstand earthquakes, the design project cannot take into account earthquakes of extreme magnitude and peak ground accelerations due to evident technical and economic difficulties in the design of structures subjected to such heavy seismic actions.
Smith (1996) and Smith & McCluskey (1997) provide an overview of the inspection and monitoring regime for a sample of the UK's civil nuclear reactors. For the safety-critical structural components of nuclear reactors, instrumentation for measuring structural response is used to validate and calibrate designs during performance testing and also contributes to the condition monitoring during normal operation.

2.2.5 Tunnels and excavations

The monitoring of tunnels is aimed at the observation of their deformation in terms of stability limits and the effects on or from adjacent structures (Okundi et al. 2003). Monitoring of heritage and other structures during nearby tunnelling or mining is a major concern; examples include the monitoring of Mansion House in London during construction of extension to an underground railway (Prince *et al.* 1994) and monitoring of listed nineteenth century mining facilities in Australia close to explosive blasting in nearby open-cast mining operation (Roberts *et al.* 2003). These ground surface monitoring exercises are temporary, but feature all the technology of permanent monitoring systems (Brownjohn 2007).

2.2.6 Buildings

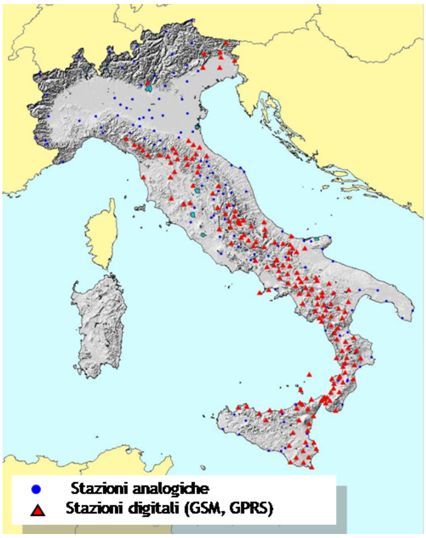
Historically, developments in monitoring of the buildings have been motivated by the need to understand their performance during earthquakes and storms. Ambient vibration tests were performed since the 80's (Hudson 1977; Jear & Ellis 1981) to understand the dynamic response of buildings. However, the knowledge of the dynamic behaviour of structures during major events, such as earthquakes, led to the implementation of permanent monitoring systems. In California, mandatory structural health monitoring is managed by the California Strong Motion Instrumentation Program (CSMIP) (California Geological Survey 2003), which performed the installation of accelerometers on buildings and other structures (Fig. 2.4). While such data can provide feedback on the structural health, the goal is to obtain information to improve the design of structures subjected to earthquakes. The need to identify full-scale structural performance has always been central to research in seismic engineering.

Fig. 2.4 - California Strong Motion Instrumentation Program

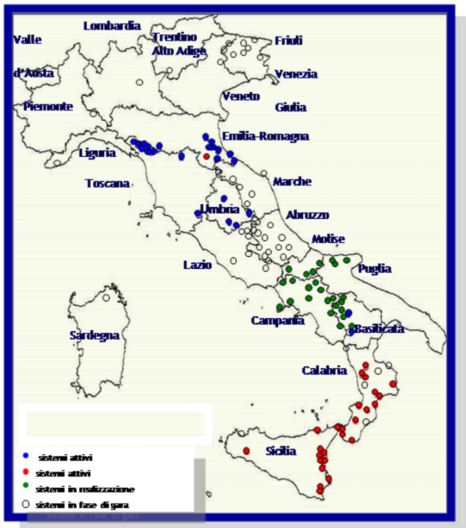
Most of the monitoring activities on buildings and towers have been performed to improve the understanding of loads and response mechanisms, not only against earthquakes, but also wind loads, for example the Bank of Commerce Building in Toronto (Dalgleish & Rainer 1978) and Hume Point in London (Littler & Ellis 1990).

Even more recently, skyscrapers in Dubai (but also in other areas of the world) are equipped with monitoring systems and modern technologies such as the use of TMD (Tuned Mass Damper or harmonic absorber). This approach falls in the idea of creating *smart structures* (Dyke *et al.* 2003) able to reduce the human intervention to a strict minimum (*Self-healing systems*) (Ghosh *et al.* 2007).

Significant motivation for SHM of buildings has also resulted from recent major earthquakes, such as in Kobe, Japan in 1995 and Northridge in California in 1994, where timely information on the status of the structures would be invaluable to assess the safety and the need for interventions (Mita 1999). These events created an opportunity for both the birth of some organizations that provide accurate data after major seismic events and support citizens during emergency actions (such as ANSS*, Advanced National Seismic System* and FEMA*, Federal Emergency Management Agency* in the USA *or RAN, National Strong Motion Network* in Italy, (Fig. 2.5a) and the development of an integrated SHM approach involving autonomous sensors, embedded systems, communications, data management, etc.. .



a)



b)

Fig. 2.5 - a) Italian National Strong Motion Network (RAN); b) Seismic Observatory of Structures (OSS)

In Italy is also active the OSS (*Seismic Observatory of Structures*) (Fig. 2.5b), a national network of permanent monitoring of the seismic response of buildings, designed, built and managed by the Department of Civil Protection (DPC), with both cognitive and control purposes. The OSS allows assessing damages caused by an earthquake to the monitored structures or similar buildings in the same seismic crater, providing useful information for the planning of the post-earthquake activities and the emergency management of the Civil Protection immediately after a big seismic event. Data are collected in an acquisition unit of DPC and analyzed and distributed via Internet. The OSS consists of the so-called sub-network of the *Fundamental Samples* (105 schools, hospitals and town halls, 10 bridges and several dams, subjected to sophisticated monitoring systems provided with complete dynamic monitoring devices, based on a layout of 16 to 32 acceleration transducers) and of the sub-network of *Integrative Samples* (300 strategic buildings for the management of the emergency activities, equipped with a simplified monitoring system, based on 7 acceleration transducers).

Regarding the legislation aspects connected to the application of SHM on civil engineering buildings, despite the growing interest in Europe on those topics, only a short comment is introduced in the *Eurocode 8* (*Design of Structures for Earthquake Resistance*) in Appendix B: "For the assessment of structures, in addition to gather general information and historical data, during the inspection phase tests and in situ measurements has to be performed studying the evolution over time of dimensions, alignments, eccentricities, crack pattern and deformations, especially in case of aftershocks (with the possible installation of monitoring systems)". There is also the ISO 16587 *Mechanical vibration and shock - Performance parameters for condition monitoring of structures* issued in 2004 but only marginally addresses to this topic.

However, some guidelines and codes were issued for condition monitoring of structures, using parameters typically used to measure or monitor structure performance (Maguire 1999; Mufti 2001).

In addition, several national and international groups with a strong scientific interest on SHM created some research networks, such as SAMCO (*Structural Assessment, Monitoring and Control, www.samco.org*) (Rucker *et al.* 2006), ISIS (*Intelligent Sensing for Innovative Structures, www.isiscanada.com*) and ISHMII (*International Society for Structural Health Monitoring of Intelligent Infrastructure, www.ishmii.org*). ISHMII's goal is to enhance the connectivity and information exchange between participating institutions and individual members and to increase the awareness of the structural health monitoring disciplines and tools among end users.

In other cases, strict collaborations between industry and government organizations have been promoted, such as SIMONET (*Structural Integrity Monitoring Network, www.simonet.org.uk*) managed by the University College of London and the Cranfield University to facilitate communication between companies, researchers and all those interested in the field of structural health monitoring and non-destructive testing.

2.2.7 Cultural Heritage buildings

In recent years, in the scientific community it has been surveyed a growing interest in the definition and implementation of procedures for structural assessment and monitoring of monuments and historic buildings. Several national and international research projects, also funded by the EU, are investigating the possibilities and limitations of monitoring techniques and procedures as well as non-destructive testing for the assessment of masonry structures (ONSITEFORMASONRY 2002-2004; Improving the Seismic Resistance of Cultural Heritage Buildings 2004-2006; NIKER 2010-2012).

Although there is still not a reference standard, in some countries such as Italy, some guidelines have been issued on these topics. For example the relevance of monitoring as an appropriate tool for a correct conservation strategy of CH buildings is now clearly specified in the “Guidelines for the evaluation and mitigation of the seismic risk to Cultural Heritage buildings”, in line with the new “Technical Standards for Constructions”, Directive (2007), which contain a specific paragraph on the topic [§4.1.9 Monitoring], saying: “the assessment of the building carried out on a periodic basis is a strongly advisable practice since it represents the main tool for a correct conservation strategy, allowing to program maintenance activities and to carry out on time - if quantitatively demonstrated their real necessity - repair interventions, in case of structural damage, or strengthening, aimed to prevention. To define a monitoring protocol it is necessary to preventively execute a detailed structural analysis of the structural functioning, and then to obtain an interpretation of the ongoing deterioration processes, in a way to define the most relevant parameters which, continuously acquired or with adequate time steps, allow to define their satisfactory functioning or - on the contrary - to point out possible dangerous damage processes for the stability of the whole structure or part of it.

Visual inspections, intended as a periodic check of the onset of visible crack patterns, deterioration phenomena, transformations in the structure or surrounding environment can be considered the starting point of such activities. More specific information may be collected through instrumental monitoring of some parameters which can be considered meaningful from a structural point of view (cracks movements, absolute or relative displacements of specific points of the building, rotations of walls or other elements). […] Sometimes, when the deterioration process is properly identified, and safety thresholds can be defined, SHM may represent an alternative to the structural intervention, to the benefit of conservation. […] If seismic safety is in particular addressed, being the earthquake a rare and unforeseeable event, it is clear that SHM (as intended here) does not represent an early warning procedure or a method for assess the seismic behavior of the monitored structures”.

Also in the ICOMOS Charter - Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage (ISCARSAH Principles) similar concepts are presented and discussed: "The diagnostic is based on historical, qualitative and quantitative approaches: qualitative aspects are essentially based on the direct observation of structural instability and degradation of materials, as well as the historical and archaeological research, while quantitative aspects are essentially based on direct measurements, the investigation of materials and structures, monitoring and analysis on structural experienced by more or less sophisticated methods of calculation."

Recently, the application of SHM to historic structures and buildings has become in Italy more and more widespread due to the extensive and important cultural heritage patrimony of the country.

One of the first examples of SHM applied to cultural heritage is the case of the monitoring system of the dome of Santa Maria del Fiore in Florence (Bartoli & Blasi 1993), already active in 1988 and consisting of more than 160 static sensors due to the complexity of the crack pattern (Fig. 2.6).

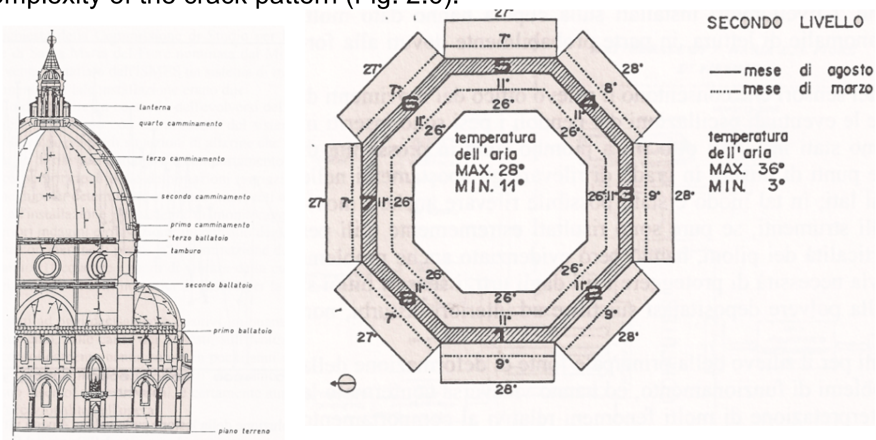


Fig. 2.6 - The monitoring system of the dome of Santa Maria del Fiore in Florence

The University of Padova has promoted and installed several monitoring systems on Cultural Heritage structures both in normal conditions (Modena *et al.* 2008; Casarin *et al.* 2008; Gaudini *et al.* 2008) and during the post-earthquake emergency activities in l'Aquila (after the Abruzzo earthquake in 2009) (Casarin *et al.* 2010; Binda *et al.* 2010).

The research group at the University of Trento developed a system of wireless sensor networks for permanent SHM of historic buildings (Zonta *et al.* 2010) and installed monitoring systems on some important historic towers (Pozzi *et. al* 2009; Zonta *et al.* 2008;).

Also Gentile & Saisi (2007) used ambient vibration testing for the structural identification ad damage assessment of historic masonry towers.

The research groups at the University of Minho in Portugal is very active in the field of SHM applied to cultural heritage buildings (Ramos *et al.* 2010) and damage identification on masonry structures based on vibration signatures (Ramos 2007).

The University of Pisa performed dynamic characterizations of masonry bell towers (Beconcini *et al.* 2006).
De Stefano & Ceravolo 2007 and De Stefano 2007 proposed a methodology for the assessment of the health state of ancient structures using vibrational tests and SHM. Moreover they suggest the use of structural safety formulations conceived to take into account the presence of periodic monitoring systems. Redefining structural safety in terms of residual lifetime provides the theoretical framework for the introduction of vibration-based monitoring activities in probabilistic formulations (Ceravolo *et al.* 2007).
Abruzzese *et al.* 2009 approached the study focusing mainly on wireless sensors, while Serino *et al.* 2009 published an interesting application of seismic isolation in combination with SHM.
Another recent application of SHM to architectural heritage buildings is proposed by Del Grosso *et al.* 2004 in the Royal Villa of Monza subjected to foundation settlement: a monitoring system has been installed to control its movement before and during the refurbishment works and the subsequent service life.
In conclusion it is possible to state that the application of monitoring techniques to cultural heritage structures is developing more and more and the future research will concentrate on the possibility of exporting methodologies and techniques, already widely used and tested in other sectors of civil engineering, also to the historic architectural heritage.

2.3 The monitoring process

Structural Health Monitoring is a broad multi-disciplinary field both in terms of the diverse science and technology involved as well as in its varied applications. The technological developments necessary to enable practical SHM are originating from scientists and engineers in many fields including physics, chemistry, materials science, biology, and mechanical, aerospace, civil and electrical engineering (Chang *et al.* 2002).
Monitoring is a non-invasive survey process of significant structural quantities or responses under known or unknown actions, in which data are recorded over a predefined period of time. Relevant parameters are monitored continuously or at short/medium time intervals in order to evaluate the persistence of the same structural state as defined at the beginning of the monitoring period, allowing to identify the presence of structural damages.

The SHM process can be subdivided into three main stages, starting from the classification proposed by Kullaa 2008 (Fig. 2.7):

(i) Instrumentation and data acquisition;
(ii) Signal processing and feature extraction;
(iii) Damage detection, alarms and reports;

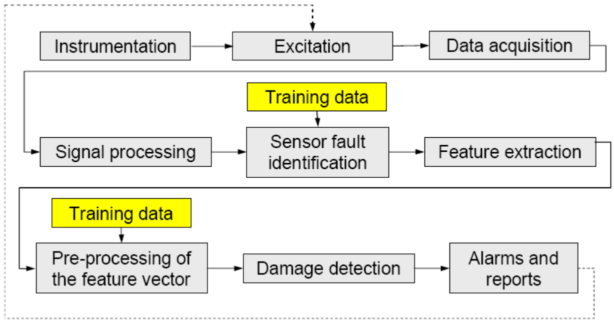


Fig. 2.7 - Structural Health Monitoring process (Kullaa 2008)

In the following section the monitoring process will be described in detail.

2.3.1 Instrumentation and data acquisition

The first stage of SHM begins to set the limitations on what will be monitored and how the monitoring will be accomplished, including economic justification for performing SHM (Farrar *et al.* 2001). A monitoring system is composed by: (i) sensors; (ii) acquisition unit (including a signal conditioning process and an *Analog-to-Digital Converter*-ADC); (iii) database for data collection and storage (Fig. 2.8).

From a theoretical point of view, any structure can be instrumented at each significant point that can be useful to understand the structural behavior or to identify on-going damaging processes. On the practical side, however, the number of sensors to be implemented for an efficient monitoring, is closely tied to the available financial resources, in terms of instrumentation and data acquisition equipment.

It is therefore of major importance to minimize the number of transducers on the structure, to be able to control the fundamental parameters, avoiding any loss of information. Once identified the correct location and position of sensors, the response of the structure has to be measured. Data recorded by sensors are sent to peripheral Data AcQuisition system (DAQ) that amplify, filter and convert the acquired signals through discrete time series at a specific sampling rate.

The monitoring strategy selection is an important phase which include both the decision of quantities to be measured and the selection of transducers and their positions.

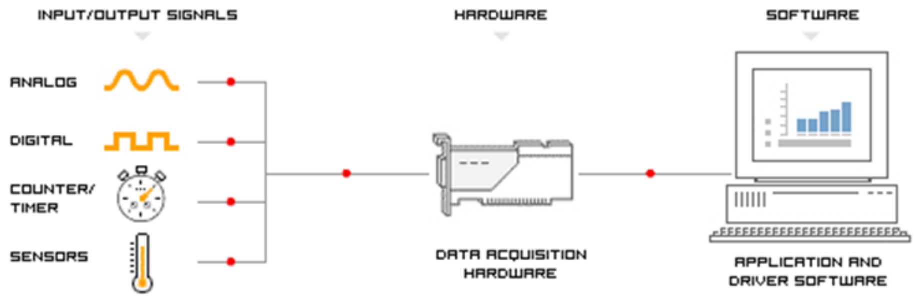


Fig. 2.8 - Scheme of a monitoring system

2.3.1.1 Monitoring strategy: static vs. dynamic

The monitoring strategy to be applied includes a distinction between *static monitoring*, aimed at the continuous measurement of gradually, slow-varying parameters over a long period, and *dynamic monitoring,* oriented to the control of dynamic properties of the monitored structure both under operational conditions (cyclic variation of modal parameters) and during exceptional events (e.g. earthquakes).

Static monitoring requires the regular measurement of small variations over long periods of time. In principle, a few measurements per minute, or even per hour, may be enough to characterize the variations caused by daily climatic cycles or other periodical or gradual effects.

Dynamic monitoring is intended to characterize the dynamic or seismic response of the building. It can be carried out punctually or periodically by means of dynamic tests measuring the vibration characteristics of the structure induced by forced or natural phenomena. Another possibility is to install a permanent system capable of self-activating and capturing the motion of the structure at every occurrence of a micro-tremor or any other significant vibration source above a certain threshold (trigger-based monitoring). Finally, continuous dynamic monitoring is also possible and need the implementation of huge storage capacity acquisition systems. Dynamic monitoring is largely implemented both to implement damage detection algorithm based on changes of modal parameters of the structural system (i.e. natural frequencies, mode shapes and damping ratio) that might be related to damage and to control its dynamic response during significant events (e.g. strong wind events, micro-tremors or earthquakes).

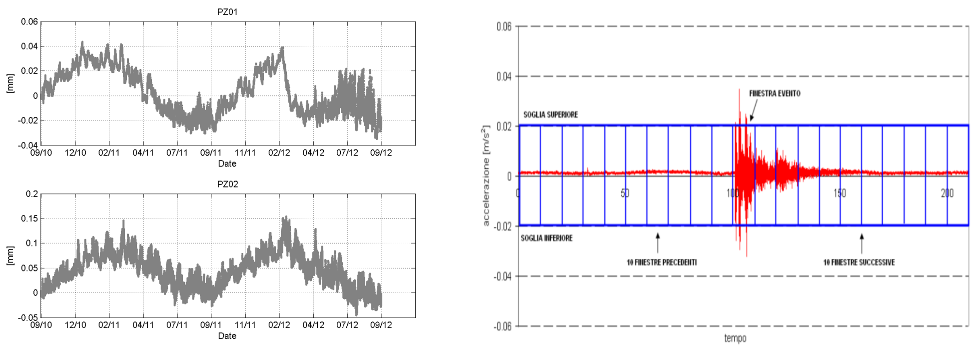


Fig. 2.9 - Static monitoring (left): cyclic daily and seasonal variation of a monitored crack; dynamic monitoring (right): trigger-based monitoring to control the dynamic behavior under operational condition and during exceptional events.

2.3.1.2 Requirements

In the design of an automatic monitoring system the following aspects need to be carefully evaluated:

- Environmental conditions: the choice of instrumentation and wires scheme should be decided after a detailed analysis of the environmental conditions to ensure the necessary protection of the system, the absence of electrical noise and accessibility conditions for sensors' installation.
- Accuracy: it has to be defined analyzing all types of errors (systematic or random) that might affect the instrumentation. It is important to evaluate not only the precision of each transducer but also of the entire system.
- Reliability: Since usually a monitoring system is used on a permanent basis, a high level of reliability is required. It is therefore necessary to detect automatically possible sensor faults or system malfunctioning.
- Flexibility: System updating, substitution and/or integration of some sensors may be necessary during the life-cycle of the system.
- Maintenance: periodic inspections must be carried out to ensure the reliability of the system and are strongly necessary after exceptional events such as earthquakes.

2.3.1.3 Sensors typologies

A transducer is a device able to convert a physical quantity which usually defines the system response (such as displacements, velocities, accelerations, stresses, strains, forces, etc..) into a proportional electrical signal, processed by the data acquisition unit. The recorded signal can be analog or digital, active or passive (in this case it requires an external power source). Sensors have to satisfy the following performance characteristics: sensitivity, resolution, range, linearity, hysteresis, accuracy, error, precision, isolation, low cost, ease of installation and durability.

In the case of an analog sensor the output variable is continuous and requires an A/D conversion, while in the case of a digital sensor the conversion is not required since the output is internally converted by the sensor itself (Fig. 2.10).

•ANALOG: temporal and spatial continuos

temperature	Thermocouple, pt100	ΔV, ΔΩ
load/force	Load cell	ΔΩ
strain	Strain gauge, fiber	ΔΩ
acceleration	Accelerometer	ΔQ
sound	Microphone	ΔQ
displacement	Potenziom, LVDT, resolver	ΔV

•DIGITAL: temporal discrete

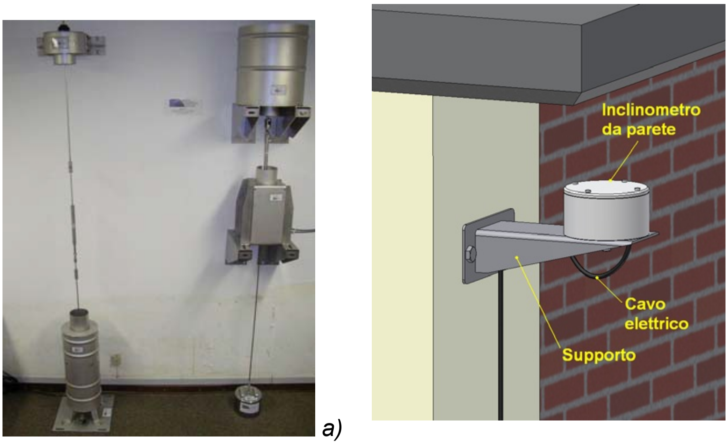
displacement	Encoder, CCD, Laser	pulse
state	Limit switch	ΔV
frequency	counter	pulse

Fig. 2.10 - Analog and digital sensors for SHM

Instrumentation and sensors commonly implemented into monitoring systems are listed and briefly described hereafter. Two main categories of sensors are reported: *a) common sensors* and *b) innovative sensors*

a) Traditional sensors

- *Pendulum*: Hanging or inverted Pendulums are used for accurate and long term monitoring of horizontal structural movements of large structures such as dams, bridges and other tall buildings (Fig. 2.11a). The upper end of a steel wire is anchored to the structure under observation. A weight suspended from the lower end is free to move in an oil tank, the oil serving to damp oscillations of the wire. Displacements relative to the wire may be measured using a portable optical readout unit or for remote reading, an automatic x- y coordinator.
- *Servo-inclinometer probes*: allow to detect relative displacements in underground or above ground structures.
- *Inclinometers*: measure the variation of inclination of a structural element. The output signals are proportional to the angle of inclination reached by the instrument with respect to the vertical direction (Fig. 2.11b).



a) b)

Fig. 2.11 - (a) Inverted and hanging pendulum systems; (b) Inclinometer

- *Strain gauges*: measure strains as an elongation between couples of structural points (ε components of the strain tensor) (Fig. 2.12a). There are two types of commonly used sensors as reported in Ko & Ni 2005: electrical resistance strain gauges and vibrating wire strain gauges. Widely developed in several monitoring systems are also fiber optic strain gauges such as the Fiber Bragg-Grating (FBG) as reported in literature (Maaskant et al. 2007; Todd et al. 1997; Ou & Li 2010).

- *Displacement transducers (linear variable differential transformer LVDT)*: used to describe the development over time of the crack pattern in specific area of the structure accurately selected (Fig. 2.12b). In the general case two transducers are implemented in the plane of the structural element to follow both the axial deformation (contraction or expansion along the perpendicular direction of the crack) and sliding phenomena. If possible, redundant measures ensure high reliability of the recorded data.

b)

a)

Fig. 2.12 - a) Strain gauges for steel elements: b) Displacement transducers (LVDT)

- *Thermocouples* (or *Thermometers*): to measure air temperature. In order to measure temperature on a structural element at specific position of interest thermally sensitive resistors such as Thermistors and Resistance Temperature Detectors (RTD) are generally implemented.
- *Hygrometers*: to measure relative humidity of the air.
- *Anemometers*: to measure speed or wind pressure.
- *Accelerometers*: to measure accelerations induced by natural or forced vibrations. The operating principle is based on the detection of the inertia of a mass when subjected to an acceleration.

  Acceleration transducers can be classified into: (i) Piezoelectric accelerometers; (ii) Piezoresistive and capacitive accelerometers; (iii) Force-balance accelerometers.

  Piezoelectric accelerometers are one spring-mass-damper system which produces signals proportional to the acceleration in a frequency band below their resonant frequency. The active element of the accelerometer is a piezoelectric material. One side of the piezoelectric material is connected to a rigid post at the sensor base. A so-called seismic mass is attached to the other side. When the accelerometer is subjected to vibration a force is generated which acts on the piezoelectric element. This force is equal to the product of the acceleration and the seismic mass. Due to the piezoelectric effect a charge output proportional to the applied force is generated. Advantages of this type of accelerometers are: active transducers, i.e. do not use external power source, stability, good signal-to-noise ratio, linear behavior over a wide frequency and dynamic range. The main disadvantage is that they are not able to measure the DC components (0 Hz), like the acceleration of gravity *g*. This characteristic is unfavorable to measure very flexible structures (such as suspension bridges).

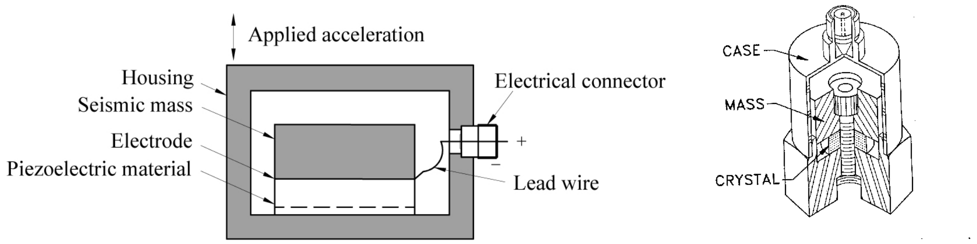


Fig. 2.13 - Piezoelectric accelerometer: cross section and 3D view (PCB, 2004)

  Piezoresistive and capacitive accelerometers are structured with a diaphragm, which acts as a mass that undergoes flexure in the presence of acceleration. The piezoresistive ones are composed by a cantilever beam with a mass at the free end. Strain gauges are fixed to the beam and, when excited, the differential output signals of the strain is proportional to the acceleration. In capacitive accelerometers two fixed plates sandwich the diaphragm, creating two capacitors, each with an individual fixed plate and each sharing the diaphragm as a movable plate. The flexure causes a capacitance shift by altering the distance between two parallel plates, the diaphragm itself being one of the plates.

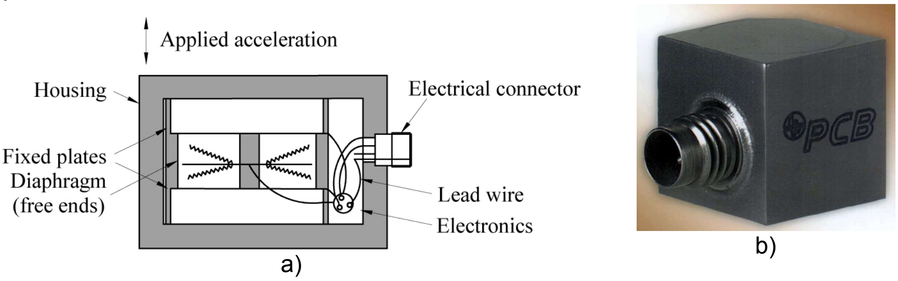


Fig. 2.14 - Capacitive accelerometer: cross section and triaxial model which simultaneously measures vibration in 3 orthogonal directions

Force-Balance accelerometers: Unlike conventional accelerometers, the servo type contains a freely suspended mass constrained by an electrical equivalent mechanical spring. There are two classes of servo force balance accelerometers: the pendulous type, having an unbalanced pivoting mass with angular displacement, and the non-pendulous type, having a mass which is displaced linearly. The force balance sensor is intended for DC and low frequency acceleration measurements, such as those encountered in the motion of vehicles, aircrafts and ships. These sensors are capable of measuring levels from as low as 0.0001g up to 200g's over a frequency range from DC to 1000Hz. In addition, due to their inherent sensitivity to gravity, the force balance accelerometers with certain modifications or special features become excellent instruments for measuring angles of inclination. This type of sensor, often referred to as an inclinometer

b) Innovative sensors

In the following section some innovative sensor devices and data acquisition and transmission units are reported, according to the state of the art technology development.

- *Fiber Optic sensors*: In recent years, fiber optic sensors have assumed considerable importance in the field of SHM (). They represent the ideal choice for many applications, being easy to handle, dielectrics, immune to electromagnetic interference and capable of detecting very small deformation with high accuracy (2 microns, regardless of the length of the measurement base) for long periods of observation, although costs are higher than the traditional methodology (Moss & Mattews 1995). The *Applied Computing and Mechanics Laboratory* (IMAC, *Ecole Polytechnique Fédérale de Lausanne*) and the *Institute of Mechanics of Materials* (IMM, Lugano) developed a measurement system called SOFO (*Surveillance of Ouvrages par Fibers Optiques*, Structural Health Monitoring Fiber Optic), based on low-coherence interferometry in singlemode optical fibers that allows the measurement of deformations of the order of 1/100 mm. This system is especially useful for the long-term monitoring of civil structures such as bridges, tunnels, dams and geostructures. The SOFO system requires the installation of two fibers in the structure to be monitored. The first fiber should be in mechanical contact with the structure in its active region and follow the structure deformation in both elongation and shortening. The second fiber has to be installed freely in a pipe near the first one. This fiber acts as a reference and compensates for the temperature dependence of the index of refraction in the measurement fiber (Inaudi *et al.* 1996).

The SOFO system has been used successfully in Europe, USA and the Far East. In Italy there have been some interesting applications thanks to the collaboration between SMARTEC, TECNITER and the Department of Structural and Geotechnical Engineering, University of Genoa. Recent developments allow its use even for dynamic measurements (Inaudi 2004)

Fig. 2.15 - Fiber Optic Sensors

- *Piezoresistive cement-based strain sensors*: Piezoresistive cement-based material is a relatively new civil engineering material produced by adding an appropriate amount of short carbon fibers into cement-based material. It can sense the stress and strain in it better compared to conventional cement-based material (Ou & Han 2008). Extensive studies on the percentage of mixture, manufacturing process, properties and methods of measurement of the detection sensors, have been performed even if further research on these topics is necessary (Han *et al.* 2008)

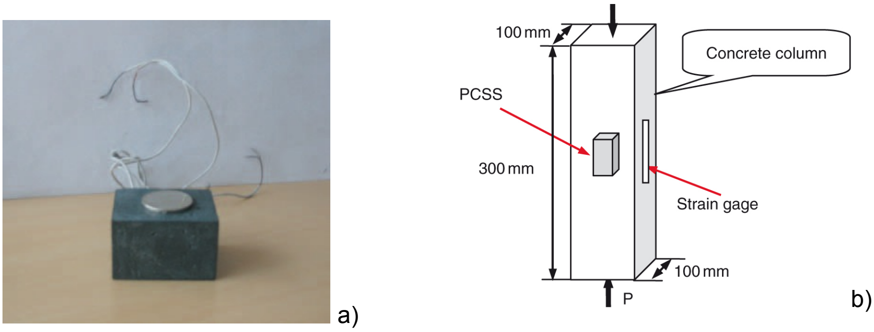


Fig. 2.16 - a) Piezoresistive cement-based strain sensors (PCSS); b) testing setup of PCSS in a reinforced concrete column subjected to axial load (Ou & Han 2998)

- *Corrosion monitoring sensors*: Corrosion can cause a severe deterioration of the structural performance of a mechanical or civil engineering system. Thus it is important to monitor precisely damages induced by this phenomenon. A sensor for measuring such a severe problem with high accuracy and durability has been developed and verified experimentally by Qiao & Ou 2007. Time-frequency analysis approach (wavelet transform) is used to diagnose the presence of corrosion. In the electrochemical corrosion, the electric power generated by the reaction can be collected and then used as a battery for wireless sensors; this type of sensor is called *self-harvesting wireless sensor* (Li *et al.* 2008).
- *GPS systems*: GPS has provided new possibilities for the direct measurement of displacements in the structures, substituting more traditional solutions such as communicating vessels, the plumb line or the laser interferometry. A differential GPS, which depends on the accurate measurement of the flying time of radio waves from satellite to receiver, allows a direct measurement of absolute deflections avoiding several problems associated with traditional optical systems. Some recent projects demonstrate the potentiality of GPS systems for SHM such as a radio tower in Japan (Li *et al.* 2004) and a group of three high-rise buildings in Chicago, where a fast post-processing procedure of data acquired by GPS systems is implemented (Kijewski *et al.* 2003).
  The most successful application of GPS systems is the SHM of long suspension bridges, characterized by low frequencies and thus slow displacements induced by ambient vibrations. Interesting examples of this kind of application is reported by Wong *et al.* 2001.
- *Wireless sensor network (WSN)*: a WSN can be described as a network comprising a set of distributed sensors, communicating between each other in order to detect, share and process data acquired from the physical environment. Typically these networks are composed by a number of elements

able to communicate with each other, called nodes. A node usually works thanks to a battery and it is equipped with a variable number of sensors or actuators. Wireless sensor networks (WSNs) are an attractive technology: compared to traditional wired systems, they consistently reduce the installation time and costs, as described by Celebi 2002, and are not subjected to wires wear and tear or breakage caused by harsh weather conditions or other extreme events. However, the wireless sensor nodes forming the network do not guarantee the same reliability and speed of the data link: radio packets can go lost during the transmissions, and the employed transceivers usually have limited power and bandwidth (Bocca *et al.* 2009). However, the large diffusion of this technology in recent years, even if not standardized yet, allowed the researchers to apply extensively WSN especially in the field of Civil Engineering and, particularly, for SHM. In the last decade, an intensive research effort have been made on these topics, as described by Farrar & Doebling 2001. Other methods have been designed specifically for wireless sensor nodes, able to process collected data locally and reduce the number of radio transmissions in the receiver node of the network, as proposed by Lynch 2004 and Mizuno *et al.* 2008. Wirelss sensor nodes and WSNs for SHM have been an object of intensive research. Prototypes have been proposed by Paek *et al.* 2005, Kim *et al.* 2007, Chen & Tomizuka 2008 and Ceriotti *et al.* 2009.

2.3.2 Signal processing and feature extraction

Signal processing is typically adopted in the field of dynamic monitoring to extract features directly or to act as a pre-processing step in the feature extraction. Signal processing extracts some useful information from the time histories using their stochastic properties or certain assumptions (Kullaa 2008).

Some of the most important and common signal conditioning functions implemented during the signal processing phase are presented and briefly described hereinafter.

Usually signal information are acquired in time domain but, for practical reasons and computation requirements, frequency domain is preferred. The *Fast Fourier Transform* FFT (Cooley & Tukey 1965) is an important tool to convert signals between the time domain and the frequency domain. The method is fast and no information is lost in the transformation.

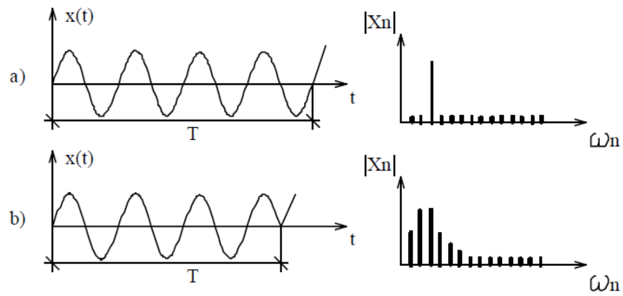


Fig. 2.17 - Leakage error: a) time history and corresponding spectrum when the acquisition time is an integer multiple of the signal period; b) leakage error occurs when the measuring time T is not a multiple integer of the signal period

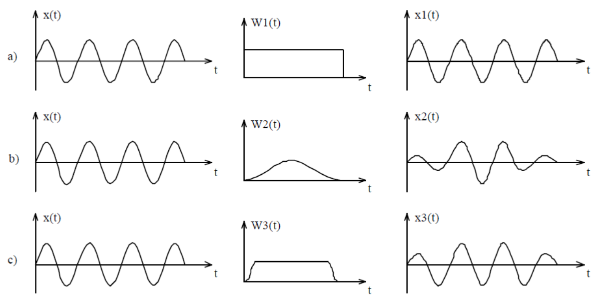


Fig. 2.18 - Signal transformation through the windowing techniques to reduce leakage: a) Box window; b) Hanning window; c) Cosine taper window

However the frequency domain brings an additional problem known as *leakage error* caused by the fact that FFT is defined for time series of infinite length, whereas the acquired time histories are finite (Fig. 2.17). Leakage can be reduced by increasing the sampling duration T so that the difference between the spectral lines is very small and minimizes the error. But the most effective technique for leakage error reduction is the signal transformation by the introduction of a window function (*windowing*), forcing the signal to be (almost) periodic (Fig. 2.18).

Another signal conditioning process related to windowing is *filtering* which produces modifications of the spectrum signal through the application of different types of windowing functions: low-pass, high-pass, band-limited, etc.

During the system identification phase it is also possible to apply a *decimation* procedure to the acquired time histories in order to alter their sampling frequency to a lower sampling rate and reduce drastically the frequency content of the signal to a range of interests for the investigated structural system.

Once the signals have been elaborated and modified it is possible to extract several functions from the time series: power spectral density functions (PSD), Impulse Response Functions (IRF), Frequency Response Functions (FRF). They can be exploited directly as features or as pre-processing step to extract other information.

The feature extraction stage is of fundamental importance in order to find those features that are sensitive to damage, filtering the effects of environmental or loading conditions.
Since monitoring systems produce a vast amount of data everyday it is crucial to automate as much as possible this important phase of the monitoring process.
Some features can be easily extracted automatically, such as information from static systems (e.g. opening of crack, variation of the inclination of structural elements, strains, etc.), while others may need a lot of supervision and rules to automate (e.g. modal parameters extraction). Typical features of a structure extracted from dynamic systems are natural frequencies, mode shapes, and modal damping. Other features include spectral functions, e.g. power spectra, FRF, AR coefficients, transmissibilities, wavelets, and modal filters (Kullaa 2008).
A specific section of this literature review is dedicated to the automation of the feature extraction phase, both for static and dynamic system and to development of dedicated software for the management of this stage. The reader may refer to §2.4 for a detailed state-of-the-art on this specific topic which is also one of the most substantial aspect of the present research.

2.3.3 Damage identification

In the most general terms, damage can be defined as changes introduced into a system that adversely affects its current or future performance. Implicit in this definition is the concept that damage is not meaningful without a comparison between two different states of the system, one of which is assumed to represent the initial, and often undamaged, state (Farrar & Worden 2007).
Changes in the behaviour of the structural system might be related to damage and are directly connected to the variation of physical or geometrical properties, boundary conditions or constraints of the structure.
All the materials employed in engineering systems have some inherent defects, which grow when the system is subjected to operating loads. In some cases this situation may occur in a relatively long period of time (e.g. in case of corrosion phenomena or settlement-induced damages) while in other cases damage may appear suddenly after an exceptional event (e.g. in case of earthquakes). Thus, the monitoring strategy must consider extent and time associated to the evolution of damage.
The identification of damage using vibration measurements is a well-established practice and its application to civil engineering structures dates back to early ‘80s. Its theoretical foundation derives from the principle that the dynamic response of the

structural system is affected by the alteration of the stiffness, mass or energy dissipation properties when damage occurs. The most widely accepted interpretation of the damage identification problem is that of statistical pattern recognition. In this approach the system is represented by a statistical model whose parameters are directly derived from the data. Each data (here referred as pattern) is condensed and expressed in terms of selected damage-sensitive features. The features extraction is generally recognized as the most crucial step in the diagnostic procedure. Its role is essential and it can highly bias the damage recognition stage (Ruocci *et al.* 2011). Deobling et al. 1996 reported an extensive literature review on damage identification and SHM of civil and mechanical systems from changes in their vibration characteristics.

The damage state of a system can be described according to the methodology proposed by Rytter 1993, where damage is classified into four levels:

1. Level One: Damage Detection
2. Level Two: Damage Location
3. Level Three: Damage Assessment
4. Level Four: Prediction

The method initially involves the identification of the damage present in the structure (level 1) and subsequently its geometric location in the building (level 2). In the third level is determined the extent and type damage and in the fourth level the useful residual life of the structure.

Farrar & Worden 2007 extend this classification clarifying that the damage state of a system can be described as a five-step process to answer the following questions:

1. Existence. Is there damage in the system?
2. Location. Where is the damage in the system?
3. Type. What kind of damage is present?
4. Extent. How severe is the damage?
5. Prognosis. How much useful life remains?

Here the identification of the type and the extent of damage is split into two levels. Damage identification can be performed in different states: a first microscopic state, useful to understand the damage level of material, a second macroscopic level, through the execution of local non-destructive testing and global structural monitoring. Finally, a third state to define the degree of damage of the entire system, by means of a statistical control of the entire process. Through this analysis it is possible to determine the safety conditions of the structure, e.g. the residual life (damage prognosis).

2.3.3.1 Data driven vs. Model driven approaches

It is possible to identify two different methods for damage identification: Model-Driven and Data-Driven approach (Worden & Manson 2008). Model-Driven approach defines a high-fidelity model of the undamaged structure or machine using physical laws based on first principles. Then changes in the measured data are related to modifications in the physical parameters via system identification algorithms based on linear algebra or optimization theory. Model-based approaches are generally more robust and can be implemented easily for the analysis of new or unforeseen situations since are able to integrate and replicate, through mathematical models, a wider range of behaviors, even if previously not observed in real systems. If the status of a system deviates from the expected operational behavior, it is possible to continue the analysis updating the physical parameters that describe the new situation. Thanks to this capability, this method does not use historical information that the Data-Driven Approach requires.

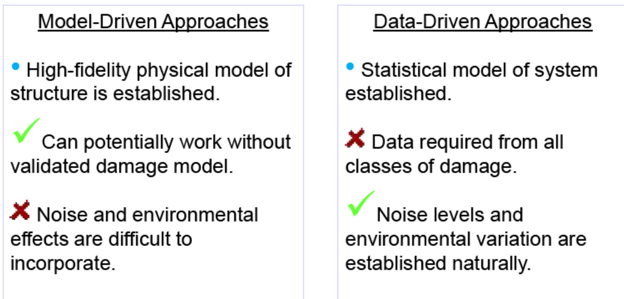


Fig. 2.19 - Model-driven vs. Data-driven approaches (Worden & Manson 2008)

Data-Driven approach treats damage identification as a pattern recognition problem. Pattern recognition algorithms are implemented to assign a damage class to the measured data of the system. They are based on comparative assessments of the health state of the system under investigation with other known events. Until the behavior of the monitored structure is similar to the reference healthy one, ongoing damaging processes can be excluded. When the measured behavior deviates from the reference one, a fault/damage is detected. There is no sensors capable of measuring damage. The latter, however, might be identified by processing the acquired data using three types of algorithm that can be distinguished depending on desired diagnosis: (i) novelty detection; (ii) classification; (iii) regression.

2.3.3.2 Environmental effects

Before applying any damage identification algorithms it is of fundamental importance to filter out the environmental and loading effects from the dynamic response of the structure and from the cyclic behavior of 'static' features (e.g. crack opening, strains, variation of the inclination of structural members, etc.). Both the static and dynamic behavior of a structural system is in fact strongly influenced by environmental effects, in particular temperature and relative humidity.

In case of masonry structures, it is expected that also moisture may have any influence, especially on the dynamic behavior. The possible effects on stiffness and mass change due to moisture is still a challenge to be addressed by scientific community (Ramos 2007).

Sohn 2007 and Moser & Moaveni 2011 review several studies which indicate that temperature, boundary conditions and mass loading effects largely influence the dynamic response of structures. For example temperature affects the Young's modulus of steel (Khanukhov *et al.* 1986) and concrete (Sabeur *et al.* 2007). In a nonlinear structure, dynamic properties change with excitation amplitude (Siringoringo *et al.* 2006). All these factors can influence the modal parameters of a bridge but temperature is the most commonly considered environmental variable (Moaveni *et al.* 2009). Changes in natural frequency in the order of 10% from environmental or operational sources are not unusual (Cornwell *et al.* 1999; He 2008). Peeters & De Roeck 2000 demonstrate the temperature dependence of the Z24 Bridge in Switzerland, in terms of the first natural frequencies. The authors described a bilinear behavior of the first natural frequencies vs. temperature, strongly influenced by a freezing process of the asphalt on the road pavement during cold periods (Fig. 2.20).

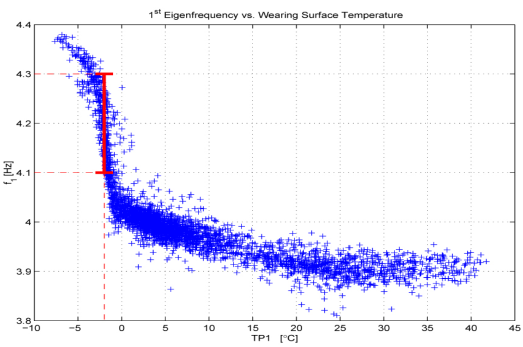


Fig. 2.20 - 1st eigenfrequency vs. wearing surface temperature of the Z24 bridge (Peeters & De Roeck 2000)

Changes to monitored parameters induced by environmental effects can be larger than changes due to significant damage (Peeters 2000). It is thus of major importance to filter out the environmental and loading effects in order to eliminate all the factors that can shadow an ongoing damaging process of the structure. This can be done through development of mathematical models correlating the modal parameters (e.g., natural frequencies) to environmental effects (e.g., temperature) as reported by Kullaa 2009, Vanlanduit *et al.* 2005, Yan *et al.* 2005.

One approach, instead of using a sophisticated model able to represents all the physical phenomena behind the parameters change, is to implement black box models whose parameters are tuned using a large number of observations, to establish relations between the extracted features (modal parameters or static data) and the factors that may influence them, using regression analysis. An interesting study was performed by Magalhães *et al.* 2011 in the contest of the monitoring program of the of the Infante D. Henrique Bridge: environmental and operational effects on the natural frequencies of the structure were removed through the implementation of regression models and Principal Component Analysis (PCA) (Fig. 2.21).

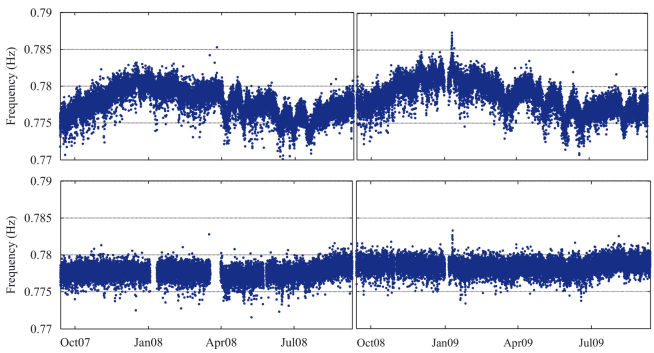


Fig. 2.21 - Time evolution of the first natural frequency before and after the elimination of the environmental and operational effects on the SHM system of the Infante D. Henrique Bridge (Magalhães et al. 2011).

Temperature effects on natural frequencies can be accurately modeled using simple linear regression models (Fig. 2.22) or polynomial and non-linear regression models. However these models, also known as static models, are not able to take into account the thermal inertia of the structure, which is an important property of the dynamic response of a structural system

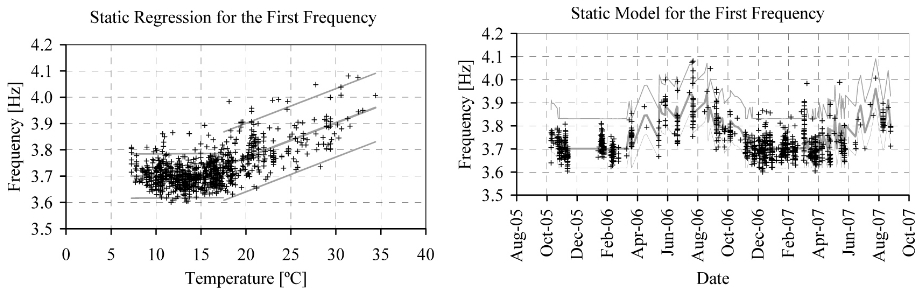


Fig. 2.22 - Linear regression models for the dynamic response of the church of the Monastery of Jerónimos in Lisbon (Ramos 2007)

An alternative is the use of autoregressive models or dynamic models such as AutoRegressive models with eXogenous inputs (ARX models) (Peeters & De Roeck 2000; Ramos 2007; He 2008). The ARX family of models (Ljung 1999) can account for dynamics in a relationship by relating present outputs to past inputs and outputs. This is ideal for representing thermal inertia phenomena.

In this model the response y_k(e.g. resonant frequencies) in the instant k does depend not only in the input u^{env} (e.g. temperature) for the same instant, but also on the evolution of the previous inputs and outputs (thermal inertia modelling). The estimated response can be given by:

$$\hat{y}_k + a_1 y_{k-1} + \cdots + a_{na} y_{k-na} = b_1 u_k^{env} + b_2 u_{k-nk-1}^{env} + \cdots + b_{nb} u_{k-nk-nb+1}^{env} + e_k \quad (2.1)$$

where a_{i} and b_{i} are coefficients for the autoregressive and exogenous part, respectively, na is the autoregressive order, nb the exogenous order, nk is the number of delays from input to output, and $e_k = y_k - \hat{y}_k$ is the unknown residuals that can be assumed Gaussian. y_k and $\hat{y}_k$ are the actual (real) and the estimated (model) responses respectively.

To simplify, Eq. (2.1) can be rewritten using a shift operator $q^{-1} y_k = y_{k-1}$:

$$A_q \hat{y}_k = B_q u_{k-nk}^{env} + e_k \quad (2.2)$$

with the operator polynomials A_q and B_q given by:

$$\begin{aligned} A_q &= 1 + a_1 q^{-1} + \cdots + a_{na} q^{-na} \\ B_q &= b_1 + b_2 q^{-1} + \cdots + b_{nb} q^{-nb+1} \end{aligned} \quad (2.3)$$

It is common to refer to a ARX model by the orders na and nb, the delay nk, being the model characterized by these three parameters (e.g. ARX[1,2,3]).

Peeters (2000) and Ramos (2007) presented a procedure to model the environmental effects and detect the presence of damage based on ARX models. A similar approach is adopted in the present research work.

2.4 Automated monitoring data processing

SHM can be defined as the continuous real time control of the behavior of a building based on the feedback of the structure during its lifecycle, in terms of response and performance under operating loads and environmental conditions. The central core of each monitoring system is the capability to automatically identify structural damages, which means determine that a certain type of damage has occurred, its location, extent and severity (Sikorsky 2000). The automation of SHM is therefore of fundamental importance in order to obtain a continuous and real time monitoring.

In general, the development of effective methods of monitoring depends on two key factors: the signal acquisition technologies and the signal processing interpretation algorithms. The first ones achieved a good level of accuracy in recent years, whereas several dynamic based damage identification methods have been proposed, related to many and different structural systems (Kim e*t al.* 2003).

Many applications have been implemented and documented in literature, as well as several methods aimed at the evaluation of the current performance of a civil engineering structure under operating conditions.

Some of these methods are based on the direct or indirect control of variations of the structural response, strictly related to the dynamic characteristics of the system (natural frequencies, mode shapes, etc.), especially as a result of damages caused by exceptional events. However, one of the main limitations of these techniques is that they need a certain level of user intervention during the extraction process of modal parameters of the structure. This aspect does not meet the normal requirements of a monitoring system, which should be, theoretically, completely autonomous. It is a problem of not simple solution: the dynamic identification techniques require, in fact, a strong interaction with an expert user. In addition, the computational effort is another factor that adversely affects the dissemination of dynamic identification methods for SHM and damage assessment purposes.

At the state of the art there is no clear distinction in literature between Modal Parameter Estimation (MEP), which is the extraction of modal parameters from a single record of measured data, and modal monitoring, i.e. the study of the evolution of the modal parameters of a structure through repeated MPE.

In recent years, in parallel, also the damage detection procedures based on static monitoring have become increasingly important especially through a combination of both approaches.

Self-managed monitoring systems are able to continuously evaluate the structural behavior. In addition, these approaches are able to actively support the operator in charge of the monitoring. On the basis of precise knowledge of the structural conditions, damage and deterioration of a structure can be accurately predicted. Furthermore, the lifecycle of a structure can be extended with timely and relatively cheap maintenance interventions. As a result, operating costs and repairs can be lowered and structural safety significantly increased. Stand-alone systems must meet two basic requirements that make the difference between a conventional system (automated) and an autonomous one:

- Provide a self-management system as an intrinsic property, which includes self-recovery, self-configuration, self-optimization and self-protection
- Check and carry out any monitoring process independently

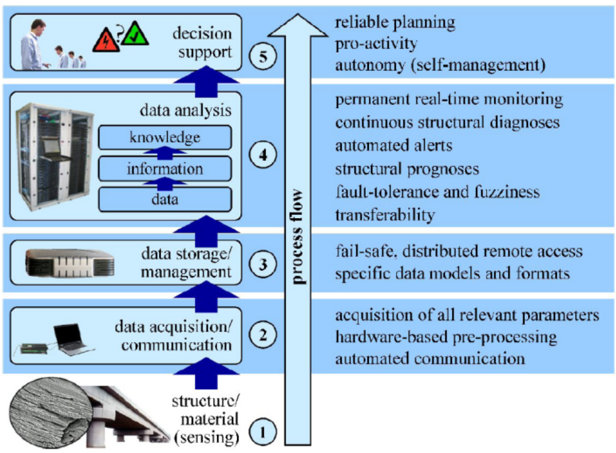


Fig. 2.23 - Knowledge-based process flow of SHM. Fundamental features that have to be provided by modern monitoring systems

These monitoring processes, which are cyclically executed by autonomous SHM systems, are shown in Fig. 2.23. Through a sensor network installed on the structure (1), structural and environmental data are detected by measuring devices (2) that preprocess and transmit them to the acquisition unit for the permanent storage (3) and for the successive in-depth analysis (4). The output of the autonomous process acts, therefore, as a basis for the decision stage (5). In fact, if the software is the heart of the monitoring system, the operator is always the brain as it is the only one with the necessary skills and knowledge to make system-outputs-based decisions: on-site inspections and controls, design of structural interventions and repair measures, emergency actions. The sum of all monitoring processes shown in Fig. 2.23 is the heart of the knowledge-based approach of SHM

2.4.1 Automated monitoring of static data

Static-based monitoring techniques allows identifying damaging processes by measuring changes in the static structural response (usually displacements or deformations under environmental thermal loads). This type of control requires the implementation of complex permanent monitoring systems, allowing in principle a continuous assessment of the actual safety conditions of structures.

The static structural response is compared with prediction statistical models (Robert-Nicoud *et al.* 2005). This is often referred to as system identification. However, the creation of such models may become costly in terms of computational effort and they may not accurately reflect the actual behavior. Difficulties and uncertainties increase in presence of complex civil structures, as in the case of monumental buildings, and a reference and unique behavioral model of the structural response cannot be clearly identified (Del Grosso & Lanata 2001).

When behavioral models are not used (data-driven approaches) long periods of observation are needed in order to produce reliable information (Masri *et al.* 2004).

For static monitoring, research on identification and localization of damage has been addressed only recently (Del Grosso *et al.* 2000; Omenzetter *et al.* 2004). Despite the increasing and constant research for the improvement of continuous static monitoring systems, reliable damage detection strategies have not been proposed nor tested yet for large civil engineering structures (Lanata 2005).

The number of monitored structures is growing. Static monitoring (Sohn *et al.* 2000) produces large amounts of data in different formats, from which it is necessary to extract information. Usually a statistical approach is required (Lanata & Del Grosso 2004). These methodologies examine and detect any changes over time of the time series and can be fully inserted in the family of data-driven approaches (Brownjohn et al. 2004; Omenzetter & Brownjohn 2006; Jaenisch *et al.* 2003; Posenat *et al.* 2008).

In 2006 Del Grosso *et al.* proposed a procedure able to identify anomalous behavior in acquired data without the use of reference behavioral models. This methodology can also be applied to general classes of civil structures with a low risk of generating false positives and false negatives. Another important aspect of this methodology is that it is applicable for the entire lifecycle of structures, having the capability to adapt itself to the new structural states.

Smarsly 2010 has recently introduced a hybrid monitoring approach (static and dynamic), called *Autonomous Monitoring System Based On Software Agents* (AMBOS).

In 2011, finally, Spencer & Agha 2011 of the University of Illinois at Urbana-Champaign released an open source software developed in Matlab (ISHMP, *Illinois Structural Health Monitoring Projec*t) for automatic hybrid monitoring systems (static and dynamic) with wireless sensors expressly designed for large structures, in particular for bridges.

2.4.2 Automated identification of modal parameters

The first approach to the problem of automatic identification of modal parameters is quite recent (Verboven *et al.* 2002). In fact with the rapid development and proven reliability of modal analysis techniques under operating conditions, there has been an increasing attention in the scientific community on these issues, with the formulation of several methods for the automatic identification and monitoring of modal parameters (Brincker *et al.* 2007; Deraemaeker *et al.* 2008; Rainieri *et al.* 2007). Being derived from the available techniques of output-only dynamic identification, these procedures can be classified into two categories: techniques based on (i) systems and controls theories (time or frequency domain) or on (ii) the classical signal analysis.

In the first case, when the classical methods of modal identification are applied to the experimental data, the order of the model is generally overestimated, so as to identify all the physical modes present in the frequency band of interest. However, physical and mathematical (or spurious) modes must then be separated. This problem requires a strong interaction with an expert user and the use of specific tools, such as stabilization diagrams, of proven effectiveness (Soderstrom 1975).

The selection of physical poles is a complex task though: difficulties and computational time needed to analyze data depend on several factors, such as performance of the estimator (although interesting progress has recently been registered), quality of data (always affected by a certain level of noise) and user experience (Lanslots *et al.* 2004). It is clear that extensive interaction between users and analysis tools constitutes a major obstacle to the integration of these methodologies in the context of continuous SHM.

The first proposed methodology for the automatic identification of modal parameters was based on the so called *Least Square Complex Frequency* (LSCF) technique (Verboven *et al.* 2003). The selection of the physical poles was based on a series of both deterministic and stochastic criteria, as well as on a fuzzy approach for

grouping into clusters. However, the algorithm for the selection of physical poles is quite complex and requires a heavy computational effort.
Using the time domain identification technique SSI (*Stochastic Subspace Identification*) (Van Overschee *et al.* 1996) in 2006 an algorithm in Matlab has been implemented for dynamic monitoring applied to reinforced concrete bridges and for the detection of induced damages (Mita *et al.* 2006).
Deraemaeker *et al.* 2008 proposed an automatic procedure for output-only modal analysis also based on SSI technique. It is effective for monitoring the evolution over time of modal parameters but it always requires an initial user interaction. In fact, an initial set of modal parameters must be firstly identified through SSI using a stabilization diagram.
A fully automated procedure for the identification of modal parameters always based on SSI technique was proposed by Andersen *et al.* 2007. It is based on the creation of clear stabilization diagrams according to the so called multipatch subspace approach. The extraction of the physical pole is then performed using the graph theory. The algorithm appears to be rather fast and, therefore, suitable for use in the context of monitoring systems. However, further improvements are needed with respect to the numerical efficiency of the method.
In the case of identification techniques based on the analysis of the signals, the so called *Time Domain Filtering* method (Guan *et al.* 2005) was introduced in the continuous monitoring and automatic extraction of modal parameters. It is based on the application of a bandpass filter to the response of the system in order to separate the modes in the spectrum. However, the limits of the procedure is that the filter is static and, above all, specified by the user on the basis of the Power Spectral Density (PSD) of the response signals. Therefore, since the excitation is unknown, it can be quite difficult to identify frequency bands of modes only by means of the PSD; moreover, in case of closely-spaced modes, it is rather difficult, if not impossible, to correctly define those limits and thus be able to follow the natural frequencies variations due to, for example, environmental factors.
Brinker *et al.* 2007 presented an algorithm for automating the *Frequency Domain Decomposition* (FDD) technique (Brinker *et al.* 2000) in order to remove any kind of user interaction and give the possibility to use the algorithm for dynamic monitoring. The algorithm is based on the identification of the so-called modal domain around each identified peak in the singular values decomposition of the PSD matrix, by means of the definition of limits for modal domain and modal coherence functions. It is suggested the adoption of a limit value of 0.8 for both functions. However, while the limit for the modal coherence indicator is justified on the basis of the standard deviation of the correlation between random vectors and on the number of

measurement channels, few indications are reported about the limit of the modal domain indicator.

Magalhães *et al.* 2008 slightly modified the above-mentioned approach for automatic identification of modal parameters and applied the updated version to the continuous monitoring of the *Infante D. Henrique* bridge. In this case also an automatic procedure for dynamic identification based on the *Covariance-Driven Stochastic Subspace Identification* (cSSI) method was proposed together with an hierarchical clustering algorithm to group into clusters stable poles.

Regarding the automatic FDD algorithm it is necessary to note that, the more the noise level in the spectra increases, the more the procedure loses efficiency. Moreover, after the definition of the frequency resolution and the frequency band, it is required to set a rejection threshold based on the Modal Assurance Criterion MAC (Allemang & Brown 1982). This value has to be identified for each monitored structure by means of a number of sensitivity analyses and, therefore, through a time-consuming calibration process. It has been proposed to use a very low value (0.4) of rejection level, with the result that if the number of sensors is limited, the structure has closely-spaced modes or the mode shapes are similar, the procedure presents some performance problems.

The automation of the cSSI seems to be more efficient in case of closely spaced modes but it shows a decreased capability to identify poorly excited modes. The application of the clustering algorithm allows a reliable identification of the structural modes: however, also in this case it is necessary to define a number of parameters by an initial calibration of the system.

Automatic algorithms for identification of modal parameters have been recently proposed also for the so-called *Second Order Blind Identification* (Poncelet *et al.* 2008) and for methods based on the calculation of transmissibility functions (Devriendt *et al.* 2008).

For *Second Order Blind Identification* methods, all the frequencies out of the frequency range of interest are rejected as well as all sources whose time series are characterized by an interpolation error higher than 10% of the theoretical autocorrelation function for single degrees of freedom systems. Afterwards, the actual selection of the structural modes is based on the calculation of a confidence factor. The main advantage of this procedure is represented by reduced computational costs compared to SSI methods. Furthermore, it is not necessary to define in advance the system order of the model. However, the major limitation is linked to the need of having a number of sensors not less than the number of active modes. So far, the algorithm has been applied only to simulated data. Its effectiveness in the case of real measurements has not been validated yet, although the first applications provide very encouraging results.

Output-only dynamic identification procedure based on transmissibility functions take advantage from the combined use of singular value decomposition and stabilization diagrams for the selection of structural modes. The calculation of the stabilization diagrams from transmissibility functions provides vertical alignments of stable poles which, nevertheless, does not necessarily correspond to structural poles of the system. It is therefore necessary to implement a further selection tool, which is the calculation of the singular value decomposition of a two-columns matrix, representative of the transmissibility functions evaluated for distinct load conditions. Since all transmissibility functions converge to the same value in correspondence to the stable poles of the system, this matrix assumes for each pole a rank equal to one. Therefore, it is sufficient to look at the graph of the second inverse singular value to distinguish the actual structural modes from its peak. Peaks selection is, also in this case, dependent from the definition of a threshold. In case of measurement noise, in addition, this approach does not seem to be very reliable. In order to overcome this limitation, it has been proposed the use of a smoothing function. Further improvements are, therefore, necessary to ensure the effectiveness and reliability of the algorithm in the application on real cases.

Rainieri 2008 recently proposed an automatic algorithm for the identification of modal parameters using the FDD technique. It is based on the repeated calculation of the singular value decomposition of the PSD matrix for a number of records and on the construction of a MAC diagram in function of the frequency, which looks like the coherence function. Examining the sequence of MAC values at each frequency, it is possible to identify the effective bandwidth of each mode to be used for the extraction of the modal parameters.

Rainieri & Fabbrocino 2011 proposed a new strategy for automatic operational modal analysis based on the combination of different output-only modal analysis methods. The basic idea is the possibility to simplify the analysis and the interpretation of stabilization diagrams typical of parametric methods and, therefore, the separation between physical and mathematical (spurious) modes through a preliminary phase of *Blind Source Separation* (BSS) (Ans *et al.* 1985; Poncelet *et al.* 2007) performed according to the so called *Second Order Blind Identification* (SOBI) algorithm (Belouchrani *et al.* 1997). The main goal of the developed strategy is an accurate and robust identification of modal parameters and, in particular, of the damping ratios of the monitored structure.

In the same year, Reynders & De Roeck 2011 have presented a fully automatic procedure for the estimation of modal parameters through the application of clustering analysis in three phases for the interpretation of the stabilization diagrams; through the application of this algorithm to 153 sets of ambient vibration data, obtained during a progressive damage test on a three-span concrete bridge, the stability and robustness of the procedure was proven.

Ubertini *et al.* 2011 presented an alternative approach for the automatic selection of the estimates of the modal parameters obtained with the SSI method. The proposed methodology is an improved version of existing approaches and it is based on noise reduction procedures and clustering analyses.

In order to optimize the monitoring of dynamic properties of the new stadium in Braga - Portugal, Amador *et al.* 2011 developed a software (Amador 2009) to process raw data time series, providing important results on the influence of environmental factors on modal parameters.

2.5 Problems and limitations in the development of SHM

In the following section main aspects that prevent a wider diffusion of SHM together with the principal problems and limitations that are still a challenge to be addressed by scientific community will be presented and discussed, following a similar review proposed by Brownjohn 2007:

(A) Reliability of the system

Since SHM systems are designed to assist infrastructure operators / owners in the management stage of their facilities, a comprehensive cost-benefit analysis is to be performed before the application of monitoring systems. The initial costs can be rather high, but the long-term benefits of SHM of strategic structure and infrastructures and also of important CH monuments are extraordinary and largely justify the initial economic investment. Nevertheless it is necessary to keep under control operation and maintenance cost of SHM systems.

(B) Low cost instrumentation and sensor overload

It is of fundamental importance to select the optimum number, typology and location of sensors on the monitored structure. Sometimes there is the tendency to apply more sensors than necessary, implementing low cost and thus not reliable instrumentation. This aspect affects the quality of the equipment and the final effectiveness of the monitoring system. The structural system need to be carefully evaluated in order to assess its structural performance and identify vulnerabilities and possible active damaging processes. In this way it is possible to reduce the number of sensors and extract only the fundamental features from monitoring measures. It is necessary that the design, implementation and management phases of monitoring are supervised by experts and specialist consultants.

(C) Data storage

This stage is strictly linked to the previously described problem. The storage capacity of monitoring systems are not unlimited. Overloads of data caused by wrong and excessive implementation of sensors lead to the acquisition of large amount of data that can be hardly stored and processed.

(D) Data transmission

Data transmission from the central unit of the monitoring system to the server of the institution that has in charge the management activities is a rather difficult task.

Wireless connections, dial-up modems or permanent routers are able to establish rather stable connection for data exchange and transmission. However due to the limited capacity of some internet connections, it is necessary to compress or pre-process data before the transmission.

(E) Environmental factors and noise

This problems was highlighted in § 2.3.3.2 for full-scale structures. Environmental and loading conditions have to be accurately removed before any attempt of applying damage detection algorithms. Partial mitigation of these effects can be achieved through physics-based or statistical models, even if a certain level of noise will always be present.

(F) Presentation of monitoring results

One of the most significant issues with SHM is converting data to information. It is necessary to present the obtained information in a simple and prompt manner, using charts and diagrams easily readable and understandable by operators and customers that, in general, are not familiar with sophisticated underlying numerical procedures.

(G) Development of standards and codes for SHM

Although the use of SHM in civil engineering structures is growing more and more, together with a great research interest on these topics among the scientific community, clear standard and codes for a correct application of SHM are still missing. In this contest it is necessary to issue recommendations for the design of the measurement equipment and data acquisition systems to be used in both laboratory applications and full-scale structures. Moreover there is the need to define and elaborate a rigorous normative approach and stress the importance of monitoring also from a legislative point of view, to be propose finally to designers, builders, developers and managers in a context of a mature and aware regulatory framework.

2.6 Conclusions

In this chapter a state of the art about SHM was presented. Addressed issues include a review on the application of monitoring techniques to civil engineering structures during the last century, description of the monitoring process with particular attention on the automated feature extraction phase. To conclude problems and limitations of SHM are reported.

SHM was initially applied (starting from the Sixties/Seventies) in the field of mechanical and aerospace engineering to study and evaluate the health state of complex but rather small structural systems. In the last decades then its application moved to other disciplines, including large civil engineering structures and infrastructures, new and historic buildings, bridges, tunnels, factories, power plants, offshore oil platforms, port facilities and geotechnical structures. Since the present research treats the development and application of monitoring techniques to cultural heritage buildings and monuments a greater attention is paid on the state-of-the-art of monitoring in this specific sector of civil engineering.

The monitoring process is a quite complicated and complex task. The final aim is to better understand and increase the knowledge on the actual behavior of a structural system and to identify possible ongoing damaging processes. It is possible to distinguish three main phases in the application of SHM: (i) Instrumentation and data acquisition; (ii) Signal processing and feature extraction; (iii) Damage detection, alarms and reports.

In the first phase it is necessary to select both the monitoring strategy and the optimal sensors to be applied on the investigated structure. A detailed review on this fundamental stage of SHM is reported, stressing the attention on the report of traditional and innovative sensors available at the state of the art to monitor civil engineering structures.

The second phase need to be automated as much as possible in order to manage huge amount of data and extract only fundamental features, avoiding loss of information. Since in the present research an important section is dedicated to the development of automated feature extraction algorithms both for static and dynamic systems, a comprehensive review on these topics is reported and dedicated software for the management of SHM systems are described.

The last phase of the monitoring process is the damage identification stage which can be performed applying both Data-Driven and Model-Driven approaches. In real case studies to detect the presence of damaging processes it is necessary to evaluate and filter out the so called environmental and loading effects from both the static and dynamic response of structures. A review on the statistical approach based on black box models is reported: those techniques are able to “model” environmental factors and, afterwards, identify damages comparing the predictions of such models with the real measured data (residual analysis).

3 BASIC DYNAMICS AND OPERATIONAL MODAL ANALYSIS

3.1 Introduction

In this chapter some basic concepts of structural dynamics and a state-of-the-art review of the most common Operational Modal Analysis (OMA) techniques is reported. In particular the work is focused on the description of three frequency-domain methods with different formulations, which proved to be very effective in the estimation of modal parameters of civil structures and are extensively used within the thesis: two non-parametric method, i.e. Frequency Domain Decomposition (FDD) and its enhanced version EFDD and a parametric technique, i.e. poly-reference Least Square Complex Frequency domain (p-LSCF).
The aim is to integrate the literature review proposed in Chapter 1 on SHM with more specific notions on the application of dynamic identification methods, exploited subsequently both for the execution of Ambient Vibration Tests (AVT) on selected case studies (Chapter 5) and in the development of automated algorithms for OMA (Chapter 7).

3.2 Basic dynamics

3.2.1 Classical formulation

The dynamic behavior of a structure can be represented either by a set of differential equations in the time domain, or by a set of algebraic equations in the frequency domain. Equations of motion are traditionally expressed in time domain, thus obtaining, for a general Multi-Degree-Of-Freedom (MDOF) system, the following set of linear, second order differential equations, expressed in matrix form:

$$\boldsymbol{M}\ddot{\boldsymbol{q}}(t) + \boldsymbol{C}\dot{\boldsymbol{q}}(t) + \boldsymbol{K}\boldsymbol{q}(t) = \boldsymbol{p}(t) \tag{3.1}$$

Where $\ddot{\boldsymbol{q}}(\boldsymbol{t}), \dot{\boldsymbol{q}}(\boldsymbol{t}), \boldsymbol{q}(\boldsymbol{t})$ are the acceleration, velocity and displacement vector respectively, while $\boldsymbol{M}$, $\mathbf{C}$, $\boldsymbol{K}$ denote the mass, damping and stiffness matrices; $\boldsymbol{p}(t)$ is the generalized excitation vector. This matrix equation is written for a linear, time

invariant ($\boldsymbol{M}$, $\mathbf{C}$, $\boldsymbol{K}$ are constant), observable system with viscous or proportional damping.
To solve Eq. (3.1) Fourier transform functions can be used, establishing the direct relation between excitation and response, through the FRF:

$$\boldsymbol{H}(\omega) = [-\omega^2\boldsymbol{M} + j\omega\boldsymbol{C} + \boldsymbol{K}]^{-1} \tag{3.2}$$

the calculation of the FRF is tedious because it is necessary to calculate the complex inverse of the matrix of $n \times n$ dimension for each frequency ω. An alternative solution is the modal approach, which starts from the assumptions of a undamped problem by the homogeneous differential equation:

$$\boldsymbol{M}\ddot{\boldsymbol{q}}(t) + \boldsymbol{K}\boldsymbol{q}(t) = 0 \tag{3.3}$$

A solution of this differential equation is given by:

$$\boldsymbol{q}(t) = \varphi_i e^{\lambda_i t} \tag{3.4}$$

where φ_i are the real eigenvectors (i = 1,..., n) and λ_i are the real eigenvalues, which in case of undamped systems are equal to the natural frequencies ω_i ($\lambda_i = j\omega_i$). Introducing Eq.(3.3) into Eq.(3.4) results in:

$$[\boldsymbol{K} - (-\lambda_i^2)\boldsymbol{M}]\varphi_i = 0 \quad \vee \quad \boldsymbol{K}\Phi = \boldsymbol{M}\Phi\Lambda \tag{3.5}$$

The modes are commonly grouped in the modal matrix Φ where each column represents the eigenvectors, and the eigenfrequencies ω_i are grouped in a diagonal matrix Λ. The orthogonality properties of the modes shape hold:

$$\boldsymbol{\Phi}^T\boldsymbol{M}\boldsymbol{\Phi} = [m_i] \quad , \quad \boldsymbol{\Phi}^T\boldsymbol{K}\boldsymbol{\Phi} = [k_i] \tag{3.6}$$

where m_i are the modal masses, k_i the modal stiffness and the superscript T denotes transpose. The eigenvectors can also be mass-normalized by the matrix Φ_m, composed by the normalized eigenvalues ($\varphi_{m,i} = \varphi i/\sqrt{mi}$), leading to the following relations:

$$\boldsymbol{\Phi}_m^T\boldsymbol{M}\boldsymbol{\Phi}_m = \boldsymbol{I} \quad , \quad \boldsymbol{\Phi}_m^T\boldsymbol{K}\boldsymbol{\Phi}_m = \boldsymbol{\Lambda}^2 \tag{3.7}$$

Where $\boldsymbol{I}$ is the identity matrix of dimension n x n.
If Eq. (3.5) is pre-multiplied by Φ^{T} and taking into account the Eq. (3.6), the natural undamped frequencies of each mode i can be obtained similarly to one degree of freedom system problem:

$$\omega_i^2 = k_{e,i}/m_i \tag{3.8}$$

Adding now proportional damping to the system, i.e. damping which gives a linear relation between the stiffness and mass, and assuming that the damping matrix can be also diagonalized, it is possible to obtain:

$$\boldsymbol{\Phi}^T \boldsymbol{C} \boldsymbol{\Phi} = [c_i] = [2\xi_i \omega_i m_i] = \Gamma[m_i] \quad \text{with} \quad \Gamma = [m_i] \tag{3.9}$$

Introducing the coordinate transformation $\boldsymbol{q}$(t) = Φ $\boldsymbol{q}_m(t)$, where $\boldsymbol{q}_m(t)$ are the modal displacements, and pre-multipling Eq. (3.1 by $\boldsymbol{\Phi}^{\mathrm{T}}$, the following simplified equation, with all the left diagonal side terms, is obtained:

$$\boldsymbol{I}\ddot{\boldsymbol{q}}_{\boldsymbol{m}}(t) + \Gamma\dot{\boldsymbol{q}}_{\boldsymbol{m}}(t) + \Lambda^2 \boldsymbol{q}_{\mathrm{m}}(t) = \left[\frac{1}{\mathrm{m_i}}\right] \boldsymbol{\Phi}^{\mathrm{T}} \mathrm{p(t)} \tag{3.10}$$

Again, if this equation is assumed to be homogeneous, the solution form is equal to the one adopted in Eq. (3.4 and the eigenvalues satisfy the condition:

$$\lambda_i^2 + 2\xi_i \omega_i \lambda_i + \omega_i^2 = 0 \tag{3.11}$$

With the solution:

$$\lambda_i = -\xi_i \omega_i + j\omega_i \sqrt{1 - \xi_i^2} \quad , \quad \lambda_i^* = -\xi_i \omega_i - j\omega_i \sqrt{1 - \xi_i^2} \tag{3.12}$$

$$\omega_i = |\lambda_\mathrm{i}| \quad , \quad \xi_i = -\mathrm{Re}(\lambda_\mathrm{i})/|\lambda_\mathrm{i}| \tag{3.13}$$

Where the superscript * denote the complex conjugate.

3.2.2 Transfer function and modal model

The matrix of differential equations (3.1) becomes a set of linear algebraic equations using the Fourier transform or of the Laplace transform, and of their properties (the interested reader can refer to classical signal processing books, or to Heylen *et al.* 2002); in particular, through the application of the Fourier transform to the equation of motion, one obtains:

$$\boldsymbol{M}s^2\boldsymbol{Q}(s) + \boldsymbol{C}s\boldsymbol{Q}(s) + \boldsymbol{K}\boldsymbol{Q}(s) = \boldsymbol{P}(s) \tag{3.14}$$

or

$$[\boldsymbol{M}s^2 + \boldsymbol{C}s + \boldsymbol{K}]\boldsymbol{Q}(s) = \boldsymbol{P}(s) \quad \Leftrightarrow \quad \boldsymbol{Z}(s)\boldsymbol{Q}(s) = \boldsymbol{P}(s) \tag{3.15}$$

Where $\boldsymbol{Z}(s)$ is the dynamic stiffness matrix.
Therefore, it is possible to relate the Laplace transform of the outputs $\boldsymbol{Q}(s)$ with the Laplace transform of the inputs $\boldsymbol{P}(s)$ through a matrix called transfer function:

$$\boldsymbol{Q}(s) = \boldsymbol{H}(s)\boldsymbol{P}(s) \text{ with } \boldsymbol{H}(s) = \boldsymbol{Z}(s)^{-1} = [\boldsymbol{M}s^2 + \boldsymbol{C}s + \boldsymbol{K}]^{-1} \quad (3.16)$$

where $\boldsymbol{H}(s)$ is generally called transfer function matrix since it relates the outputs or the response $\boldsymbol{Q}(s)$ and the input or excitation $\boldsymbol{P}(s)$. Each element of $\boldsymbol{H}(s)$ is a complex valued function and the values s at which the element of $\boldsymbol{H}(s)$ present infinite values are designated as *poles.* The poles, represented by λ_k, are complex numbers whose real and imaginary components are related with the natural frequencies and modal damping ratios of the dynamic system by equation:

$$\lambda_k = -\xi_k\omega_k + i\sqrt{1 - \xi_k^2\omega_k} \quad (3.17)$$

Substituting s by ωi it is possible to obtain the so called frequency response function FRF $\boldsymbol{H}(\omega)$, whose diagonal terms is a $n\times n$ matrix given by (Heylen *et al.* 2007):

$$\boldsymbol{H}(\omega) = \sum_{i=1}^{n_2}\left[\frac{\phi_i\gamma_i^T}{i\omega - \lambda_i} + \frac{\phi_i^*\gamma_i^H}{i\omega - \lambda_i^*}\right]$$

$$\text{with} \quad \begin{matrix} V = [\phi_1 \dots \phi_{n2} \;\; \phi_1^* \dots \phi_{n2}^*] \\ L = [\gamma_1 \dots \gamma_{n2} \;\; \gamma_1^* \dots \gamma_{n2}^*] \end{matrix} \qquad \Lambda = \begin{bmatrix} \lambda_1 & & & & & \\ & \ddots & & 0 & & \\ & & \lambda_{n2} & & & \\ & & & \lambda_1^* & & \\ & 0 & & & \ddots & \\ & & & & & \lambda_{n2}^* \end{bmatrix} \quad (3.18)$$

where n_2 is the number of modes, •* is the complex conjugate of a matrix or vector, •^{H} is the complex conjugate transpose of a matrix or vector, ϕ_i is a column vector containing the n_2 components of mode shape i, γ_i^T is a line vector with the n_2 components of the modal participation factor of mode i and λ_i, for $i = 1, \dots, n_2$ are the structure poles.

3.2.3 State space model

The matrix of differential equations (3.1) can be transformed into a first-order one, the state equation, using simple mathematical manipulations. A state-space model is obtained by combining the state equation with the observation equation:

$$\begin{aligned} \dot{x}(t) &= \boldsymbol{A}_C x(t) + \boldsymbol{B}_C u(t) \\ y(t) &= \boldsymbol{C}_C x(t) + \boldsymbol{D}_C u(t) \end{aligned} \quad (3.19)$$

where A_C, designated state matrix, is a n-by-n matrix, with n equal to $2n_2$ and $x(t)$ is the state vector, which contains the displacements and the velocities vectors of the dynamic system. The state-space formulation constructs mathematical models in which experimental data can be coupled to discrete models. This formulation represents a powerful tool, especially in case of stochastic time series and the presence of noise in the experimental data.

For further readings about State-Space models the reader can refer to Peeters 2000, Rodrigues 2004, Juang 1994, Van Overschee & De Moor 1996.

3.2.4 Right matrix fraction model

Input-Output relationships can be also expressed as the rational fraction of two polynomials, being the denominator polynomial common to all input-output relations. In this case the so called common denominator model or scalar matrix fraction model is considered.

Following this approach the transfer function matrix (3.16) $\boldsymbol{H}(s)$ can be expressed as follow:

$$\boldsymbol{H}(s) = \frac{\boldsymbol{N}(s)}{d(s)} \tag{3.20}$$

where:

$$\boldsymbol{N}(s) = adj[\boldsymbol{M}s^2 + \boldsymbol{C}s + \boldsymbol{K}] \tag{3.21}$$

$$d(s) = det[\boldsymbol{M}s^2 + \boldsymbol{C}s + \boldsymbol{K}] \tag{3.22}$$

Where adj indicates the adjont matrix, so that $\boldsymbol{N}(s)$ is the adjont matrix of $\boldsymbol{Z}(s)$ containing polynomials of grade $2N_m$-1. Using the common denominator model, each relation between one input and one output can be separately considered. And this results in a great advantage for identification procedures.

Matrix fraction models (fully described by Cauberghe 2004) are more abstract models that can be more easily fitted to measured data and then used to obtain estimates of modal parameters. In particular, the Right Matrix Fraction Model defines the Transfer Function as the right division (B/A) of two polynomial matrices:

$$\boldsymbol{H}(s) = \boldsymbol{B}\boldsymbol{A}^{-1} \quad \Leftrightarrow \quad \boldsymbol{H}(s) = \left[\sum_{r=0}^{p_b} \boldsymbol{B}_r s^r\right]\left[\sum_{r=0}^{p_a} \boldsymbol{A}_r s^r\right]^{-1} \tag{3.23}$$

where the matrices $\boldsymbol{B}_r$ are n_0-by n_i and the matrices $\boldsymbol{A}_r$ are n_i-by n_i. p_a and p_b are the orders of the denominator and numerator polynomials. Once the model matrices have been estimated, then natural frequencies and modal damping ratios can be extracted from the coefficients of the denominator polynomial and the mode shapes determined from the coefficients of $\boldsymbol{B}_r$. Instead, it is also possible to convert the estimated right matrix-fraction model into a state-space model (Eq. (3.19)) .The equations needed to make this transposition have been derived in Reynders 2009, assuming $p_a = p_b = p$ and that the degree of the polynomial with the determinant of $A(s)$ is equal to $p \cdot n_i$:

$$\boldsymbol{A}_C = \begin{bmatrix} -\boldsymbol{A}_p^{-1}\boldsymbol{A}_{p-1} & -\boldsymbol{A}_p^{-1}\boldsymbol{A}_{p-2} & \cdots & -\boldsymbol{A}_p^{-1}\boldsymbol{A}_1 & -\boldsymbol{A}_p^{-1}\boldsymbol{A}_0 \\ \boldsymbol{I} & 0 & \dots & 0 & 0 \\ \vdots & \ddots & \dots & \vdots & \vdots \\ 0 & 0 & \dots & \boldsymbol{I} & 0 \end{bmatrix} \qquad (3.24)$$

$$\boldsymbol{B}_C = \begin{bmatrix} \boldsymbol{A}_p^{-1} \\ 0 \end{bmatrix}$$

3.2.5 Output spectrum and half-spectrum

Frequency domain OMA techniques require output spectra as primary data and under the basic assumption of white noise input, such spectra can be modeled similarly to the transfer function matrix $\boldsymbol{H}(s)$ or the FRF matrix (Peeters & Van der Auweraer 2005). In fact as described above, the FRF matrix is composed by the cross section of the elements of $\boldsymbol{H}(s)$ along the imaginary axis $s = i\omega$. The output spectrum matrix S_{yy} restricted to $s = i\omega$ is related to the input spectrum matrix S_{uu} by the following equation (Ljung 1999):

$$\boldsymbol{S}_{yy}(\omega) = \boldsymbol{H}(\omega)\boldsymbol{S}_{uu}(\omega)\boldsymbol{H}^H(\omega) \qquad (3.25)$$

When the input is assumed to be a white noise (OMA), the output spectrum depends only on the system transfer function $\boldsymbol{H}(\omega)$ and on a constant matrix, which is the white noise input correlation matrix:

$$\boldsymbol{S}_{yy}(\omega) = \boldsymbol{H}(\omega)\boldsymbol{R}_{uu}\boldsymbol{H}^H(\omega) \qquad (3.26)$$

Considering Eq.(3.18) in Eq.(3.25) and converting it to the partial fraction form, the modal decomposition of the output spectrum is:

$$\boldsymbol{S}_{yy}(\omega) = \sum_{i=1}^{n_2} \frac{\phi_i g_i^T}{i\omega - \lambda_i} + \frac{\phi_k^* g_i^H}{i\omega - \lambda_i^*} + \frac{g_i \phi_i^T}{-i\omega - \lambda_k} + \frac{g_i^* \phi_i^H}{-i\omega - \lambda_i^*} \qquad (3.27)$$

Where g_i are the operational reference factors for the i^{th} mode, which replace the modal participation factors when only output data are available.

The modal decomposition of the output spectrum shows that this has four poles ($\pm\lambda_i, \pm\lambda_i^*$) for each mode. Thus the power spectrum model is twice the model order of the FRF model, while in modal analysis, models of lower order can be fitted without affecting the quality (Peeters & Van der Auweraer 2005).
The so called positive or half-spectrum S_{yy}^+ allows not to lose this advantage:

$$\boldsymbol{S}_{yy}(\omega) = \boldsymbol{S}_{yy}^+(\omega) + \boldsymbol{S}_{yy}^+(\omega)^H \tag{3.28}$$

The modal decomposition of the half-spectrum is given by:

$$\boldsymbol{S}_{yy}^+(\omega) = \sum_{i=1}^{n_2} \frac{\phi_i g_i^T}{i\omega - \lambda_i} + \frac{\phi_k^* g_i^H}{i\omega - \lambda_i^*} \tag{3.29}$$

As a consequence, the same lower order models used to model the FRF matrix can be also adopted to model the half-spectrum matrix. Half-spectrum matrix will be later considered in the explanation of the so-called p-LSCF identification technique.

3.3 Operational Modal Analysis

Experimental identification of modal parameters proved to be a very effective tool among other Non-Destructive Tests (NDT) for the achievement of an advanced knowledge of the structural behavior of civil engineering structures (Modena *et al.* 2001; Jaishi *et al.* 2003; Gentile *et al.* 2004; Ramos *et al.* 2006). It is a research topic with more than six decades of history which started to be applied in Mechanical and Aerospace Engineering to determine in laboratory the dynamic behavior of mechanical systems or part of them. The interest in such technique lies in the fact that while other methodologies provide qualitative/quantitative information on local properties of the constitutive materials or structural elements, dynamic testing permit to measure experimentally parameters related to the global structural behavior. Dynamic (modal) testing corresponds to the process of testing structures with the objective of obtaining a mathematical description of their dynamic behavior (Ewins, 1984). The determination of mathematical models of dynamic systems from vibration measurements is a problem, commonly called system identification, of considerable importance in the area of applied mechanics (Peng, 1987).
Contextualizing this issue to the present work, such importance derives from the fact that it can be considered hardly possible to develop fully realistic, reliable theoretical and computational models for "real" historical masonry structures. In

situations where a more accurate interpretation/prediction of structural behavior is required, it is often necessary to develop an experimentally verified model, based on response data measured during dynamic tests or natural excitations.

Dynamic identification tests are thus intended for the characterization of the modal response of a structure, being the obtained results referable to several structural/physical parameters, such as mass distribution and geometry (including e.g. connections effectiveness), stiffness and boundary conditions. The models derived from system identification may be used to assess the actual behavior of the structural system, helping in the prediction of the system response to future excitations, such as earthquakes (Casarin 2006).

It is possible to state that the identification of modal parameters can be exploited for different but connected purposes: (i) Calibration, validation and updating of reference numerical model of the structure under investigation; (ii) Application of damage detection techniques based on vibration signatures; (iii) SHM; (iv) Development of early warning system for special structures such as bridges, high-rise buildings, etc.

The first approach to the experimental identification of modal parameters involved the execution of Forced Vibration Tests (FVT) in which actuators or special devices (e.g. vibrodyne, Electric mass shaker, Impulse/Impact hammers) produce controlled inputs and response transducers (e.g. accelerometers) measure the outputs. From the relation between the applied inputs and the observed outputs the modal parameters (natural frequencies, mode shapes and damping ratio) are then calculated by estimating the Frequency Response Function (RFR) or the Impulse Response Function (IRF), in frequency or time domain respectively. This approach is also known as Experimental Modal Analysis (EMA).

Given the technical problems and limitations in the application of controlled and measurable dynamic excitations to large civil engineering structures (e.g. dams, bridges, buildings), but also to mechanical systems under operational conditions (e.g. cars during road testing, aircrafts during flight tests), significant research interests have been recently increased to the execution of Ambient Vibration Tests (AVT). These techniques, also known as Operational Modal Analysis (OMA) or output-only modal analysis, are based on the dynamic response measurements of a virtual system under natural (ambient or operational) conditions and the assumption that the unknown inputs are a realization of a stochastic process (white noise).

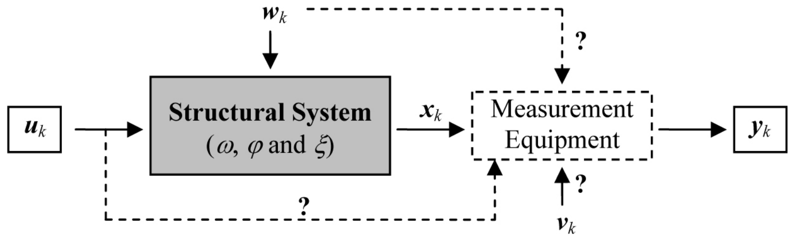


Fig. 3.1 - Scheme of Output-only identification techniques (Ramos 2007)

Looking at Fig. 3.1 (Ramos 2007), the ambient excitation $\boldsymbol{u}_k$ is assumed to be a zero mean stationary Gaussian white noise stochastic process. The structural system excited by $\boldsymbol{u}_k$ should be linear and time-invariant. The measured response $\boldsymbol{y}_k$ includes the modal contribution of the ambient forces, the contribution of the structural system $\boldsymbol{w}_k$ and certain level of noise induced by external sources $\boldsymbol{v}_k$. Although such ideal assumptions are rarely fulfilled in reality, they represent the basis for the theoretical development of Output-Only identification techniques.

In the following paragraph a brief review of the most diffused and implemented OMA techniques is reported, based on the contribution of Magalhães & Cunha 2010.

3.3.1 Review of OMA techniques

A first classification of modal parameters identification techniques based on the measurement of the response of dynamic systems to ambient vibrations can be performed between frequency domain and time domain methods.

3.3.1.1 Frequency domain methods

Frequency domain methods start from the analysis of the output spectrum or half-spectrum matrices (as defined in §3.2.5) and can be further classified into non-parametric or parametric methods. Non-parametric frequency domain methods are faster in the processing time and much more user friendly. The Peak-Picking method (Ewins 2000; Maia & Silva 1997) is the simplest and most well-known. The name of the method is related to the fact that natural frequencies are determined as the peaks of the Power Spectral Density plots, obtained by converting the measured data to the frequency domain by the Discrete Fourier Transform (DFT). The Frequency Domain Decomposition (FDD) (Brincker *et al.* 2000) and the Enhanced Frequency Domain Decomposition (EFDD) (Brincker *et al.* 2001) methods are more sophisticated non-parametric methods and will be briefly presented in the next section. Parametric frequency domain methods are based on the fitting of a model to the output spectrum or half-spectrum matrix, from which modal parameters are extracted in a second phase, using for example the modal model (see §3.2.2), the common-denominator model, the right and left matrix-fraction model (see §3.2.4). Cauberghe 2004 provides a comprehensive classification of all possible combinations of the above-mentioned models and frequency-domain system identification procedures. Furthermore for a detailed description of other frequency-domain parametric models the reader can refer to Guillaume *et al.* 1999 and Devriendt & Guillaume 2007. In the following section a famous parametric frequency domain method will be described in detailed, i.e. the poly-Least Square Complex Frequency domain method, also known by its commercial name PolyMAx, after its implementation in the commercial software Test.Lab from LMS.

3.3.1.2 Time domain methods

Time domain methods are based on model fitting of every measured point by means of correlation functions or time history series. Time domain methods applied to OMA derived directly by already existing techniques for the analysis of impulse response functions recorded by forced vibration tests. In this way the so called Eigensystem Realization Algorithm (ERA) method largely applied in the EMA was converted into an OMA method, the Natural Excitation Technique - NExT, by James *et al.* 1992.

Today the most diffused and applied time domain methods in OMA are based on two types of models: discrete-time stochastic state-space models and autoregressive moving average (ARMA) or auto-regressive (AR) models.

The first group of models use the formulation of state-space models (see §3.2.3) and deal directly with time series processing. Models can be identified both from correlations (or covariances) of the output (Covariance driven Stochastic State Space identification - SSI-cov) or directly from the recorded time series by the use of projections (Data driven Stochastic State Space identification - SSI-data) (Van Overscheee & De Moor 1996). State-space models approach is largely adopted for civil engineering applications.

Methods based on ARMA or AR models are not so frequently implemented to identify the dynamic response of large structures. For a detailed reading on ARMA models see Andersen 1997, Ljung (1999) and Peeters (2000).

Tab. 3.1 summarizes some Output-Only dynamic identification techniques.

Tab. 3.1 - Classification of some OMA identification algorithms

Frequency Domain	*Peak Picking (PP)*
	Frequency Domain Decomposition (FDD)
	Enhanced Frequency Domain Decomposition (EFDD)
	poly-Least Square Complex Frequency domain (pLSCF)
Time Domain	*Random Decrement (RD)*
	Recursive Techniques (ARMA)
	NeXT
	Maximum Likelihood Methods
	Stochastic Subspace Identification Methods (SSI-data, SSI-cov)
Time-Frequency Domain	*Time-Frequency Instantaneous Estimator (TFIE)*

3.3.2 FDD and EFDD

The Frequency Domain Decomposition (FDD) technique presented by Brincker *et. al* 2000 can be considered as an evolution and extension of the Peak Picking (PP) technique.
According to the modal decomposition of the transfer function, presented in Eq. (3.18), and taking into account the relationship between the output spectrum and the transfer function (Eq. (3.26)), the following matrix expression for the output spectrum is obtained:

$$\boldsymbol{S}_{yy}(\omega) = V(i\omega I - \Lambda)^{-1} L^T \boldsymbol{R}_{uu} L^* (i\omega I - \lambda)^{-1*} V^H \tag{3.30}$$

With the assumption that the inputs are not correlated (R_{uu} is a diagonal matrix) and that the mode shapes and the participation factors are orthogonal, then the previous equation can be written as:

$$\boldsymbol{S}_{yy}(\omega) = \mathrm{VC}(\omega) V^H \tag{3.31}$$

where $\mathrm{C}(\omega)$ is a diagonal matrix composed by functions of ω, each of them dependent on the natural frequency and modal damping ratio of only one mode of the structure, and V is a matrix whose columns represent the mode shapes.
The FDD method is based on the Singular Values Decomposition (SVD) of the estimate of the complete output spectrum matrix, given by:

$$\widehat{\boldsymbol{S}}_{yy}(\omega_j) = \boldsymbol{U}_j \boldsymbol{S}_j \boldsymbol{U}_j^H \tag{3.32}$$

where U_j is a orthonormal matrix ($\boldsymbol{U}_j \boldsymbol{U}_j^H = I$) that contains the singular vectors of $\widehat{\boldsymbol{S}}_{yy}(\omega_j)$ and $\boldsymbol{S}_j$ is a diagonal matrix with the singular values, positive and real eigenvalues of the matrix $\widehat{\boldsymbol{S}}_{yy}(\omega_j)$, distributed in decreasing order. The first singular value corresponds for each frequency to the power spectrum of one degree of freedom system in accordance to the significant mode shapes that contribute for the response. The next step is the representation and the spectrum analysis of the singular values $\boldsymbol{S}_j$ for the selection (picking) of the resonant peaks and corresponding mode shapes and the evaluation of the modal component of the measured degrees of freedom.

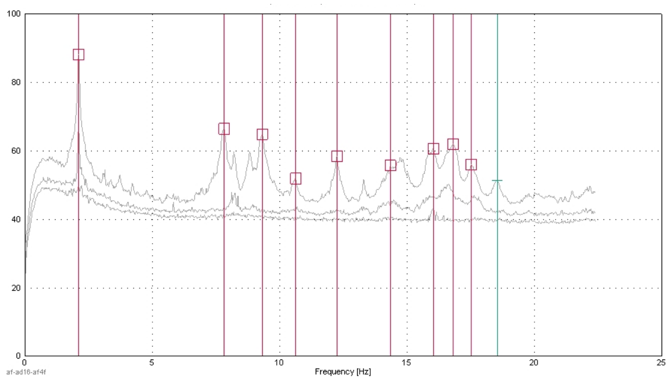


Fig. 3.2 - FDD method, Peak Picking: Singular Values Decomposition (SVD) of Spectral Density Matrices (PSD)

The FDD method previously described was improved by Brinker *et al.* (2001) with the Enhanced Frequency Domain Decomposition (EFDD) technique. Basically, the first steps of the EFDD is equal to the FDD method but the estimation of frequency values and damping coefficients are calculated by the application of the inverse FFT to a set of points with similar singular vectors around the selected pick, known as spectral bell. The obtained auto-correlation response function with the contribution of a single mode is now a typical response of a free-decay single degree of freedom dynamic system. By fitting an exponential function to relative maxima of the obtained correlation function it is possible to extract modal damping ratios. Before the determination of the modal damping ratio, an enhanced estimate of the natural frequency may be obtained from the time intervals between zero crossings of the correlation function.

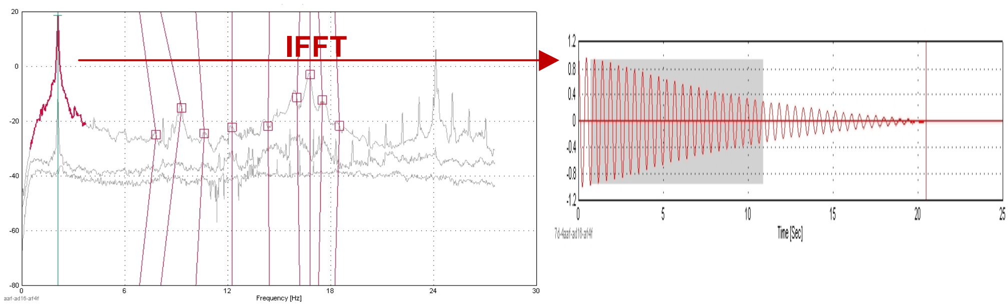


Fig. 3.3 - EFDD method: estimation of the modal damping ratio through the application of the Inverse FFT

FDD and EFDD are user-friendly and fast dynamic identification techniques and their success lead to the implementation of their algorithm into many commercial software such as ARTEMIS (SVS, 2006). Those methods have been largely implemented also within this research work.

3.3.3 p-LSCF

The poly-reference Least Square Complex Frequency domain (p-LSCF), also known by its commercial name PolyMax (LMS) is a recent parametric frequency domain technique, firstly developed as an input-output method to perform the identification of modal parameters from FRFs. However, for a system excited by white noise, there are some similarities between the modal decomposition of a FRF (Eq.(3.18)) and of the half-spectrum (Eq.(3.29)). Thus the techniques was extended to an output-only version by Peeters & Van der Auweraer 2005.
The method starts from the half-spectrum matrix and the right matrix-fraction model (see §3.2.4) in the so called z-domain, i.e. a frequency-domain model that is derived from a discrete-time model ($z = e^{j\omega\Delta t}$). Therefore considering Eq. (3.23), adopting polynomials of the same order p for $\boldsymbol{B}_r$ and $\boldsymbol{A}_r$, the half spectrum, which is assumed to represent the measured FRFs, evaluated at a given discrete frequency ω_j is modeled by:

$$\boldsymbol{S}_{yy}^{+}(\omega_j) = \boldsymbol{B}\boldsymbol{A}^{-1} = \left[\sum_{r=0}^{\mathrm{p}} \boldsymbol{B}_r e^{j\omega_j \Delta t r}\right]\left[\sum_{r=0}^{\mathrm{p}} \boldsymbol{A}_r e^{j\omega_j \Delta t r}\right]^{-1} \tag{3.33}$$

where $\boldsymbol{S}_{yy}^{+}(\omega_j) \in \mathbb{C}^{l \times m}$ is the half-spectrum matrix containing the FRFs between all m inputs and all l outputs; $\boldsymbol{B}_r \in \mathbb{R}^{l \times m}$ are the numerator matrix polynomial coefficients; $\boldsymbol{A}_r \in \mathbb{R}^{l \times m}$ are the denominator matrix polynomial coefficients, p is the model order and Δt is the sampling time.

Eq. (3.33) can be written for all values of the frequency axis of the FRF data. Basically, the known polynomial coefficients $\boldsymbol{B}_r$ and $\boldsymbol{A}_r$ are then found as the least-square solution of these equations, after linearization. For a detailed description of this method the reader can refer to Guillaume *et al.* 2003 and Peeters *et al.* 2004. Once the denominator coefficients $\boldsymbol{A}_r$ are determined, poles and modal participation factors are retrieved as the eigenvalues and eigenvectors of their companion matrix:

$$\begin{bmatrix} 0 & I & \dots & 0 & 0 \\ 0 & 0 & \dots & 0 & 0 \\ \dots & \dots & \dots & \dots & \dots \\ 0 & 0 & \dots & 0 & I \\ -[\alpha_0^T] & -[\alpha_1^T] & \dots & -[\alpha_{P-2}^T] & -[\alpha_{P-1}^T] \end{bmatrix} \cdot V = V\Lambda \tag{3.34}$$

The modal participation factors are the last m rows of $V \in \mathbb{C}^{mp\times mp}$; the matrix $\Lambda \in \mathbb{C}^{mp\times mp}$ contains the (discrete-time) poles on its diagonal. They are related to the eigenfrequencies ω_j (rad/s) and damping ratios ξ_j (–) as follows (* denotes complex conjugate):

$$\lambda_j, \lambda_j^* = -\xi_j\omega_j \pm j\sqrt{1-\xi_j^2\omega_j} \tag{3.35}$$

This procedure has to be repeated as many times as the number of tried model orders p, so that the modal parameters associated with different model orders are represented in a stabilization diagram, which facilitates the selection of the physical modes of the tested structure. Theoretically, once poles and modal participation factors are estimated, mode shapes could be derived from the coefficient $\boldsymbol{B}_r$ and $\boldsymbol{A}_r$. However, in order to estimate the mode shape it is also possible to consider the so called pole-residue model:

$$\boldsymbol{S}_{yy}^{+}(\mathrm{j}\omega) = \sum_{j=1}^{n} \frac{\{v_i\} < l_i^T >}{j\omega - \lambda_i} + \frac{\{v_i^*\} < l_i^H >}{j\omega - \lambda_i^*} - \frac{[LR]}{\omega^2} + [UR] \tag{3.36}$$

where n is the number of modes; $.^H$ denotes complex conjugate transpose of a matrix; $\{v_i\} \in \mathbb{C}^l$ are the mode shapes; $< l_i^T > \in \mathbb{C}^m$ are the modal participation factors and λ_i are the poles (Eq.(3.35)). $[LR]$ and $[UR]$ are respectively the lower and upper residuals modeling the influence of the out-of-band modes in the considered frequency band. The interpretation of the stabilization diagram yields a set of poles λ_i and corresponding participation factors $< l_i^T >$. Since the mode shapes $\{v_i\}$ and the lower and upper residuals are the only unknowns, they are readily obtained by solving Eq. (3.36) in a linear least-squares sense. This second step is commonly called Least Squares Frequency Domain (LSFD) method (Peeters *et al.* 2004).

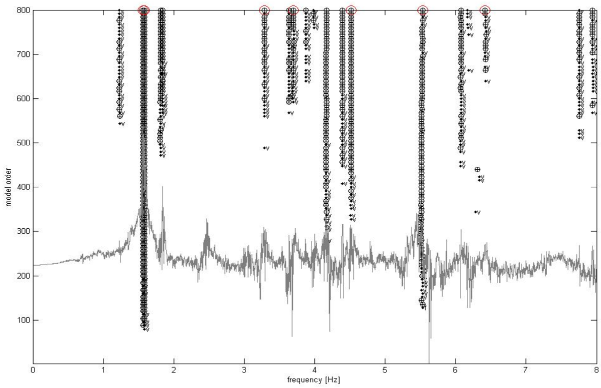


Fig. 3.4 - p-LSCF method: stabilization diagram implemented for the selection of physical modes

The p-LSCF algorithm is considered one of the most robust and accurate parametric OMA technique in frequency domain. The obtained stabilization diagrams look very clear as the p-LSCF method forces mathematical (spurious) modes, that arise due to an over estimation of the model order, to have negative damping. The method was implemented also in the Matlab Toolbox (MACEC 3.2). For these reasons the method was chosen for the implementation of the automated modal parameter estimation, presented within this thesis.

3.4 Conclusions

In this chapter an introduction to the basic concepts of structural dynamics together with a state-of-the-art review of some Operational Modal Analysis techniques was presented.

Dynamic identification of modal parameters through output-only modal analysis proved to be a very effective tool, especially when applied to large civil engineering structures or to structural systems under operational conditions. A big research effort was performed especially in the last decade in order to improve the available OMA techniques and develop robust and reliable method to extract fundamental information about the dynamic response of structures.

After a short introduction on basic dynamics, three frequency domain identification techniques were reviewed and analyzed in detail: the FDD and EFDD methods, non-parametric techniques, fast to implement and extremely user-friendly, and the

p-LSCF, a parametric technique with a more complex theoretical background that prove to be very robust and effective.
All methods have been extensively applied to several case studies presented within the present research work, as well as during the development of automated algorithms for OMA..

4 SHM OF CULTURAL HERITAGE BUILDINGS: METHODOLOGICAL ASPECTS

4.1 Introduction

This chapter provides the development of a methodology for the application of monitoring techniques to historic buildings and monuments. The European research project NIKER, recently concluded, tried to face the issues related to SHM application to CH structures. In one of NIKER's Workpackages (i.e. WP9) the tentative application of inspection techniques and monitoring strategies to selected CH buildings was proposed in order to define a knowledge-based procedure for their assessment and protection, in particular through SHM. The outcomes of this work, to which the University of Padova actively contributed, are partially reported and discussed within this chapter.

4.2 General knowledge-based methodology

Knowledge-based methodologies for the study of heritage buildings are based on the exploitation and integration of different approaches including historical research, inspections, monitoring and structural analysis. Monitoring, in particular, is carried out across the entire process and may be utilized not only for diagnosis but also as an auxiliary tool during the intervention and even during the later post-intervention period for control and preventive maintenance purposes.
In the complex knowledge process of historic buildings four different activities are developed in a parallel and interconnected way. The proposed general strategy, as represented in Fig. 4.1, includes:

(i) Inspection
(ii) Monitoring
(iii) Modeling (or structural analysis)
(iv) Intervention

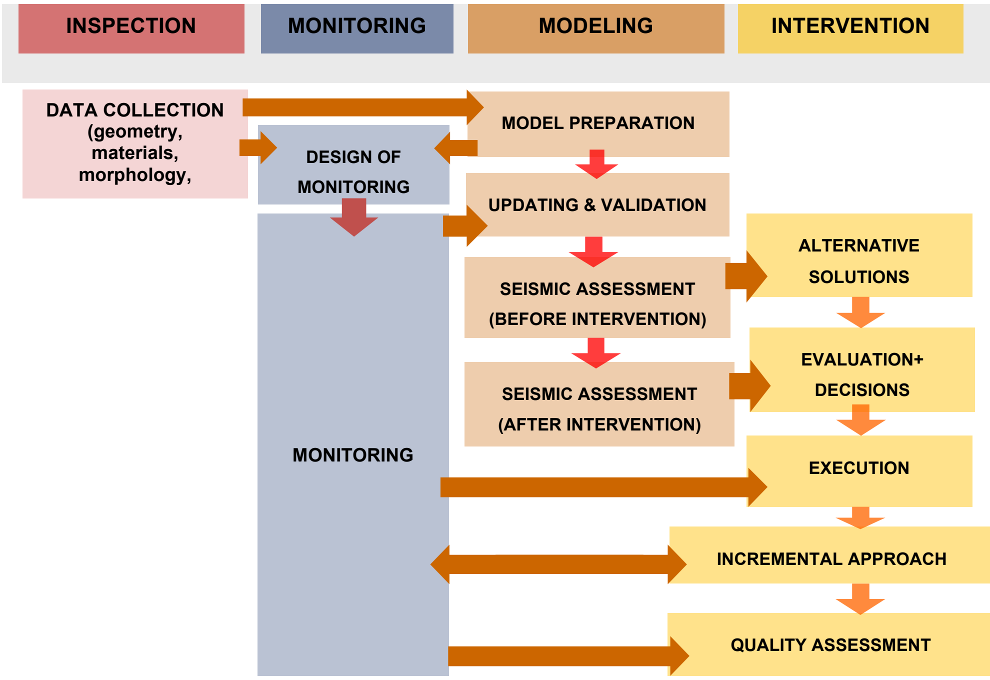


Fig. 4.1 - General knowledge-based strategy for the study and assessment of CH buildings (WP9, NIKER Project 2012)

Firstly the inspection phase is carried out to provide all the data needed for a characterization of the building regarding geometry, materials, morphology and damage. This information is used for two different purposes: (i) preparation of the numerical model input data and (ii) design of the monitoring system and selection of the best monitoring strategy. Inspection comprises visual inspections and Non-Destructive Tests (NDT) oriented to identify the material composition, existing alterations, working stress levels, material properties and damage distribution, among other aspects. In turn, an initial structural analysis may assist in taking decisions on the type of monitoring to be implemented (type, accuracy and range of sensors, number of measurements and critical locations).

Monitoring is carried out during the entire duration of the process in parallel to the inspection, modeling and intervention design activities. A first set of monitoring results (comprising both static and dynamic monitoring) can be used to carry out model updating and validation. In particular, modal matching based on dynamic monitoring results can be used to update reference FE models. The model can be as well validated or upgraded using results from inspection by comparing the numerical predictions (on cracking, deformation, work stresses, etc.) with real observations on the present condition of the structure.

Once the model is sufficiently updated and validated, it is used for seismic assessment using different approaches of analysis. In particular, significant effort is given to the comparison between the response of the structure before and after the execution of strengthening interventions that are going to be implemented. Structural analysis can be used in order to simulate the performance of the structure, strengthened with different proposed solutions and hence conclude on the distinct benefit provided by each of them. The different solutions can be categorized regarding their capacity to improve the seismic performance and efficiency. However, compliance with restoration principles and minimum intervention are also to be considered. Normally, the solution to be chosen should be the one which, while providing the targeted performance and safety requirements, causes the minimal loss of cultural value in the building.

The general strategy also accounts for tight interaction between monitoring and intervention. Monitoring is necessary for control purposes during and after the implementation of the upgrading solution. In particular, a step-by step (incremental) approach can be envisaged where each operation (intervention increment) is closely assessed by means of monitoring in order to appraise its effect and benefit on the structural performance. Depending on the results of monitoring, additional operations may be gradually considered. Finally, the adequate performance of the strengthened structure and the strengthening methods and devices will be subjected to short and long term survey (quality assessment) to evaluate and ensure their adequate condition and performance (WP9, NIKER Project 2012).

4.3 Role of monitoring

Looking more in detail at the role of monitoring within the previous defined general strategy for the study and assessment of historic buildings, it is possible to identify different phases according to the nature of activities carried out on the investigated building. In each phase monitoring plays a fundamental and active role. The different phases considered are:

(i) Investigation phase
(ii) Intervention phase
(iii) Evaluation phase
(iv) Maintenance phase

In spite of the distinction between the different phases, they are intimately connected and any general action on a historical structure should plan all of them according to unified approaches and criteria. Ideally, similar experimental and numerical tools should be utilized in all the phases, and an adequate and balanced technical and budgetary effort should be devoted to all of them. However, it is also recognized that different problems (regarding the condition of the building, the seismicity, the structural technology and the economical or technical constrains) may lead to combine the phases in different ways or to assign them different relative weights.

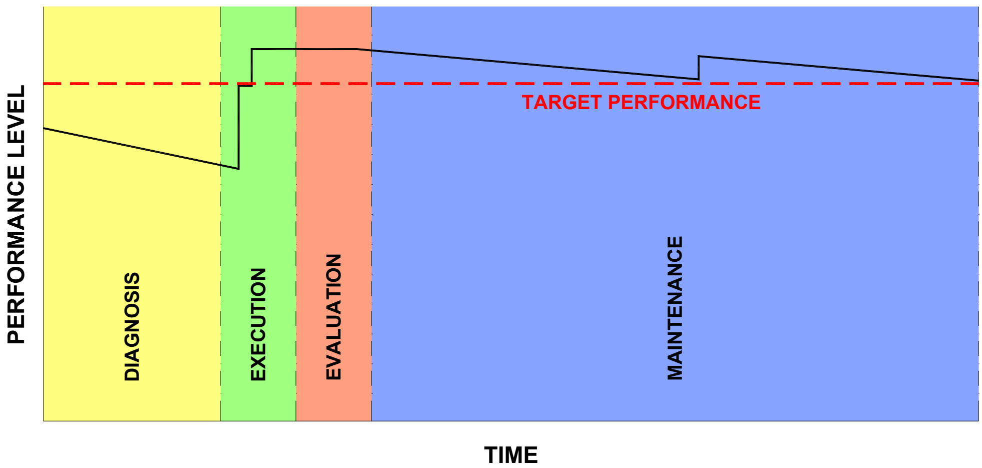


Fig. 4.2 - Monitoring phases across the knowledge-based process for the assessment and protection of CH buildings (WP9, NIKER Project 2012)

Fig. 4.2 schematizes the subsequent phases in the process involving the study, intervention and later control of a cultural heritage structure. In the example, the investigation reveals that structure is not attaining the necessary seismic capacity. The target performance (expected seismic capacity) is attained and even surpassed after the execution of intervention, which may include some distinct increments if an incremental approach is applied. An evaluation phase follows to verify the efficiency of the intervention. The last phase, and longer one, is the maintenance one during which the structure is monitored to verify the maintenance of the expected seismic capacity. Possible correction actions are undertaken if it decreases below the tolerable limit. At the end of the maintenance period a new seismic assessment, with a possible new intervention, may be necessary.

4.3.1 Investigation phase

This phase represent the initial and crucial stage in the knowledge-based assessment of historical buildings. The aim is to characterize structural conditions and reliability of the investigated building and the intervention needs. More specifically, it involves detailed inspections, diagnosis on the ultimate causes of damage and deformations, structural verification (including seismic assessment) and identification of the need for stabilization, repair or upgrading.

During this phase monitoring can be integrated for different, but connected, purposes:

- *Dynamic characterization*. The application of OMA techniques or dynamic monitoring (both continuous or trigger-based) permit to extract and identifies the dynamic properties of the structure both under operational conditions and in case of exceptional events. Parameters related to the dynamic response are: natural frequencies, mode shapes and damping ratios. The distinction and comparison of the dynamic response under normal conditions or in case of relevant events becomes crucial for the structural characterization, which can be also considerably different when the building is subjected to vibrations of large amplitude. In post-earthquake scenarios, a continuous dynamic monitoring can be implemented to characterize the marginal response of damaged (or even severely damaged) structures under the effect of meaningful replicates.
- *Model updating*. Outcomes of monitoring (both in terms of static measurements and dynamic response) can be exploited to calibrate and updated reference numerical models. During this process, experimental outputs are compared with numerical predictions resulting from a numerical model. As a second possibility, a dynamic analysis in the time domain can be performed by analyzing the response of the numerical model for accelerograms corresponding to earthquakes conveniently captured during the monitoring period. The dynamic response actually displayed by the building during the earthquake, regarding displacements, damage and possible collapse mechanisms, is then compared with the numerical prediction and again the model is improved until satisfactory agreement is obtained. The modal improvements or updating may target to the material properties, the geometry and morphology of structural members, the modeling of the connections, the influence of the soil and possible soil-structure interaction effects, the influence of neighboring buildings and the damage distribution (particularly, the influence of large individual cracks or separations).
- *Damage identification*. The application of damage detection algorithms (as described in §2.3.3) can be performed both on static and dynamic data to identify any possible active deterioration or damaging process. For this purpose it is essential to monitor also environmental effects (especially temperature and relative humidity) that might have a large influence on the variation of the monitored parameters (daily and seasonal cycles). Once the environmental

effects are filtered out, it is possible to accurately decompose the measurements into their reversible and irreversible components, the latter being associated to the active deteriorating processes. Damage identification techniques applied to masonry structure based on vibration signatures become a very difficult task. Another possibility is to use dynamic monitoring as a tool allowing some characterization of the influence of major damage (as large individual cracks or separation between structural members) on the overall structural response. This characterization can be carried out through an updating process over a structural model in which this type of damage is conveniently simulated.

- *Need for emergency actions*. Finally, both static and dynamic monitoring can be considered to detect anomalous changes alerting of a possible worsening of the performance or the stability condition, especially in post-earthquake scenarios. This evidence can assist in decision taking on emergency stabilization or strengthening actions.

4.3.2 Intervention phase

The proposed general strategy also accounts for a strong interaction between monitoring and intervention. Monitoring is necessary for control purposes during and after the implementation of the upgrading solution. Monitoring can be used (and should normally be used) during the execution of the intervention to control and verify its correct implementation. For this purpose, the monitoring implemented during the previous investigation phase must be kept active, with the necessary modifications, during the execution and after.

Moreover, monitoring can have a very active role in decision taking on the extent of the intervention through an incremental approach. A step-by step (incremental) approach can be envisaged where the response of the structure after each operation (intervention increment) is closely assessed in order to evaluate the resulting gain in structural performance and the possible need for further upgrading. Depending on the results of monitoring, additional intervention increments may be gradually considered. This procedure can be utilized to produce actually optimized interventions and to obtain an accurate understanding of their effect on the structure.

4.3.3 Evaluation phase

After the full execution of the intervention, a subsequent monitoring phase is proposed, extended to a limited period of time, during which the upgrading solutions are carefully evaluated. The evaluation period is oriented to control the correct implementation of the upgrading solutions and to verify that the expected improvement has been actually achieved. The response of the strengthened structure is analyzed and the performance of the strengthening solutions is carefully investigated regarding their efficiency and actual influence on the response of the structure.

The monitoring technologies to be utilized during this phase are similar to those proposed for the investigation and intervention phase; however, their use is extended to a longer period allowing an appreciation of the variation (or maintenance) of the upgrading effect on the structure.

Sensible variations observed through the new monitoring during this phase should lead to update the structural models and then re-use them to carry out a new seismic assessment.

The duration of the evaluation phase may involve variable periods from a few weeks or months to even one or more years, depending of the nature of the problem, the importance of the building and the strategies applied.

4.3.4 Maintenance phase

The performance level of the strengthened structure as well as strengthening methods and devices can be assessed thank to a long-term monitoring program. In this framework the purpose of long-term monitoring is to control the response of the repaired structure and the maintenance of the expected efficiency of the structural/seismic upgrading solution. The response of the structure can be continuously or periodically surveyed. One of the aims of this survey is the detection of unexpected responses that may alert of possible problems. It starts after the evaluation phase and lasts up to the end of the maintenance period. It includes the necessary quality control plans, maintenance work, preventive maintenance and corrective measures. When necessary, improvements or corrections are undertaken to secure the required efficiency levels

4.4 Compliance with conservation requirements

During the design process of the architecture of a monitoring system and the selection of the optimum monitoring strategies it is of fundamental importance to define a set of requirements oriented to ensure its adequate performance and ability to provide valuable information. One of the most important requirements considered in the selection of the monitoring techniques is their adequacy for the application on heritage structures. Such adequacy results from some general criteria on conservation and restoration of architectural heritage, firstly reported in ICOMOS/ISCARSAH Guidelines (2003) and the Italian Guidelines for evaluation and mitigation of seismic risk to cultural Heritage (2006). These general principles and their establish validity can be transferred to the specific application of SHM to CH structures, as presented hereinafter:

- *Minimum intervention.* Acceptable interventions should grant a minimum impact on the original structure. When applied to a monitoring system, this condition leads to the preference for efficient systems requiring light and small devices. Systems and devices not requiring any physical/mechanical impact on the material surfaces (for instance, thermovision) are advantageous from this point of view. The ability to obtain valuable information from structures, as surface and subsurface data, without touching the work of art, proves that the application of non-destructive tools such as the sonic pulse velocity tests is indispensable for the assessment of monumental structures with high cultural value.
- *Safety.* Solutions utilized to implement the systems should grant the necessary safety for people (for instance, avoiding the risk of detachment and fall of devices).
- *Compatibility.* All the equipment and auxiliary devices and particularly the methods utilized to fix the sensors to the surfaces should not cause any meaningful and lasting problem to the original material and structure. Chemical/physical lasting effects resulting from the use of adhesives need to be considered. In the case of walls or vaults with valuable fixed artistic contents (frescoes, mosaics), the use of adhesives or insertions may not be possible due to the need for respecting their integrity.
- *Durability.* Even if the system is intended to work during a limited period, the devices and auxiliary components should be free of possible deterioration problems (for instance, metal corrosion) which may cause deterioration on the original material and surfaces.

- *Non-invasiveness.* Any solution for implementing and fixing the systems and devices should in general avoid the need to perforate and anchor fixing systems in original material. If insertions are needed, they should preferably be inserted in joints rather than in stone surface.
- *Reversibility.* Monitoring systems will normally be implemented for a limited period of time, while also needing maintenance and substitution operations. It will be important to ascertain the possibility of dismantling or substituting partially or totally the system without causing any meaningful impact or deterioration on the surfaces and material.
- *Non-obtrusiveness.* The original aspect and aesthetics of the construction and surfaces need to be preserved when the structure is in use and visitors are allowed during the monitoring period.
- *Presentation.* However, it may be adequate to present the systems and devices in a distinguishable way (without compromising the aesthetics of the structure) in order to permit the recognition of on-going works and studies carried out on the building. For the same reason, information panels can be placed in adequate locations in the building to provide information to visitors on the works, systems utilized and expected results.

4.5 Objectives of monitoring

In the framework of the general knowledge-based methodology for the study and assessment of CH buildings and the role of monitoring within different phases, this paragraph is devoted to the discussion on the application of SHM to face specific problems, as discussed in the introduction of the thesis.

Starting from the experience gained through the implementation of monitoring systems on several and diverse case studies, SHM, as described in the introduction of the thesis, proved to be a very effective and reliable tool in order to:

a) Increase the knowledge on the structural behavior using SHM to assess strengthening needs and avoid the execution of unnecessary interventions;
b) Apply an incremental approach to the execution of strengthening interventions using SHM before, during and after the implementation, validating eventually their effectiveness;
c) Post-earthquake controls on severely damaged buildings using SHM to control the evolution of damage and verify the effectiveness of provisional strengthening measures.

4.5.1 SHM to assess strengthening needs

In the first problem type SHM is applied as a possible strategy to evaluate strengthening and retrofitting needs, avoiding, when and if possible, the execution of invasive strengthening interventions. This possibility may emerge from the increase of knowledge on the structural behavior of the investigated building, allowed by SHM. The strategy is mainly applicable to buildings for which there are some evidences (because of the structural typology, limited damage and the result of previous structural analyses) of a sufficient or almost sufficient structural/seismic capacity. Instead of implementing a new strengthening to forcefully grant the full capacity of the building, it is preferred to rely upon a detailed continuous monitoring as a way to assess the condition of the building and possibly gather additional evidence on the seismic positive response of the building. When the knowledge level on a specific structure is sufficiently high, the damage state, the structural response and the vulnerable elements may be adequately characterized. This characterization allows for the definition of safety thresholds allowing to keep the building under control by means of a SHM system, and to postpone the execution of possible interventions unless a worsening of the structural conditions is recorded (WP9, NIKER project 2012). In this research work this strategy has been successfully applied to the Arena of Verona.

4.5.2 SHM to apply sequential interventions

The second problem type refers to the on-site validation of the effectiveness of strengthening solutions by using SHM. In this case monitoring plays a very important and active role in taking decisions on type and extent of strengthening measures through the application of an incremental approach, i.e. a step-by-step procedure for the knowledge-based assessment and the progressive implementation and evaluation of interventions. In order to maximize the benefits of SHM it is necessary to apply specific monitoring techniques before, during and after the execution of interventions, following any kind of changes in the structural response of the system after each incremental step (see §4.3.2). In this way, if necessary, the strengthening system can be upgraded or substituted by a new one. For that purpose, inspection, monitoring and structural analyses are combined to allow for a better understanding of the capacity of the strengthened building and for identifying additional strengthening needs (WP9, NIKER project 2012). This approach is applied within the thesis to the case study of the Cansignorio stone tomb in Verona.

4.5.3 SHM for post-earthquake controls

Another type of problem concerns the study of the vulnerability of buildings severely damaged by an earthquake. In case of a seismic event or natural disaster, where many buildings in a small area may be affected and damaged, SHM can furthermore prove its usefulness in order to: (i) evaluate quantitatively the progression of the damage pattern, (ii) carry out effective and urgent interventions if an unsafe displacement patterns is recorded; (iii) define an early warning procedure for the safety of the workers employed in the strengthening interventions. Monitoring can also be effective as a management tool of emergency actions, when implemented on seriously damaged buildings, if the time schedule for the execution of provisional or definitive interventions is difficult to be a priori planned.

The problem also includes the design of provisional strengthening measures to prevent for further collapses and stop the on-going damage process activated by the earthquake. This strategy is oriented to analyze the response of the damaged building and to design provisional stabilization to control or stop further damage, including possible partial collapses activated by the earthquake. Monitoring is applied before and after the execution of the stabilization technique in order to obtain information on the response of the damaged building and also to control and validate the efficiency of the applied solution. After the conclusion of the emergency phase and the implementation of provisory stabilization measures, there is the need to design definitive structural solutions oriented to repair and sufficiently improve the future seismic response. In this type of problems, the application of step-by-step procedures based on monitoring is of large interest to limit the amount of strengthening provided to the structure (WP9, NIKER project 2012).

This approach has been applied to several monuments and building heavily damaged by the 6th of April 2009 earthquake. In this work the case studies of the Spanish fortress and the Civic tower will be reported.

4.5.4 Exploitation of monitoring results

In order to achieve the objectives of SHM applied to CH buildings, previously stated, a fundamental step is represented by the active exploitation of monitoring results. This phase is crucial to increase the knowledge level of the monitored structure, trying to assess its static/seismic capacity and performance, intervening, in case, through some structural improvements.

The adopted procedure should therefore provide some indications on the possibility to use monitoring outcomes that can be fruitfully addressed to:

a) Control the structural behavior under operational conditions;
b) Construct, calibrate and validate reference behavioral numerical models;
c) Characterize the structural response in case of exceptional events.

These cases will be investigated in detail in the final chapter of the thesis, where the results of monitoring applied to selected case studies are reported. For this purpose it is necessary to develop integrated methodologies based on data driven and model driven approaches to apply subsequently damage detection algorithms and model updating techniques, starting from information extracted from recorded data.

4.6 Conclusions

In this chapter some methodological aspects related to the application of SHM to CH buildings and monuments are developed and discussed, according to the fundamental outcomes of the European research project NIKER that gave an essential contribution in the transfer of already well-established monitoring techniques and methodologies in the field of conservation and protection of historical constructions. Firstly the general methodology for the study and assessment of historic buildings is reported, emphasizing the importance of the so called knowledge-based approach within which monitoring represents a key phase, able to interconnect inspections, structural analysis and interventions.

Then the role of monitoring is more specifically identified in the framework of the complex activities carried out on historic buildings, from investigations to the execution of interventions, from the evaluation of the effectiveness of the proposed strengthening solutions to the ordinary maintenance activities. In each phase monitoring plays a fundamental and active role that is clarified and defined in detail.

Afterwards it is demonstrated how some general and well-known conservation principles, traditionally applied during the design phase of interventions on CH buildings, can be successfully transferred to the implementation of SHM systems. It is important in fact to respect those principles during the entire conservation process of heritage structures.

Finally the application of monitoring techniques to face specific problems is presented and discussed as well as some procedures based on data driven and model driven approaches to exploit profitably monitoring results.

The general methodology developed and described within this chapter has been then validated through a real application on representative case studies, which will be presented in the next chapter.

5 APPLICATION OF SHM TO SELECTED CASE STUDIES

5.1 Introduction

This chapter presents and analyses some selected case studies, showing aims and needs of monitoring and presenting in detail preliminary inspections and the successive design and installation phases of the monitoring systems.
Case studies selection was performed according to the following aspects: (i) choice of a sufficiently wide range of CH buildings, characterized by different typologies, structural features, seismic vulnerabilities; (ii) seismicity levels of the sites; (iii) aims of monitoring to face different problems, following the methodology described in the previous chapter:

a) SHM as an alternative to strengthening: Arena of Verona.
b) SHM applied before, during and after interventions: Cansignorio stone tomb in Verona
c) SHM for post-earthquake controls: two representative case studies in l'Aquila, i.e. the Civic tower and the Spanish fortress.

5.2 Roman Arena of Verona: SHM as an alternative to strengthening

Ancient structures - especially very old ones - prove their soundness and the correctness of their structural layout by reaching our days in relatively good conditions. This is the case of the Roman Arena in Verona, Italy, built in the I century A.D., and still standing in the historical city center of Verona. It is undoubtedly the symbol of the city and it is open to public use for visits but also for operas, concerts and relevant shows. However with a closer look, it is possible to appraise damage that the passing of time and the natural or man induced events such as historical earthquakes, floods or wars and sieges left on the structure. Past seismic events induced serious damage to the Arena, being the cause of the almost complete collapse of the third (external) ring of the monument, today only remaining in the so called "ala" (wing) of the Arena, an isolated portion of stone blocks curved wall characterized by a repetition of arches and massive pillars.

With the purpose of evaluating the structural response of the Arena to static, dynamic (e.g. shows, concerts) and seismic loads, a SHM system was installed in the Arena in November 2011, with state of the art technology for data recording in relevant positions of the monument.

Since no specific strengthening interventions are foreseen in the immediate future, apart from the ordinary maintenance, SHM becomes an interesting tool to be applied to this specific monument as an alternative to strengthening. The idea is to increase the knowledge level on its structural performance and thus have a deeper insight on its conditions. It allows intervening with more confidence, only if really necessary.

5.2.1 Historical notes and structural features

The Roman Amphitheatre or “Arena” represents the most important roman monument in Verona, a city in Northern Italy between Venice and Milan. The building, one of the best preserved ancient structures of its kind, is internationally famous for the large-scale opera performances given there. It is still today able to accommodate an audience of 20’000 persons (Fig. 5.1).

The monument was erected between the second and the third decade of the first century AD and it was constructed outside the late Republican walls of the city.

The Arena main axis is aligned with the Cardo Maximus (main alley in Roman cities in N-S direction) (Beschi 1960).

Fig. 5.1 - Aerial view of the Roman Amphiteatre of Verona, the “Arena”

The round façade of the building was originally composed of white and pink limestone from Valpolicella, but after a major earthquake in 1117, which almost completely destroyed the structure's outer ring, except for the so-called "ala" (the remaining section of the outer ring, Fig. 5.2), the stone was quarried for re-use in other buildings.

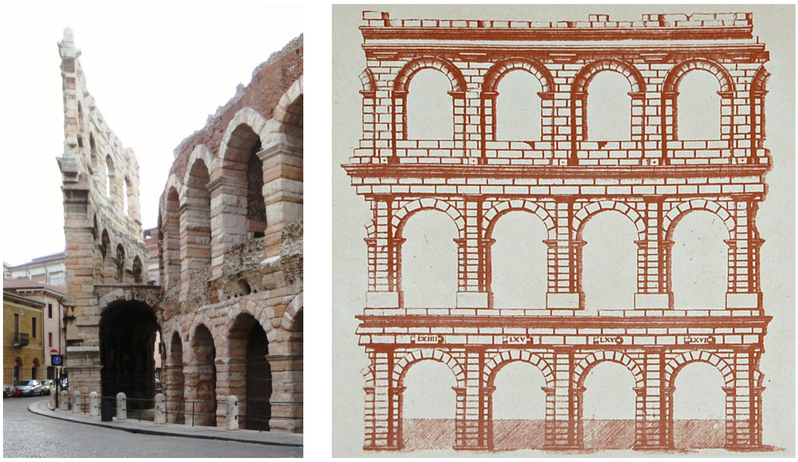


Fig. 5.2 - The symbol of the Arena: a freestanding wall characterized by a repetition of arches and massive pillars, survived from the collapse of the external ring of the monument.

Its dimensions are about 138 x 109 meters on the outside while the inner elliptic pit is about 44 x 73 meters. There are 45 tiers of steps wherein the opera audience can sit. The building, as other structures of its kind, is composed by a symmetrical arrangement of radial masonry walls separating the 72 “Arcovoli” (accesses to the courtyard) and three concentric distributive elliptical galleries. Main employed materials are the “opus caementicium” that is to say the roman concrete, and a massive squared stone masonry block masonry (Fig. 5.3).

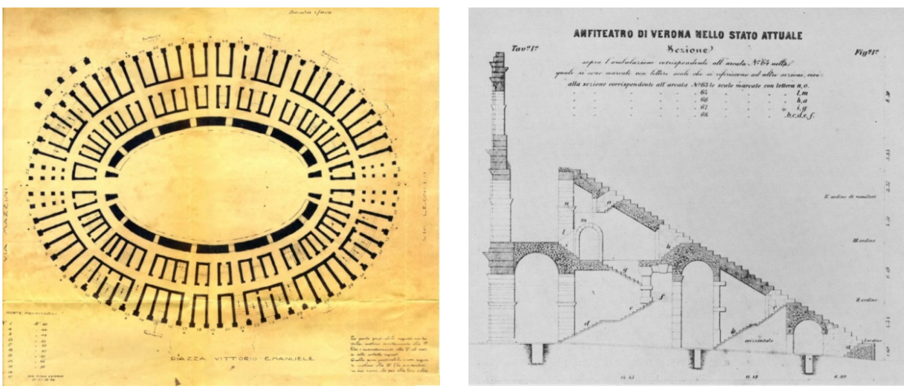


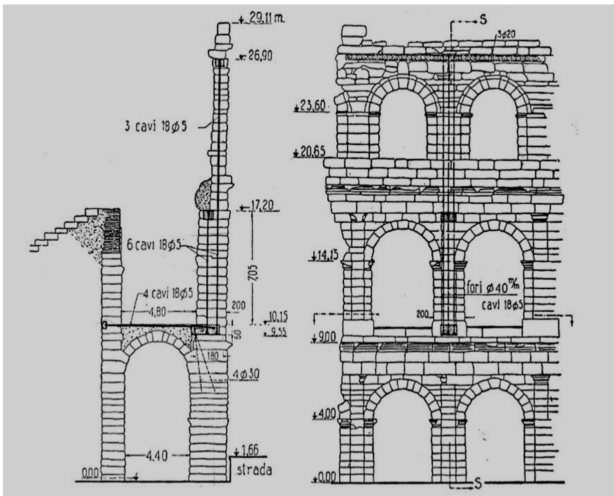
Fig. 5.3 - Plan of the ground floor and transverse section of the amphitheater

The monument was subjected to several natural disasters and man-induced damages, like the flooding of 589 AD and the earthquakes of 1116 and 1117. Those events induced major damages to the structure with the almost complete collapse of the outer ring of the Arena, the collapse of 5 arcades on Bra square in 1579, and the continuative stealing of stones and material for reuse in other constructions. Only starting from the XVIII century the Arena was treated as an "historical" monument, and several conservation actions started with archeological excavations and ambitious restoration programs.

5.2.2 Past interventions

XX century structural interventions and induced damages are documented more in detail: in 1939, both the need of intervening on the external "Ala" for manifest tilting and the necessity of protecting the same structural portion from the possible damages induced by the II World War consequences, led to the construction of massive buttresses, removed after the end of the war by an intervention of post tensioned tendons vertically placed along the entire height of the massive pillars (Morandi, 1956) (Fig. 5.4).

a)



b)

Fig. 5.4 - Repair - strengthening interventions in the "ala" of the Arena: a) 1939 stabilizing interventions; b) 1956 Morandi's intervention

The intervention, as described by engineer Morandi, consisted essentially in introducing harmonic steel wires in vertical holes (40 mm in diameter) drilled in the stone. Unfortunately, some of the most critical issues of the intervention are ignored or contradictorily described in the cited reports, so that it is very difficult now to draw any conclusion on the actual and present efficiency of the intervention itself. This is the case of the very critical stability conditions occurring at the base of the wing, where the overturning moment induced by horizontal forces has the maximum value and the stabilizing effect of the tensioned wires suddenly is missed. They are ignored in Morandi (1956), where only horizontal forces acting towards the outside of the amphitheater are considered (Zonta 2000).

5.2.3 The monitoring system

5.2.3.1 Needs for monitoring

In recent years the Verona municipality manifested a visible interest for controlling the structural response of the structure to different external actions, also considered its relevant use. Vibrations caused by pop or rock concerts were and still are in fact object of discussion, especially for what concerns structural safety of the monument connected to the use of it and the conservation issues. Several investigation campaigns were performed in the last decades, both for defining the state of stress of the composing materials and for measuring the vibration level in the masonry structures. Maintenance is another key aspect of the monument, since hundreds of thousands of people visit the arena yearly, and thus their safety has to be guaranteed.

Also, being Verona located on a seismic prone area, even if moderate, the seismic associated risk has to be evaluated and minimized, if possible, and monitoring may be a viable and effective option for assessing the response of the structure in case of an earthquake and for calibrating behavioral reference models.

In 2011 the Verona Municipality gave the task to the ICEA Department of the University of Padova to install and manage a SHM system for assessing the main structural parameters of the Arena in order to assess more in detail its structural safety. The final aim is the acquisition of the vibration characteristics of the monument by means of acceleration transducers, and the control of the surveyed crack pattern through the implementation of displacement transducers installed on the main cracks. The acquired data are constantly related to the environmental parameters (temperature and relative humidity).

The evaluation of the measured quantities, and in particular their changes over time, allows having useful indications in the definition of the structural behavior and in the determination of the presence or occurrence of damage’s phenomena.

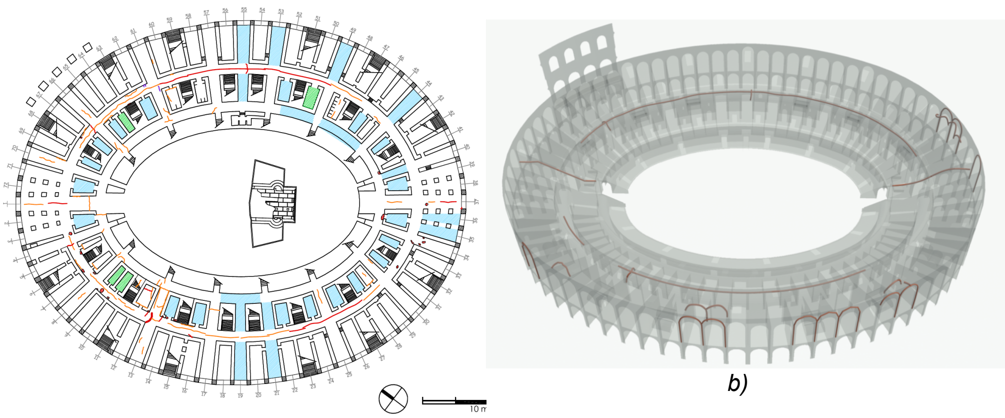
5.2.3.2 Preliminary investigations

The preliminary phase before the installation of the SHM system consisted in: (i) design of the system’s architecture (hardware - types of sensors and acquisition units); (ii) development of appropriate software in relation to the selected monitoring strategy; (iii) choice of the system’s layout and the points of structural control

according to the analysis results of reference numerical models, the survey of the damage pattern and the results of dynamic identification tests of the structure.
Several investigations were carried out in the last decade on the monument, including geometric and material survey, visual inspections and deep inspections by means of Non Destructive Tests - NDT (dynamic identification tests) and Minor Destructive Tests - MDT (Mechanical continuous coring and flat jack tests) (ISMES 1996).

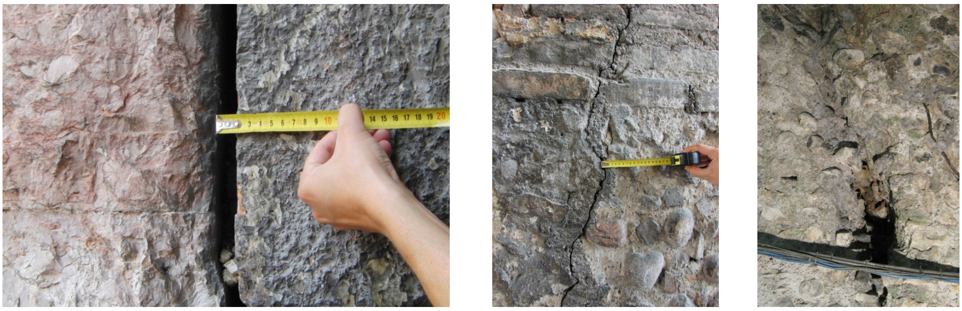
➢ Visual inspections and crack pattern survey

Before the installation of the system those investigations were integrated with the execution of an accurate and detailed crack pattern survey (Fig. 5.5) in order to choose the optimal positions for static sensors (i.e. displacement transducers): the idea is to control either local cracks and/or damaging processes, especially induced by rainwater infiltrations and entire macroelements and/or possible earthquake-induced collapse mechanisms.
It is noted that the structural conditions of the Arena are generally good. Some deep cracks were surveyed mainly on vaults. The majority of visible lesions have smooth edges, meaning that they have been present for many years and mainly caused by rainwater infiltrations and erosion (Fig. 5.6).



a) b)

Fig. 5.5 - a) Crack pattern survey of the ground level; b) 3D representation of the crack pattern main lesions and damages



a) b) c)

Fig. 5.6 - Crack and damage pattern survey: a) detachment of the external layer of squared blocks masonry; b) Cracks on the inner radial walls; c) local damages on vaults caused by rainwater infiltrations.

The general damage state of the Arena can be briefly summarized hereafter:

- The vaulted system, in particular the barrel vaults directly in contact with the terraced steps of the auditorium, shows some instability phenomena to be monitored, so as to exclude the possibility of failure;
- The vaulted niches of the first level (the so called “arcovoli”) present a diffused crack pattern that must be kept under control;
- The detachment of the outer stone leaf from the rest of the structure, visible at the first level, may be indicative of a possible overturning phenomenon of the outer wall and therefore to be monitored;
- The barrel vault of the internal gallery presents a longitudinal crack at the keystone that runs along almost the entire perimeter of the Arena: it is suggested to control the possible damage’s evolution.
- The wing of the Arena (i.e. the freestanding wall in the northern part of the monument) represents the most vulnerable structural element as it is composed by three orders of arches connected with the rest of the structure only at the first level. The strengthening intervention performed during the ‘60s (insertion of post-tensioned cables along the massive stone pillars of the wing from the top to the first level) guaranteed a certain level of pre-compression on the masonry structures of the wing, providing a system of stabilizing forces to the possible overturning mechanism. However, the most critical aspect of the intervention was that the cables stop at the first level and thus the contribution of stabilizing forces is not provided to the entire height of the structural element.

The crack pattern survey of the monument led to the definition of the optimal positions for static sensors, i.e. displacement transducers.

➢ Operational Modal Analysis (OMA)

In order to select the best positions of acceleration transducers a preliminary dynamic identification campaign was performed, concentrated in particular on the definition of the dynamic behavior of the wing, which is certainly the most vulnerable structural element of the monument. The overall dynamic behavior of the wing was experimentally analyzed by means of Ambient Vibration Tests (AVT) and output-only identification techniques.

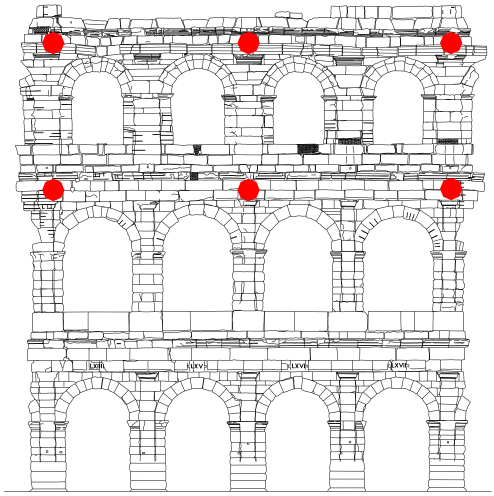


Fig. 5.7 - Positions of acceleration transducers during AVT

A compact unit provided with 24-bit digital acquisition cards, connected to six piezoelectric single axis acceleration transducers, composes the acquisition system. Accelerometers were fixed to the structure in correspondence to the second and third level of arches of the wing, along the out-of-plane direction (Fig. 5.7). Tests consisted in acquiring data using one single setup, over a predetermined period, at a specific sample rate.
Time series acquired at a sampling frequency of 100 Hz are composed by 131’072 points with an overall signal recording duration of 21’51’’. Acquired data were pre-processed by a decimation of 2 (Nyquist frequency of 25 Hz), with segment length of 2048 points and 66.67% window overlap. System identification was performed implementing two non-parametric frequency domain methods: FDD and EFDD (see §3.3.2 for more details). Peaks in the frequency domain related to structural frequencies were selected and the corresponding mode shapes defined (Tab. 5.1 and Fig. 5.9).
Both techniques prove to be very effective and reliable in the identification of modal parameters, with MAC values, calculated comparing mode shapes extracted with the two methods, closed to one (Tab. 5.1). OMA of the wing provided very good

results and 8 mode shapes have been identified in the frequency range 0-11 Hz, meaning that in case of flexible and simple structures like this, ambient vibrations are able to excite sufficiently well all the frequencies in the range of interest. This fact is confirmed looking at the Power Spectral Density matrix represented in Fig. 5.8, where all resonant peaks, corresponding to the structural modes, are well excited and very finely separated.

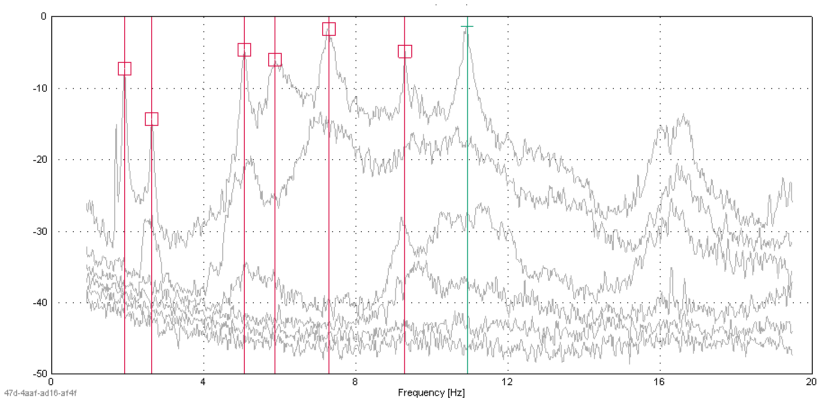


Fig. 5.8 - SVD of the Power Spectral Density (PSD) matrix: peaks in the frequency range 0-11 Hz corresponds to the structural modes of the wing

AVT were performed in October 2011, immediately before the installation of the SHM system. Those investigations follow the execution of other dynamic tests, reported by Zonta 2000, in 1996. In that case the dynamic response of the wing was identified through Forced Vibration Tests (FVT), implementing stepped-sine tests (with a stationary harmonic excitation) and shock tests (with pulse hammer measurements) and fitting algorithm to determine the Frequency Response Function (FRF) of the system. From the analysis of the FRF modal parameters were extracted. In Tab. 5.1 it is reported a comparison between dynamic identification test performed through AVT in October 2011 and FVT in 1996. During the latter tests it was possible to detect also an in-plane structural mode of the wing at 6,1 Hz. Since AVT implemented only out-of-plane sensors this mode could not be identified, although its presence is confirmed by the analysis on a reference FE model.

Tab. 5.1 - Modal parameters identified by different types of dynamic tests (AVT vs. FVT) and different output-only OMA techniques (FDD vs. EFDD)

MODE	AVT - Oct 2011				FVT - 1996		AVT vs. FVT	
	FDD	EFDD		MAC			Average error [%]	
	f [Hz]	f [Hz]	ξ [%]		f [Hz]	ξ [%]	f	ξ
1	1,93	1,92	1,36	1	1,92	1,4	0	2,94
2	2,64	2,65	1,12	0,99	2,61	1,3	1,51	16,07
3	5,08	5,08	1,07	0,99	4,83	1,8	4,92	68,22
4	5,88	5,98	3,86	0,99	5,87	6,9	1,84	78,76
5	7,30	7,29	2,07	0,99	7,10	2,3	2,61	11,11
6	9,30	9,30	0,43	0,99	8,62	1,1	7,31	155,81
7	10,94	10,92	1,06	0,99	10,65	2,6	2,47	145,28

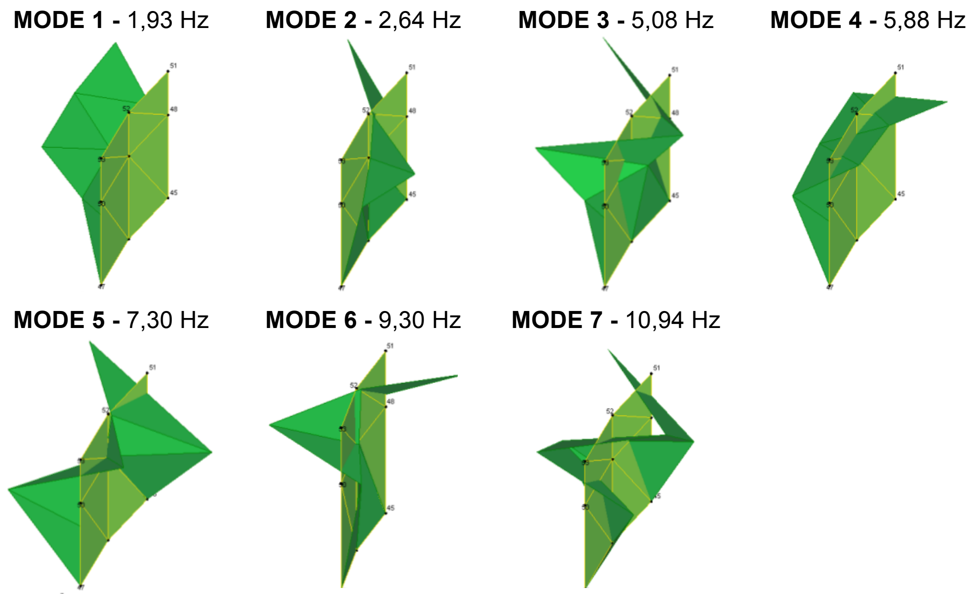


Fig. 5.9 - Natural frequencies and mode shapes of the wing identified through AVT and FDD method

The comparison between dynamic tests performed in two periods shows consistent results especially in the identification of the lower modes of the structure. Some discrepancies are recorded in the higher modes, characterized by higher average errors, induced probably by the selection of different dynamic identification methods and the difficulties, always present, in obtaining accurate identifications of higher modes.

Similar considerations can be made about the modal damping ratios, whose estimation presents always some uncertainties. As can be expected FVT are characterized by greater values of damping due to the higher level of excitation of the structure produced by this kind of tests compared with the use of low-amplitude ambient vibrations. If a structure is in fact excited by forced vibrations (such as in case of an earthquake) dissipation mechanisms can be better activated, resulting in higher values of damping. In any case both tests provide modal damping ratios significantly underestimated (a part from the 4th mode), considering that masonry structures are usually characterized by damping ratios in the range of 4-7%.

5.2.3.3 Layout of the system

Once defined the architecture of the monitoring system and sensors position the installation phase started.

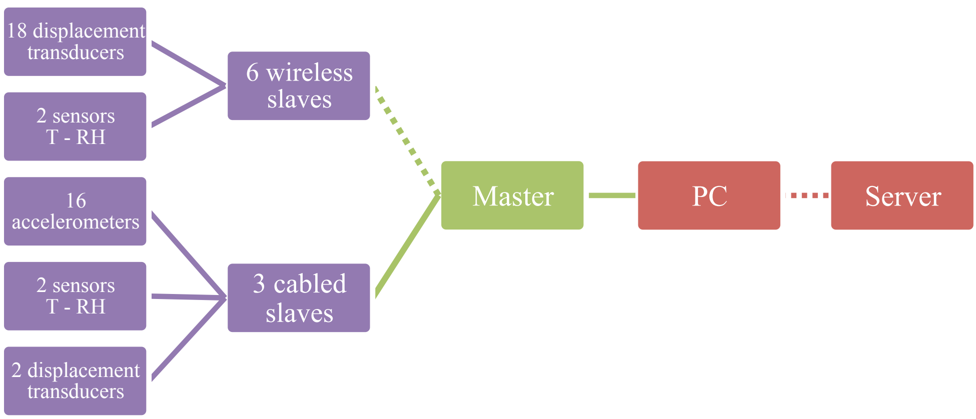


Fig. 5.10 - Scheme of the SHM system, a hybrid system composed by wired sensors and wireless nodes

The system is composed by sixteen single-axis accelerometers (acceleration transducers), twenty linear potentiometers (displacement transducer) and four integrated sensors of temperature and relative humidity. Given the huge dimensions of the structure it was decided to develop and implement a hybrid system: static sensors are connected through six wireless nodes placed around the inner gallery of the Arena to the principal acquisition unit and data are transmitted via a radio antenna, whereas the accelerometers are connected with three wired slaves that transmit signals to the master via ethernet cables (Fig. 5.10 and Fig. 5.11).

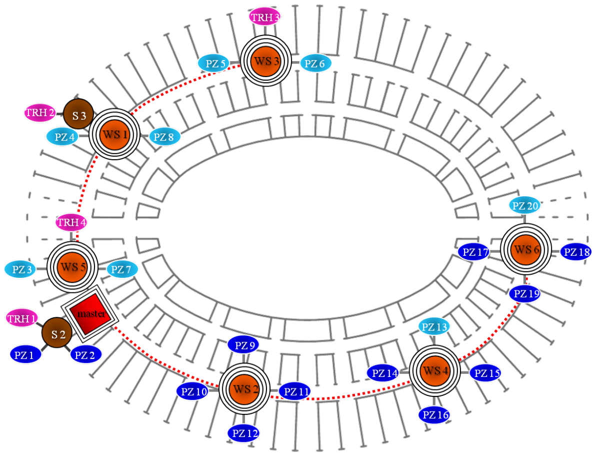


Fig. 5.11 - Wireless nodes of the monitoring system distributed along the inner gallery of the Arena

Displacement transducers are installed in correspondence of the main surveyed cracks and lesions and are connected to a wireless node, each with four channels, that send the signals to the master unit by means of a radio bridge (Fig. 5.12). The acquisition unit continuously communicates with the Wireless Sensor Network (WSN). Eight sensors are installed on the ground floor in the first inner gallery at a height of about 9 meters in correspondence to a big lesion that runs almost along the entire perimeter of the Arena's ellipse. Twelve transducers were positioned from outside in the vaulted niches (called "arcovoli") at the second order of arches of the monument: some of them on isolated cracks, some others on the surveyed detachment between the perimeter stone wall and the radial walls.

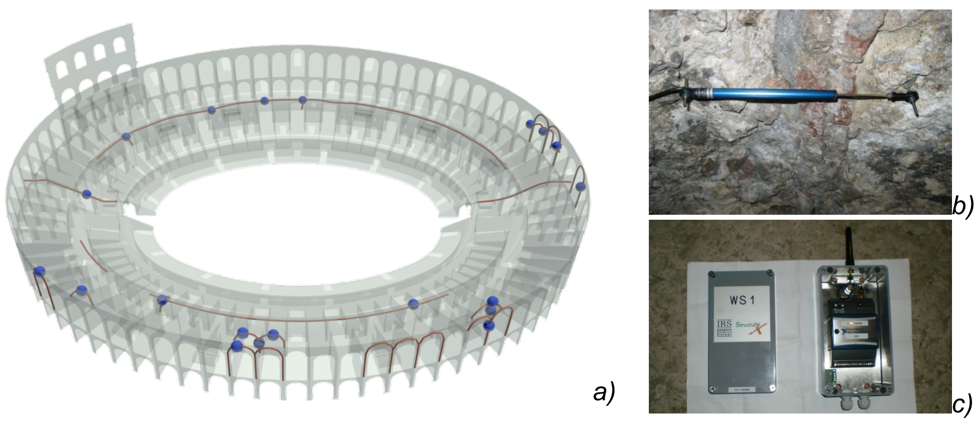


Fig. 5.12 - Static system: a) Layout and sensors positions; b) displacement transducer on a crack and c) wireless node.

The position of the sixteen single-axis piezoelectric accelerometers was decided according to the dynamic characteristics and vibration modes of the structure, evaluated through both numerical models and Operational Modal Analysis (OMA).

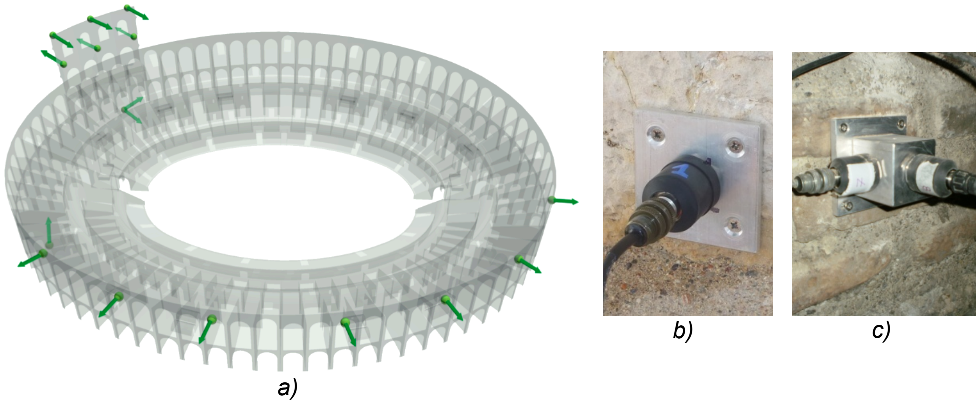


Fig. 5.13 - Layout of the dynamic system (left); accelerometers installed on the wing and at the base of the structure

Six accelerometers are installed on the wing of the Arena on two different levels along the out-of-plane direction. The wing is composed by a freestanding wall connected to the rest of the structure only at the first level and it is considered the most vulnerable structural element. Seven accelerometers were positioned on the top of the second order of arches along the elliptical perimeter of the structure, according to the radial configuration of the monument. Finally three sensors were placed at the base of the building along two orthogonal horizontal directions and a vertical one in order to record the ground acceleration in case of seismic events and evaluate the dynamic amplification of the structure (Fig. 5.13).

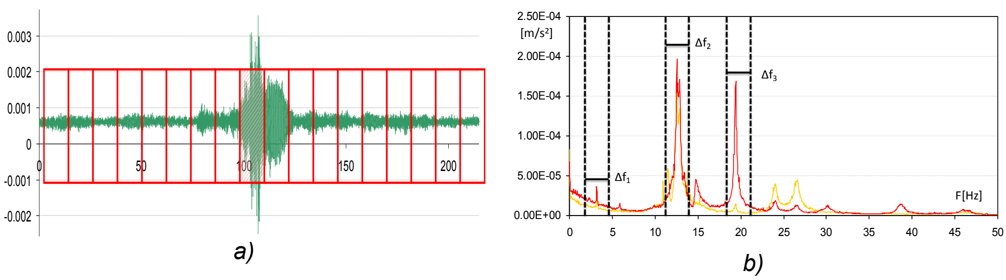


Fig. 5.14 - Monitoring strategy selected for the trigger-based dynamic system: if the signal exceeds a predefined threshold both in time (a) and frequency (b) domain, the system records and captures the event

With regard to the dynamic measurements, two strategies have been selected: "long" acquisition (corresponding to 131’072 points, or to 21’51’’ of record at a sampling frequency of 100 Hz, every 24 hours) on regular intervals to allow

successive dynamic identification of the structure with different environmental conditions (seasonal cycle), and "short" acquisition on a trigger basis (records 3’35’’ long at a sampling frequency of 100 Hz), when the signal, on one of the acceleration channels, exceeds the predefined threshold (meaningful event, e.g. earthquake) either in time or frequency domain (Fig. 5.14).
The system is equipped with a router for remote data transmission to the central server of the University of Padova.
The monitoring system allows the analysis of a huge amount of data taking into account three main aspects: (i) control of variations of the static measurements, (ii) daily extraction of the fundamental modal parameters; (iii) registration and analysis of possible seismic events.

5.3 Cansignorio Stone Tomb: SHM applied before, during and after interventions

The case study on the Cansignorio stone tomb in Verona gave the possibility to apply SHM on a special typology of CH structures, far different from traditional historical buildings due to its unique morphology and shape.
Another interesting aspect of the application of monitoring to this monument is the idea to control any structural changes during the execution of specific strengthening interventions, implemented to increase the seismic capacity of the structure.
Having active a SHM system before, during and after interventions allows in fact the application of the so called incremental approach, which means intervening following a step-by-step procedure, checking continuously the system response and modifying, if necessary, the intervention strategy according to the outcomes of monitoring.
This methodology was successfully applied to this case study. A monitoring system was installed after the execution of some preliminary investigations, at the very beginning of the knowledge phase. Then the system was kept active during the execution of interventions and for a relatively long period after their conclusion, trying to validate and confirm the effectiveness of the implemented strengthening solutions.
The case study and a detailed description of the monitoring system will be presented hereafter.

5.3.1 Historical notes and structural features

The Scaliger Tombs (in Italian: Arche scaligere) is a group of five Gothic funerary monuments in Verona, Italy, celebrating the “Scaligera” family, who ruled Verona from the 13th to the late 14th century.

The stone tomb of Cansignorio della Scala (Fig. 5.15) was built between 1374 and 1376, by the will of Cansignorio, when he was still alive. The tomb was erected next to the St. Maria Antica church, where the tombs of Cangrande and Mastino the 2nd (his grandfather and father respectively) were already built by local workers. Differing from his ancestors, Cansignorio desired a monumental tomb, where the architectural aspects were more important than the decorative ones. The work was then commissioned to Bonino da Campione, a famous master of gothic sculpture. The monument, based on a hexagonal plan, is adorned with sculptures and spired tabernacles, with the overhanging equestrian statue of Cansignorio (De Maffei 1955). Throughout the centuries, several repair interventions were necessary to preserve the delicate structure of the stone tomb, such as those carried out in the XVII, XIX and XX centuries. Between 1827 and 1829 other restoration works were carried out, raising arguments on the type of marble to be used in substitutions of the deteriorated parts. Other interventions, similar to those executed at the half of the XIX century, were carried out between 1910 and 1914. The monument was then protected against bombing during the two world wars (Mellini 1964).

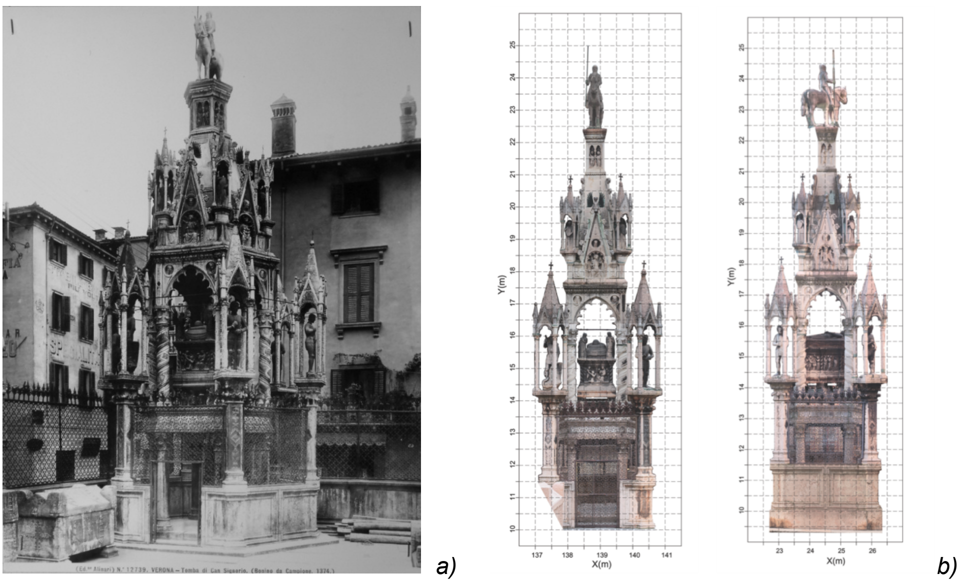


Fig. 5.15 - a) The stone tomb of Cansignorio, near St. Maria Antica church. b) c) Photogrammetric survey with two elevations of the tomb.

The tomb is surrounded by an hexagonal wrought iron fence, at whose corners rise six pillars sustaining gothic tabernacles, containing statues of the saint-warriors. The tomb starts with six columns sustaining a red marble slab on which the white marble sarcophagus is placed, sustained by eight pillars and decorated with bas-reliefs representing Gospel scenes. The cover of the sarcophagus presents a lying statue of Cansignorio, watched over by angels.

At the second level, six further spiral columns sustain the canopy with poly-lobed arches. Above these a cornice sustaining six gablets with allegorical figures representing the virtues is placed. At the corners there are six further tabernacles with statues of angels. The roof, corresponding to an hexagonal pyramid made of white marble, supports the massive equestrian statue of Cansignorio.

The stones used for the erection of the tomb are the “Candoglia” white marble, the same employed in the Milan’s cathedral, and the “Rosso di Verona” (Verona’s red marble), besides the Pietra Gallina (a soft limestone from Vicenza). The inner part of the roof (above the crossed vault and behind the stone facing of the canopy) is composed by solid brickwork masonry.

5.3.2 Strengthening intervention (2006-2008)

As previously introduced, between 2006-2008, a light strengthening intervention was designed and performed to improve the structural and seismic capacity of the monument, stabilize some critical points and intervene in deteriorated parts or elements, integrating the original materials (Gaudini *et. al* 2008). Technologies and strengthening solutions implemented can be divided into two categories: local and global level. At the global level of structural members the proposed interventions (see interventions A, B, C and D of Fig. 5.16a) aimed at the enhancement and upgrading of the existing hooping and confinement system by means of the insertion of high strength steel cables in addition to the existing iron tie rods.

At a local level two main interventions were performed:

- Application of CFRP strips (Fig. 5.16c) to repair the deteriorated parts of the equestrian statue: this reinforcement is made of carbon fibres impregnated in a polymeric matrix. Due to high strength and stiffness-to-weight ratio, fatigue and corrosion resistance, and their exceptional capacity to be tailored to specific needs, along with decreasing raw materials and production costs, advanced composites have progressively extended their initial fields of applicability even to non-structural application like in this case.
- Confinement of a cracked capital (Fig. 5.16d) by means of the application of a hooping system composed by high-strength steel cables (Ø 1.6 mm)

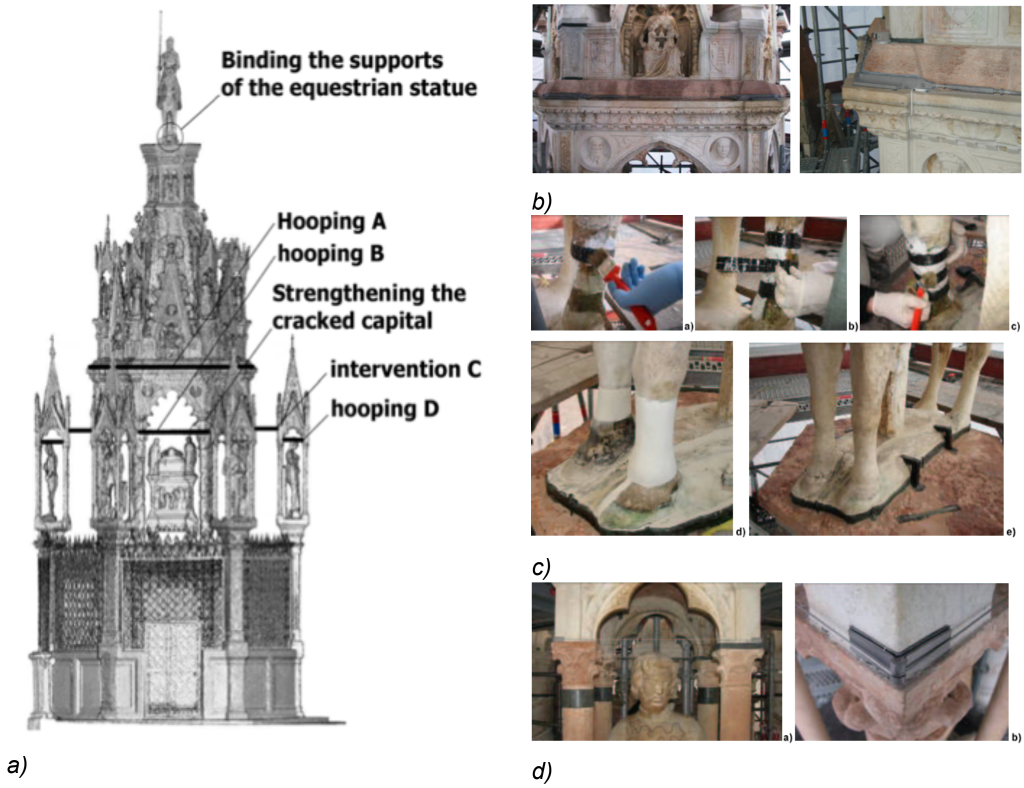


Fig. 5.16 - Strengthening interventions of the stone tomb (2006-2008): a) Summary of global interventions; b) Positioning of the confining system composed by high-strength steel cables; c) Application of CRFP strips at the base of the equestrian statue; d) Local confinements of the columns' capitals.

5.3.3 The monitoring system

5.3.3.1 Preliminary investigations: OMA

After the execution of endoscopies and tests on materials to characterize their physical and chemical properties, deep structural inspections were carried out by means of Ambient Vibration Tests (AVT) (Marotto 2008). The investigation campaign were aimed at the definition of the optimal layout of the SHM system and at the characterization of the dynamic properties of the monument for FE modeling calibration purposes. A compact unit provided with 24-bit digital acquisition cards, connected to piezoelectric single axis acceleration transducers, composes the acquisition system. Once fixed the transducers to the structure in the selected positions, tests consisted in acquiring data over a predetermined period, at a specific sample rate, implementing three different setups (Fig. 5.17).

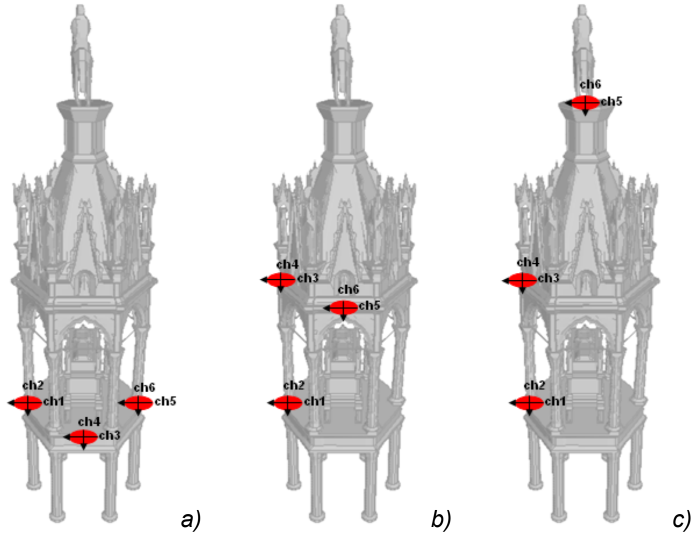


Fig. 5.17 - Setup considered in the AVT: a) Setup 1; b) Setup 2; c) Setup 3

Each test setup consisted in recording the signal two times (65'536 points each) whit a sampling frequency of 100 SPS (samples per second), with an overall setup signal recording duration of 21'51''. Acquired data series were pre-processed by a decimation of 2 (Nyquist frequency of 25 Hz), with segment length of 2048 points and 66.67% window overlap. For the identification of the modal parameters (natural frequencies and corresponding mode shapes), output only identification techniques were used.

The modal parameter extraction method selected was the FDD technique which estimates the modes, with the assumption that the excitation is reasonably random in time and in the physical space of the structure, using a Singular Value Decomposition (SVD) of each of the spectral density matrices (see §3.3.2 for more details).

Tab. 5.2 - Estimated modal parameters of the stone tomb

MODE	**FDD**	Comment
	f [Hz]	
1	3,17	1st bending NS
2	3,22	1st bending EO
3	5,91	1st torsional
4	12,60	2nd bending NS
5	12,89	2nd bending EO
6	19,43	2nd torsional

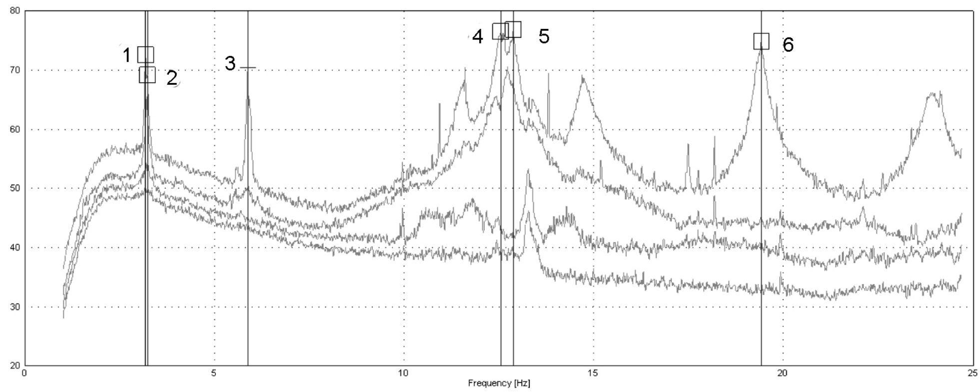


Fig. 5.18 - FDD technique: average of the normalized singular values of the spectral density matrix: peaks indicate the structural frequencies of the stone tomb (Marotto 2008)

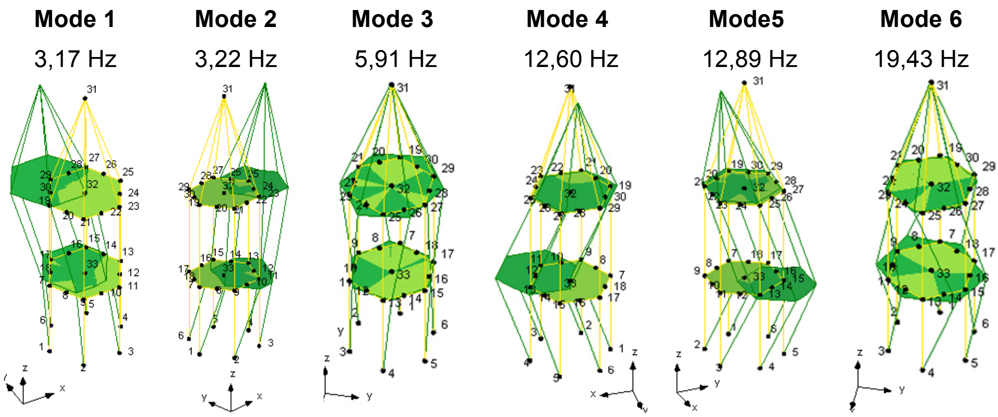


Fig. 5.19 - Mode shapes identified through the FDD method

5.3.3.2 Layout of the system

The preliminary investigation campaign and particularly the identification of the dynamic properties of the monument gave the possibility to design the architecture of the SHM system, selecting the optimal sensors' position and most suitable monitoring strategy.

The system (installed in December 2006), is aimed at the control of static and dynamic parameters related to the structural functioning of the monument. Since it was installed before the execution of the interventions, it allows evaluating on site the effectiveness of the adopted strengthening solutions.

It is composed by (Fig. 5.20):

- 1 acquisition unit
- 6 high sensitive single-axis piezoelectric accelerometers

- 2 potentiometric displacement transducers
- 1 temperature and relative humidity sensor

The central unit, located at the base of the tomb, is provided with a Wi-Fi router for remote data transmission.

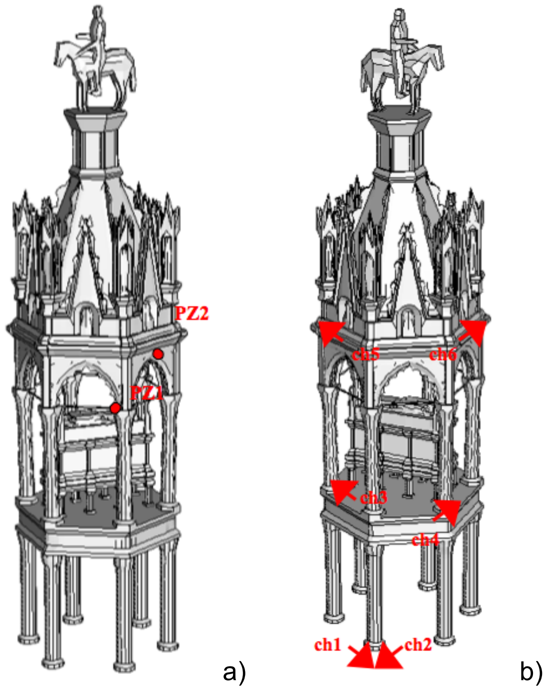


Fig. 5.20 - Layout of the SHM system: a) static transducers installed on two representative cracks and b) six single-axis accelerometers positioned on three levels of the monument.

The monitoring strategy is conceived both to collect data at predetermined time-intervals (periodic monitoring, i.e. cracks opening, changes in the dynamic response) and to automatically start to save data in case of significant external events (such as seismic events). Such controls will permit to appreciate possible variations in the assessed structural functioning with the passing of time and to have a record of the dynamic behavior of the stone tomb during severe events.
The acceleration transducers are placed in suitable positions in relation to the mode shapes of the structure, as identified by OMA. Four sensors are placed on two levels for the evaluation of the vibration in the NS and EW direction (bending modes) and in the horizontal planes (torsion modes).
A couple of reference sensors is fixed at the base to record the ground acceleration both in operational conditions (i.e. evaluation of the traffic induced vibrations) and during seismic events. A temperature/relative humidity sensor is fixed at the intrados of the marble slab (first level). The displacement transducers are positioned across significant cracks. The temperature, relative humidity and

displacement of the selected points (crack opening) are recorded each 6 hours, corresponding to 4 daily readings. Dynamic data are collected both at fixed time intervals (each 48 hours, approximately 22' of recording at a sample rate of 100 Hz) and on a trigger basis (shorter records, signals are recorded when the vibration exceeds a predefined threshold).

5.4 L'Aquila case studies: SHM for post-earthquake controls

The earthquake occurred on the 6th of April 2009 in the Abruzzo Region of Italy seriously hit the Cultural Heritage patrimony with major destructive effects on l'Aquila, a city of 70,000 inhabitants with the size and the historical and strategic importance of the capital of the Region. The severity and the extent of damages caused by the earthquake to historical buildings and monument was never reached before in the recent Italian earthquakes history. The emergency activities to protect the CH structures have been developed following two parallel levels: (i) damage survey and (ii) design and implementation of temporary safety measures.

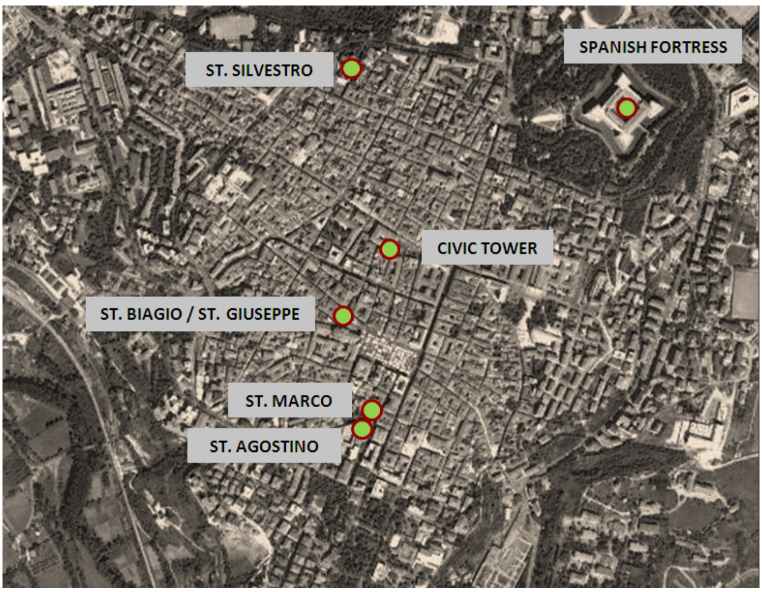


Fig. 5.21 - l'Aquila SHM network (University of Padova & Nagoya City University)

Some major monuments were soon equipped with SHM systems in order to control the progression or stationariness of the damage pattern already assessed. In a second step, further systems were applied on other CH buildings, also during the stabilization works execution, in order to appraise the effectiveness of the interventions carried out, or to denounce their inadequacy. After more almost four years, with the "heavy" reconstruction process at its beginning, a small network of SHM systems (Fig. 5.21) was set up by the Dept. of Civil, Architectural and Environmental Engineering and the Nagoya City University, whose data – also containing several aftershocks records – will constitute a sound database for investigation in the field of CH structures response and strengthening procedures. In the following paragraphs, SHM systems installed on two emblematic structures are briefly presented.

5.5 Civic Tower

The Civic Tower is an emblematic historic structure, part of 'Palazzo Margherita', the seat of L'Aquila City Hall (Fig. 5.22). The structure was severely damaged by the strong earthquake occurred on the 6th of April 2009. The tower (6.5*m* x 7*m* base and 43*m* high) is a structural element particularly vulnerable due to its tall and slender shape and the scarce mechanical characteristics of the constitutive masonry walls. The earthquake induced a diffused and serious crack pattern along the entire shaft of the tower with damage concentration at its basement.

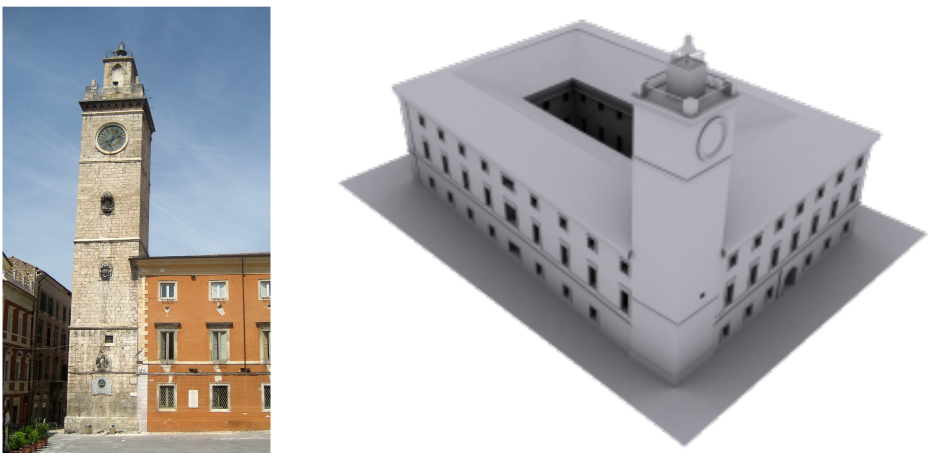


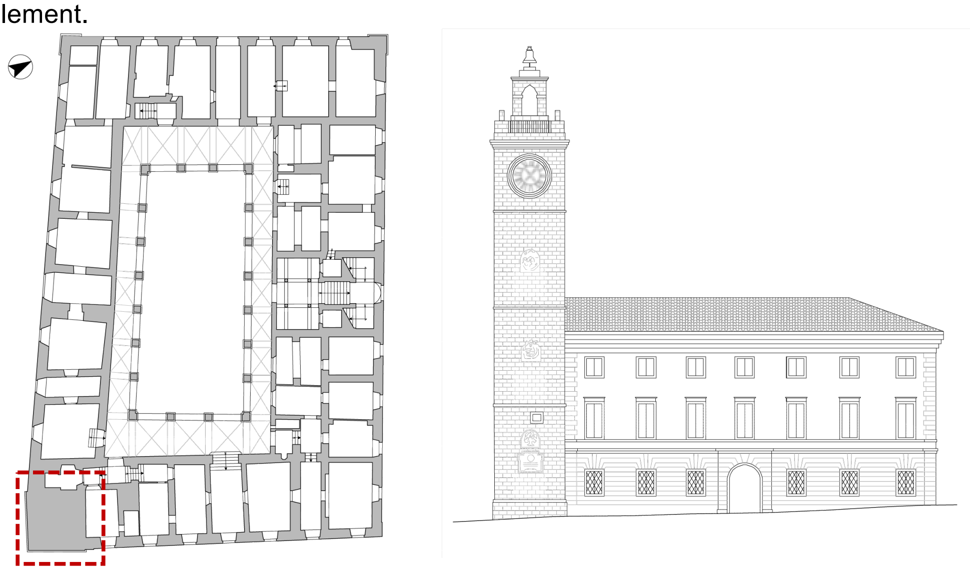
Fig. 5.22 - Civic tower and Margherita Palace: picture and 3d rendered model

The precarious structural conditions of the building suggested implementing some temporary emergency interventions in combination with the installation of a permanent SHM system, able to evaluate quantitatively the progression of the assessed damage pattern.

Ambient vibration tests were preliminarily performed to characterize the dynamic response of the structure and choose the optimal positions for the dynamic monitoring layout. Subsequently a static and dynamic monitoring system was installed, including devices to measure set of displacements and strains at critical points of the structures (displacement transducers, strain gauges, inclinometer, temperature and humidity sensors) and to record ambient vibrations (accelerometers).

5.5.1 Historical notes

The historic complex of the l'Aquila City Hall is composed by two bodies, characterized by a different historical-constructive evolution: the Civic Tower and the Margherita Palace (Fig. 5.23). The first one was constructed before the foundation of the city in 1254, and originally conceived as an isolated structural element.



a) *b)*

Fig. 5.23 - a) Plan view with the position of the tower (dashed rectangle); b) East elevation of the Civic tower-Margherita Palace historic complex

The civic tower, at the time of its construction, was 70 m tall, but in 1703 it was lowered after the earthquake and its height reduced again in 1838 because of the presence of diffused cracks and the collapse of the upper part of the structure. Nowadays the tower is 42 m high. The construction of the Margherita Palace began in 1294 and the building underwent several restorations and reconstructions over centuries. The most important intervention dates back to 1572 when Margherita of Austria, governor of the city, chose it for her residence. Then, three centuries later

(1836-46), another restoration changed drastically the spatial configuration of the building that appeared more modest and in some way more coherent with his new function as location of the Judicial District (Dander & Moretti 1974).

Palace and tower experienced several strong earthquakes over its long history: the most catastrophic and destructive happened in 1349, 1461 and in 1703. After each seismic event the buildings were strongly damaged and subsequent restoration works have been required to consolidate their structural members.

A renewal of the architectural aspects of Palazzo Margherita took place in the first half of XIX century (1838-48) causing a series of transformations also in its stylistic language (Centofanti 1979).

5.5.2 Earthquake-induced damages

The main damage mechanisms induced by the earthquake to the palace and the tower detected during visual inspection activities are briefly reported hereafter (Fig. 5.24).

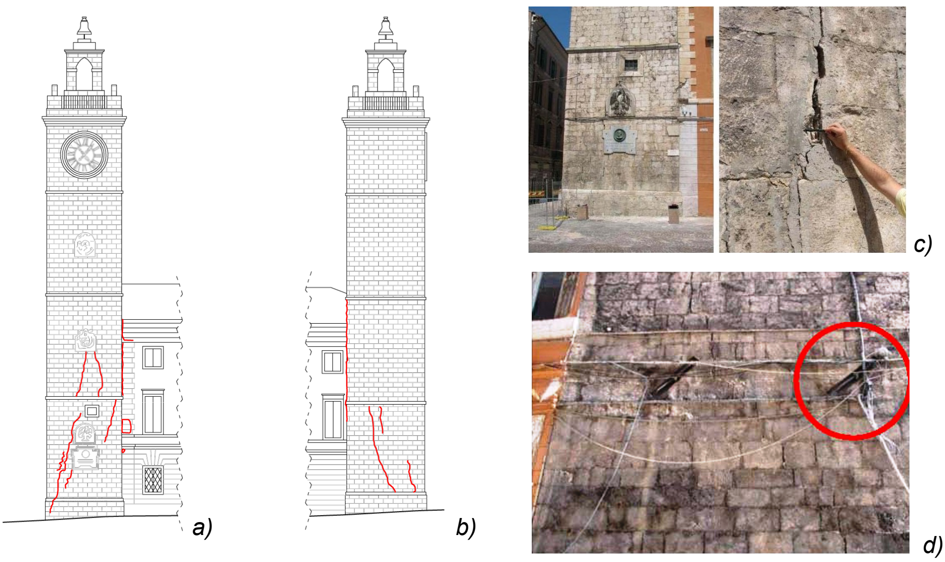


Fig. 5.24 - Crack pattern survey after the earthquake on the East (a) and South (b) elevations. c) Sub-vertical cracks at the tower's basement. d) Break of the anchor plate of a metal tie (Buffarini et al. 2009).

Masonry walls of the palace were subjected to both out-of-plane overturning mechanisms caused by seismic forces applied perpendicularly to the walls and shear mechanism caused by seismic forces applied parallel to the walls which develop the typical diagonal cracks. Other localized damages were recorded, such as damages to the roof, shear cracks in the vaults and in the stairs, complete or partial collapses of the floor slabs at the second floor.

The Tower was heavily damaged due to its tall and slender shape that made it more vulnerable to base settlements and movements induced by earthquake forces. In addition, the Civic Tower slenderness and cantilever beam-type boundary conditions has made it unsuitable for redistributing stresses and dissipating energy. Therefore it has been accompanied by a concentration of stresses at the basement that has been amplified by the brittleness of deteriorated masonry. The detailed visual inspection and crack survey showed the presence of diffused and severe crack patterns on the outer surface that also appear in the inner walls. Cracks have been observed at the base of the tower, on the South and East side. Some of those cracks were already there after 1703 earthquake and they reopened, while new cracks occurred with 2009 earthquake. The new cracks could be detected by the visual inspection of the plaster inside the tower that in some parts reach amplitudes up to 2 cm (Buffarini *et al.* 2009).

5.5.3 Emergency provisional interventions

Provisional interventions designed and implemented during the emergency phase after the earthquake on the Civic Tower are detailed hereafter. The main principle is to design some stabilization measures in order to prevent further failure and collapses. Static and dynamic conditions of the monument are constantly controlled through the monitoring system, which allows the application of the so called incremental approach, limiting the interventions (even if provisional) to a strict minimum and stabilizing the timing for the implementation of definitive interventions.

The main technologies applied to this case studies concern the implementation of stabilization measures and in particular: propping system of structural elements (wooden frames and steel scaffoldings), application of metal ties.

The main stabilization measures (Fig. 5.25) design and implemented in the building are listed hereafter:

- Positioning of a confining system of the tower composed by steel beams, ties and wooden frames at different heights along the shaft in order to stabilize the structural element;
- Improvement of the connection of the tower with the palace;
- Propping system of the wings of the palace by means of steel beams, plates and ties in order to avoid out-of-plane overturning mechanisms and collapses.

The proposed provisional interventions are considered as a first operation, that has to be necessarily complemented, in the future, with a definitive strengthening intervention, including repairs, strengthening and seismic improvement.
In this sense the application of SHM in combination with an incremental approach allow controlling the effectiveness of the implemented provisional measures and postpone the execution of definitive interventions in agreement with the economic available resources and the reconstruction plans of the city of l'Aquila.

Fig. 5.25 - Provisional interventions on the Civic Tower: a)confinement of the tower; b) and c) Propping system of the wings of Margherita palace

5.5.4 The monitoring system

5.5.4.1 Needs for monitoring

The precarious structural conditions of the building suggested implementing some temporary emergency interventions in combination with the installation of a permanent structural health monitoring system (SHM). The system gives interesting information on the progression or stability of the assessed damage pattern, with reference to the already carried out stabilization actions and the foreseen strengthening interventions. These readings are constantly related to environmental parameters (temperature and relative humidity).

A parallel aim is found in the proposal and exploration of strategies for an efficient monitoring of a historical structure in a post seismic scenario. The proposed monitoring system aims at providing relevant information with a limited number or critically located, technically advanced measurement devices. Both static and dynamic monitoring are proposed to obtain valuable information to:

- Evaluate quantitatively the progression of the damage pattern;
- Design effective and urgent interventions if an unsafe displacement patterns is recorded;
- Define an early warning procedure for the safety of the workers employed in the strengthening interventions;
- Evaluate the effectiveness of the implemented provisional interventions immediately after the earthquake.

5.5.4.2 Preliminary investigations: OMA

Before the installation of the Structural Health Monitoring (SHM) system, a dynamic investigation campaign took place in July 2010. The dynamic behavior was evaluated in the damaged conditions of the tower after the earthquake. The main aim was in fact to identify the global dynamic response of the structure after the provisional strengthening interventions executed by firemen during the emergency phase. Another important objective of dynamic tests is the definition of the optimal layout of the SHM system sensors and the characterization of the dynamic properties for FE modeling calibration purposes.

Dynamic identification tests were executed on the tower and the two wings of the palace (Eastern and Southern facades) directly connected with the tower in order to evaluate the dynamic behavior of the whole structural complex and appraise if the tower-palace system had still an unitary dynamic response even after the high level of damage and disconnection induced by the earthquake. In fact as it can be noted from the damage survey huge cracks arise along the connection between the tower and the palace.

It was decided to use 32 single-axial acceleration transducers. Once fixed the transducers to the structure in the selected positions, tests consisted in acquiring data in 3 different registrations over a predetermined period, at a specific sampling rate. A typical acquisition consisted in a record length of 144’000 samples, resulting in an acquisition time of approximately 30 minutes at a sample rate of 80 SPS (Samples Per Second). For the identification of the modal parameters (natural frequencies and corresponding mode shapes), output only identification techniques were used. In particular, the recorded ambient vibrations were related to the wind excitation and urban traffic, The signal-processing phase consisted in the elaboration of the measured data using dedicated software for OMA: SVS ARTeMIS Extractor 4.0, 2007 and MACEC 3.2, 2011. It was decided to use three frequency-domain modal parameter extraction techniques: FDD - Frequency Domain Decomposition, EFDD - Enhanced Frequency Domain Decomposition (see § 3.3.2) and pLSCF - poly-reference Least Squares Complex Frequency-domain (see § 3.3.3).

Data series acquired at 80 SPS are processed, using FDD, EFDD and pLSCF methods. Peaks in the frequency domain related to structural frequencies were selected and the corresponding mode shapes defined. During the extraction of modal parameters it was noted that the first two orthogonal bending modes have very closely spaced frequencies (in the range of 0.02 Hz) around 1.5 Hz. Implementing various identification techniques and in particular the p-LSCF, it was possible to separate clearly these coupled modes (Fig. 5.26). This phenomenon is pretty common for symmetric structures like the civic tower. It is interesting to notice that also the second order bending modes are rather closely spaced (in a higher range of 0.4 Hz between 3.3 and 3.7 Hz).

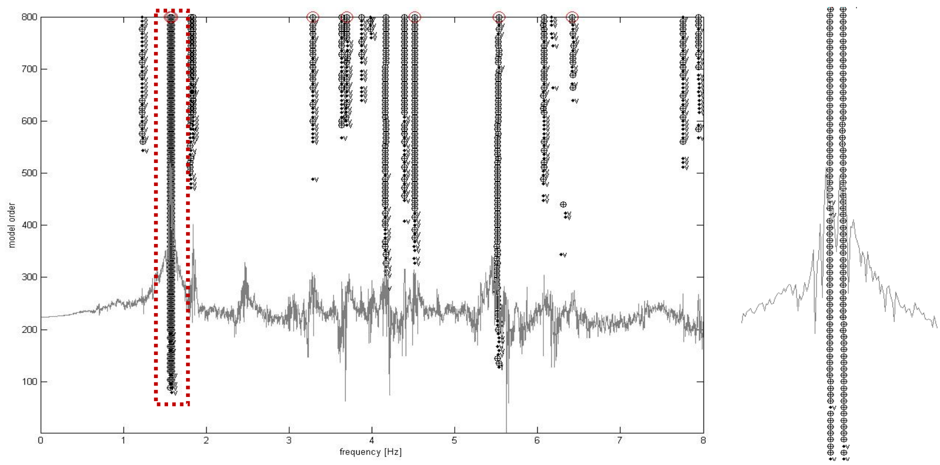


Fig. 5.26 - p-LSCF method: a) stabilization diagram (left). b) closely spaced bending modes around 1.5Hz

Tab. 5.3 summarizes the seven estimated modal parameters through three frequency-domain output-only techniques, in terms of resonant frequencies, damping coefficients and Modal Assurance Criterion (MAC) between EFDD and p-LSCF techniques. Regarding natural frequencies, the values range from 1,5 to 6,4 Hz and no significant differences could be found between the three methods. The differences on modal damping estimation are much more substantial and, on average, are equal to 56%. Observing the MAC values calculated between mode shapes extracted from EFFD and p-LSCF methods, it is possible to state that the first 2 bending modes and the torsional mode are highly correlated (values higher than 0,87). For the higher modes (3rd bending modes) the MAC index decreases to a minimum of 0,76, meaning that the estimation of higher modes is more difficult with ambient vibration measurements.

Tab. 5.3 - Modal parameters estimation of the tower implementing and comparing various identification techniques (frequency ω, damping ratio ξ and MAC value calculated between EFDD and p-LSCF methods)

MODE	FDD	EFDD		p-LSCF		MAC	Comment
	f [Hz]	f [Hz]	ξ [%]	f [Hz]	ξ [%]		
1	1,563	1,563	0,984	1,5637	0,4242	0,99	1st bend. EO
2	1,582	1,583	0,907	1,5824	0,4323	0,98	1st bend. NS
3	3,301	3,292	1,352	3,2883	0,6054	0,89	2nd bend. NS
4	3,711	3,699	1,143	3,6999	0,7028	0,87	2nd bend. EO
5	4,512	4,521	0,877	4,5151	0,6975	0,92	1st torsion
6	5,527	5,457	0,581	5,5336	0,7718	0,78	3rd bend. NS
7	6,445	6,434	1,229	6,4196	0,4620	0,76	3rd bend. EO

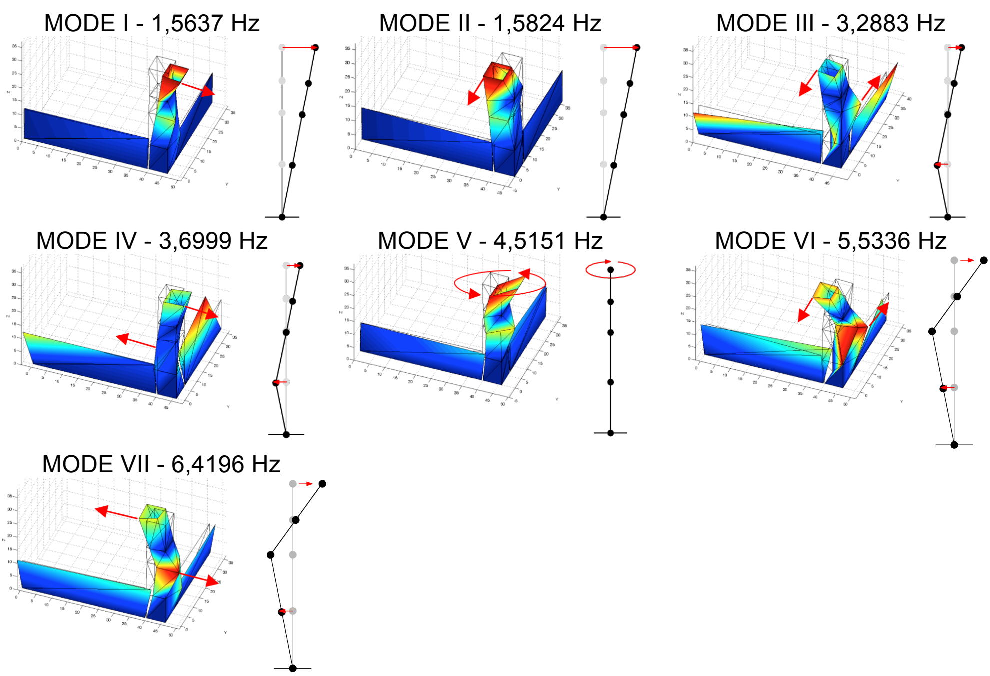


Fig. 5.27 - Mode shapes identified for the first seven natural frequencies of the tower

5.5.4.3 Layout of the system

The monitoring system installed in the civic tower is composed by: (i) static sensors to control the damage and crack pattern of the structure and (ii) accelerometers to measure ambient vibrations and capture possible aftershocks and seismic events.

The static system (Fig. 5.28) includes devices to measure a set of displacements and strains at critical points of the building. It is composed by:

- 3 displacements transducers (PZ1 to PZ3) installed on representative cracks in the lower part of the tower to control the crack width;
- 2 displacements transducers (PZ4 and PZ5) installed on the crack between tower and palace to control the relative displacements of the two structures;
- 1 inclinometer to control the displacement of the tower's top in two in-plane orthogonal directions (IN1 and IN2);
- strain gauges (ST1 TO ST6) on the existing metal ties of the tower to control the strain variation;
- 6 thermocouples (T1 to T6) to control both air and walls temperatures in different points of the structure.

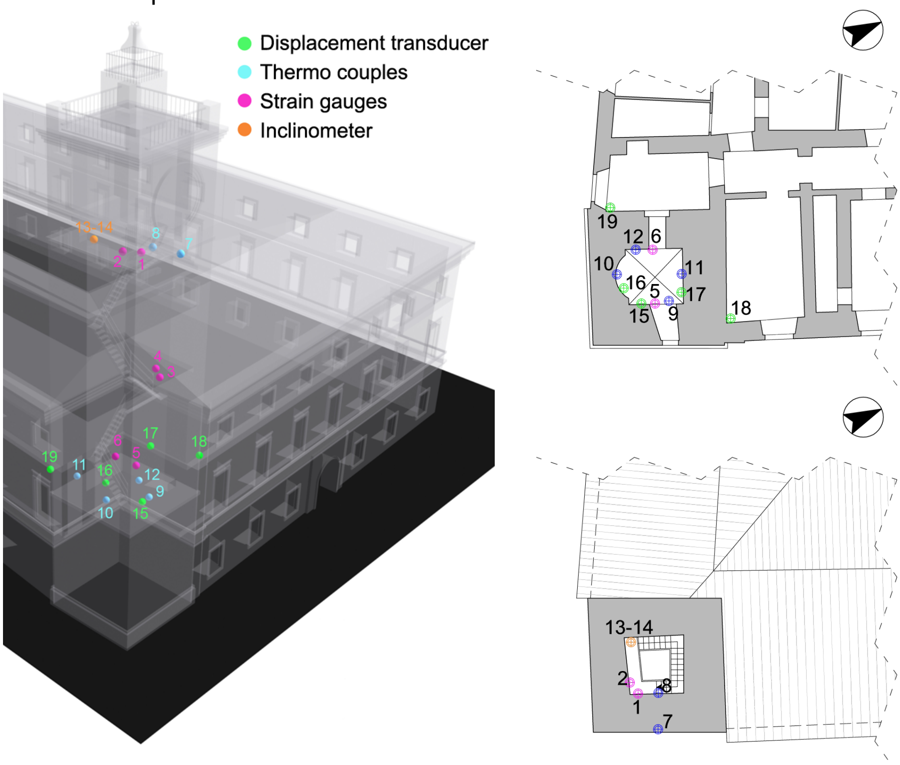


Fig. 5.28 - Layout of the static system composed by 5 displacement transducers, 1 inclinometer, 5 strain gauges and 6 thermal sensors

Data from the static system are registered every 30 minutes.
The dynamic monitoring system (Fig. 5.29) is composed by 8 high sensitivity piezoelectric accelerometers connected to an acquisition unit. Three reference sensors are fixed at the base of the structure to record the ground acceleration both in operational conditions and during seismic events. The positioning of the other acceleration sensors on tower's elevation was decided on the results of the dynamic identification. Unlike the dynamic monitoring system previously described, this system is not trigger-based but continuous and high-density (80 samples per second) dynamic information is continuously recorded.

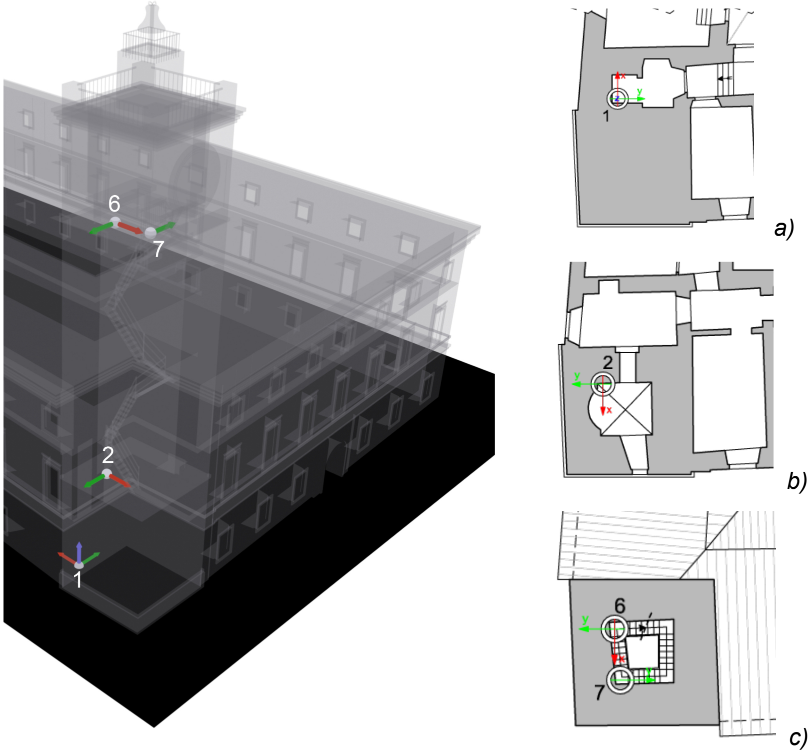


Fig. 5.29 - Layout of the dynamic system composed by 8 high sensitivity piezoelectric accelerometers: 3 installed at the ground level (a), 2 at the level of the second floor of Margherita palace (b) and 3 at the top (c)

The continuous dynamic monitoring has several connected purposes: (i) characterize the dynamic response from ambient vibrations along with its dependence with environmental parameters; (ii) capture the dynamic response in case of possible seismic events; (iii) calibrate reference Finite Element models based on the daily extraction of modal parameters from recorded vibrations.
The system is equipped with a router for remote data transmission. Given its significance, particular attention is given to the monitoring of the variation of

temperature in the building. Temperature influences in fact largely both the static and dynamic monitoring output, and a good characterization of its variation and distribution within the structure is necessary for a correct interpretation and post-processing of both results.

5.6 Spanish Fortress

5.6.1 Historical notes

The Spanish Fortress of L'Aquila (Fig. 5.30) is one of the most impressive Renaissance castles in Central and Southern Italy.

Fig. 5.30- Aerial view of the Spanish fortress of L'Aquila, before the earthquake

In the 15th century L'Aquila became the second most powerful city in the Kingdom of Naples, under the Spanish domination. In 1528 Viceroy Filiberto d'Orange ordered to build a fortress in the highest northern spot of the city, according to the project of a famous Spanish architect, Don Pirro Aloisio Escrivà. The construction started in 1534; Escrivà designed a giant fortress, composed by four bastions connected through heavy walls, 60 meters long, with a thickness of 30 m at the bottom and 5 m at the top (Fig. 5.31). All around the fortress there was a ditch (never filled with water) 23 meters wide and 14 meters deep, aimed at defending the foundations from the enemy artillery. The Fortress, which was built not to defend the city, but to control it (many cannons pointed to the city) and to be a completely self-sufficient structure, was never used in battles. Its cannons, always ready to fire, were silent throughout the centuries: the only victim was the city itself, whose decline began with the construction of the fortress and went on under the

Spanish domination. Between 1949 and 1951 the castle was restored, and chosen as the seat of the National Museum of Abruzzo (Congeduti 1988).

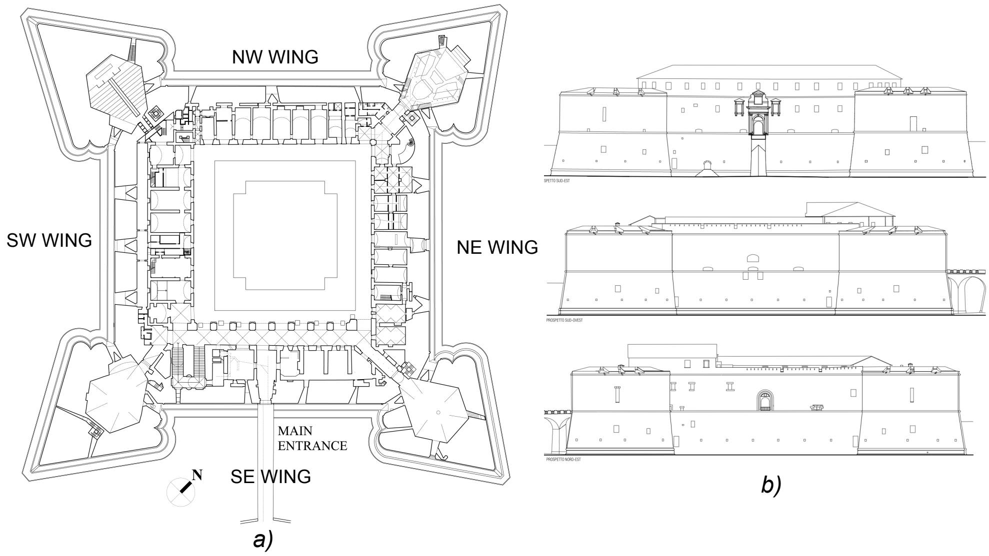


Fig. 5.31 - Geometric survey of the building: a) plan view and b) elevations

5.6.2 Earthquake-induced damages

The Spanish fortress was seriously damaged by the earthquake of the 6th of April 2009. The most relevant damages and collapses involved especially the upper floors of the Fortress.

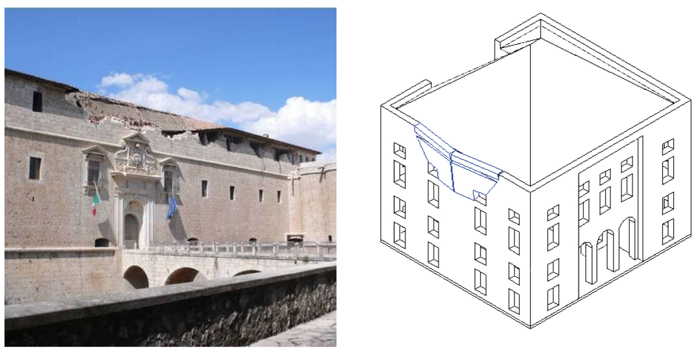


Fig. 5.32 - Out-of-plane overturning of the upper part of the main façade (South-East side)

According to the damage survey template for palaces used in the technical inspections (Model B-DP PCM-DPC MiBAC 2006), overturning and flexural mechanisms on the external walls, shear damage in the external and internal walls, damages to vaults and arches, local collapses of floors and vaults, correspond to the most worrying observations. Damages were remarkable both for intensity and distribution, and were considered so serious to be likely menacing the overall stability of some large parts of the building.

The South-East wing (Fig. 5.32) of the fortress underwent the most significant structural problems. On the external front it can be noted the overturning of the upper masonry walls; several collapses took place at the second floor together with a considerable separation of the floors from the longitudinal walls.

In the internal front of the same side the pillars of the porch arcade show crushing failure mechanisms. The seismic event damaged the walls of the first floor and produced a longitudinal crack in the barrel vault of the arcade. The crack is also visible on the floor surface above, meaning that it interests the entire thickness of the floor. There are also many shear cracks on the transverse bearing walls (Fig. 5.33).

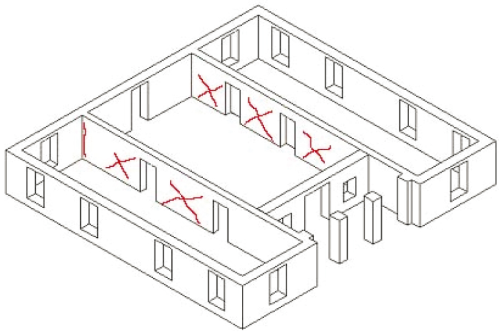


Fig. 5.33: Severe shear cracks and collapses in the internal walls of the South-East wing

In the South-West wing it can be noted an overturning mechanism of the two facades, cracks on vaults and shear cracks on the transverse walls. Moreover, on the first and second floor the separation between external perimeter walls and internal walls and floors are clearly evident.

The two facade of the North-West part showed a greater resistance to the overturning mechanism; there are no large detachments of the floors from the perimeter walls. In fact in this area of the fortress a system of tie rods connecting the perimeter walls had been inserted before the earthquake. This intervention was effective and avoided collapses and irreversible damages to the structure. However, the transverse walls are seriously damaged and on the second floor some parts of masonry walls and floor slabs collapsed.

The North-East wing is affected by a slight overturning mechanism of the two facades. There are some shear cracks on masonry walls but the most evident damages are localized on the upper floors.

5.6.3 Emergency provisional interventions

The design of temporary interventions for the safety of an historical building starts from the damage survey and from the identification of the collapse mechanisms activated from the seismic action.

According to the observed damage pattern, provisional strengthening interventions were applied to the South-East and South-West wings of the fortress, where the heaviest damages were observed. The strengthening interventions (Fig. 5.34) were carried out by different teams of specialized people from the National Fire Brigades. The interventions did not rely upon external propping structures, also considered the massive dimensions of the fortress. The structural stability was provided by relying on the remaining strength of the resisting elements, e.g. by connecting the internal and external façades of the damages wings by means of stainless steel cables, in order to avoid the observed overturning mechanisms evolution, especially taking into account that non negligible aftershocks occurred for several months.

In the South-East wing it was necessary to rebuild the roof, by using hollow section steel trusses and a light covering structure made of wood. The substitution of the original wooden structure of the roof with a heavy and stiff reinforced concrete structure, without any strengthening interventions on the underlying masonry walls, caused the collapse of the upper part of the façade. In the South-West wing steel frames were positioned in contrast to the external and internal façades before tensioning the cables.

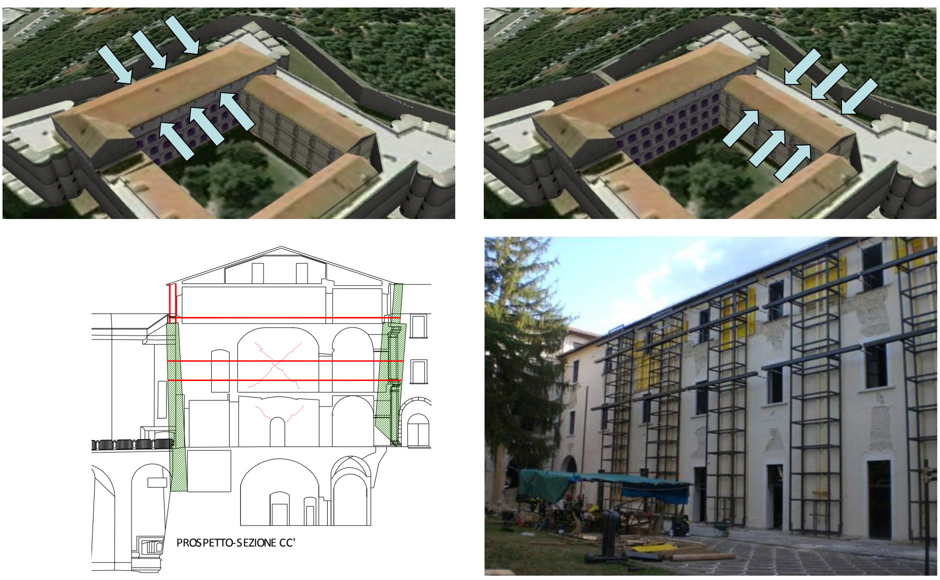


Fig. 5.34 - Provisional interventions carried out on SE and SW wings: connection of the internal and external façades by means of steel cables and steel frames against the facades

5.6.4 The monitoring system

5.6.4.1 Preliminary investigations

A series of non-destructive (NDT) and minor destructive (MDT) testing were performed in the Spanish Fortress by the research groups of the University of Padova and of the Politecnico of Milan in order to characterize and evaluate both quantitatively and qualitatively the state of damage of masonry structures (walls and pillars) and to identify the structural response of the most damaged wings of the palace (especially the South-East one). Inspections performed on the monument represent a fundamental phase both to increase the knowledge level of its structural behavior for a and to define the most vulnerable parts to keep under control with the aid of monitoring.

The experimental campaign included:

- Sonic pulse velocity tests
- Radar tests
- Thermographic tests
- Single and double flat jack tests
- Dynamic identification tests

Fig. 5.35 shows the investigated walls and pillars and the position of on-site tests at the first floor of the castle.

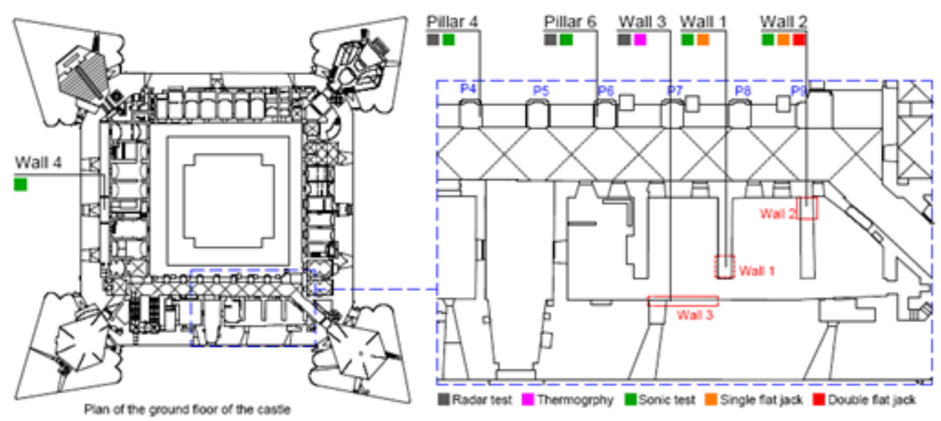
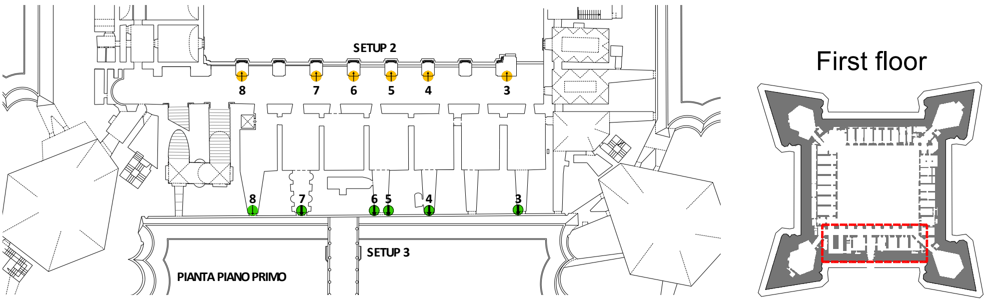


Fig. 5.35 Plan of the Spanish fortress: position of the tests at the ground floor of the SE wing

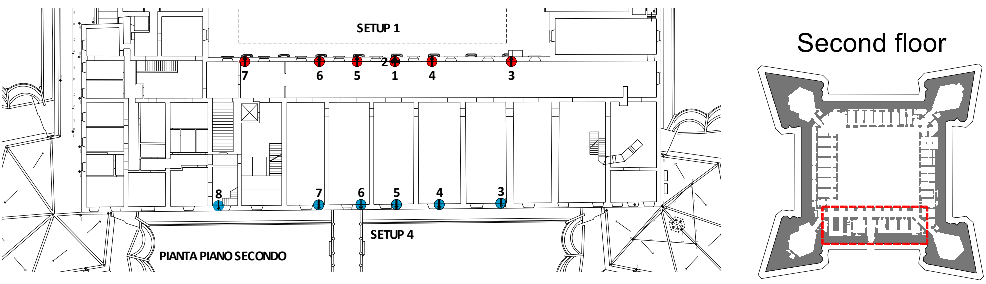
In the present work only the results of Ambient Vibration Tests, performed on the South-East wing of the fortress, will be briefly described and analyzed. The outcomes of dynamic inspections are then exploited for the design of the monitoring system, including the selection of the optimal positions of acceleration transducers, according to the mode shapes experimentally identified and extracted.

A parallel aim of OMA was the evaluation of the dynamic behavior of the main and most damaged wing of the castle, after the overturning mechanisms activated by the earthquake. From a structural point of view it was important to know if the two longitudinal walls (also thanks to the provisional tie-rods system) showed still a unitary dynamic behavior after the seismic event.

Five configuration setups were performed with 27 points of acquisition (Fig. 5.36). For each setup no more than eight sensors were used, including two fixed reference sensors, arranged in two directions (X and Y) parallel to the ground and perpendicular to each other (channels 1 and 2 at the second floor). Acceleration transducers were placed along the out-of-plane direction on both sides of the wing. The records concentrated mainly on the perimeter walls as the aim was to investigate any dissimilar dynamic behavior of the two facades due to the heavy state of damage of the structure. For each setup 3 records (65'536 points each) were acquired with a sampling rate of 100 SPS (samples per second).



a)



b)

Fig. 5.36 - Scheme of four setups (S1-red, S2-yellow, S3-green and S4-blue) defined for the first (a) and second (b) floors of the monument

Data series acquired at sampling frequency of 100Hz are processed, using FDD, EFDD and p-LSCF methods. Peaks in the frequency domain related to structural modes were selected and the corresponding mode shapes extracted. The acquisitions of the various setups were firstly analyzed separately and then put together in one global identification. The extraction of modal parameters clearly indicates four mode shapes orthogonal to the façade in the frequency range 2,9Hz -

5,6Hz, higher local mode at 8,8 Hz as well as many other peaks - at higher frequencies - difficult to assess.

Fig. 5.37 shows the diagram of the average of the normalized singular values of the spectral density matrix, whose peaks indicate the frequencies of structural interest. It can be noted the resonance peaks already seen in the range of values of 3-8 Hz.

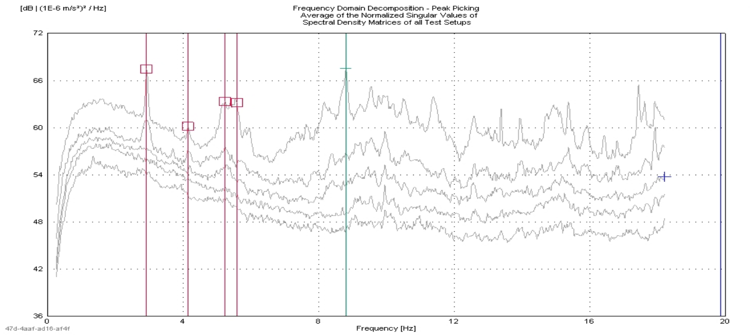


Fig. 5.37 - Frequency Domain Decomposition - Peak picking. Average of the normalized singular values of the spectral density matrix: peaks indicate the structural frequencies

In Tab. 5.4 modal parameters extracted with different OMA techniques are listed and compared, whereas Fig. 5.38 presents the first five identified mode shapes of the South-east wing of the fortress.

Tab. 5.4 - Modal parameters estimation of the fortress implementing and comparing various identification techniques (frequency f, damping ratio ξ and MAC value calculated between FDD and EFDD methods)

MODE	FDD	EFDD		MAC
	f [Hz]	f [Hz]	ξ [%]	
1	2,930	2,939	1,21	0,99
2	4,150	4,151	0,90	0,71
3	5,249	5,302	1,55	0,98
4	5,591	5,467	2,53	0,96
5	8,813	8,801	1,07	0,97

The identification of the global vibration modes of the structure indicates that the building, in spite of the high level of damage and the disconnection of the perimeter masonry walls, has still a unitary dynamic response, probably thanks to the provisional emergency interventions that provide a certain level of confinement to the out-of-plane overturning mechanism activated by the earthquake.

MODE 1 - 2,93 Hz **MODE 2** - 4,15 Hz **MODE 3** - 5,25 Hz

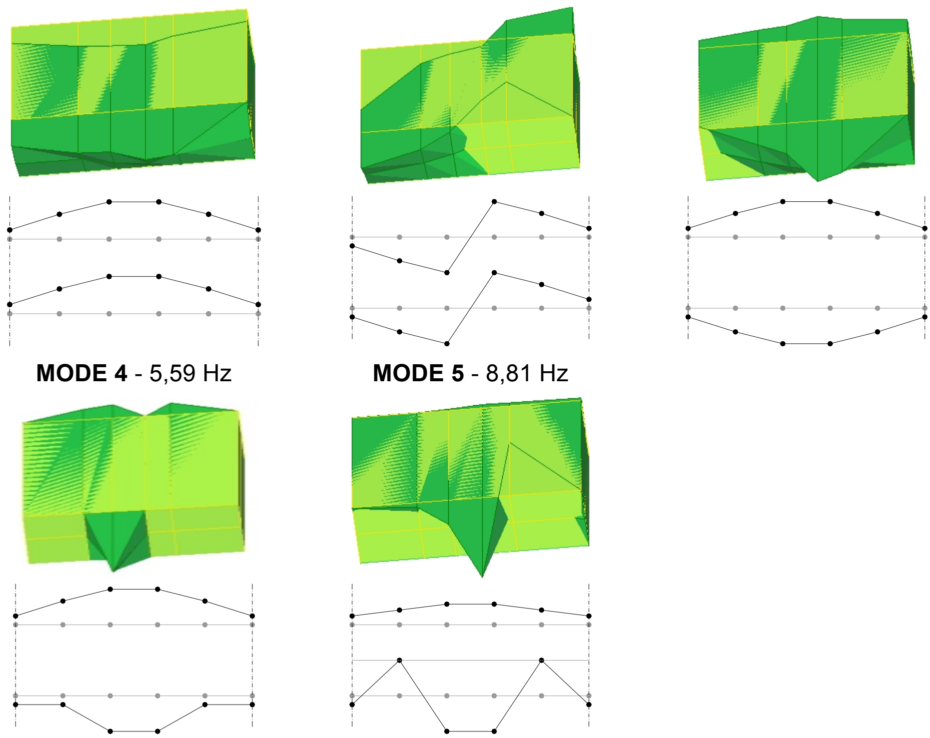
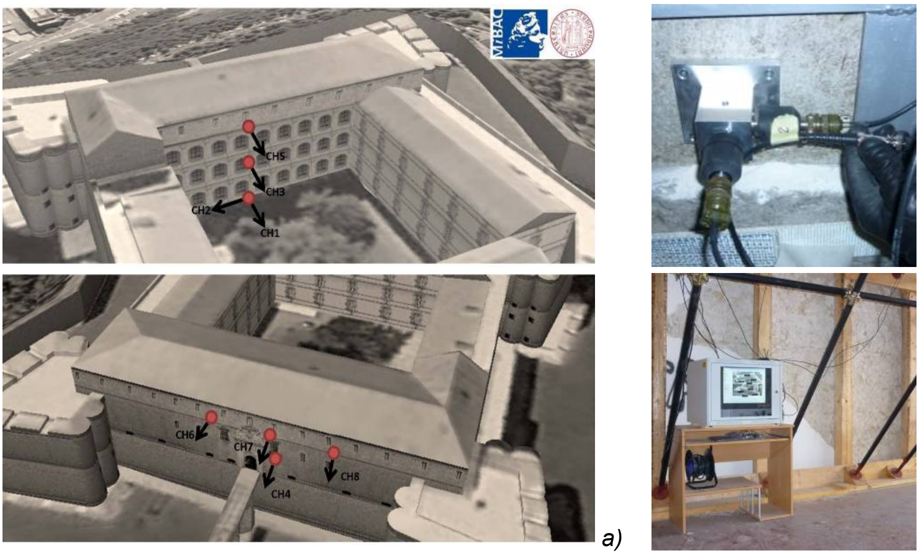


Fig. 5.38 - First five out-of-plane bending modes shapes of the fortress' wing with a schematic representation of the top view

5.6.4.2 Layout of the system

Once the investigation campaign was concluded, a dynamic monitoring system has been installed on the South-East wing of the fortress. The system complements a static monitoring installed immediately after the earthquake by the ISCR (Istituto Superiore per la Conservazione ed il Restauro - National Conservation and Restoration Institute) of Rome, devoted to the control of the crack pattern evolution and of the environmental parameters. The dynamic system is composed by an acquisition unit connected to eight high sensitivity piezoelectric accelerometers. The central unit, located at the second floor of the fortress is provided with a Wi-Fi router for remote data transmission.

A couple of reference sensors is fixed at the base of the structure (at the foot of one of the massive pillars on the inner courtyard, with CH1 orthogonal to the façade and CH2 parallel to it) for the record of the ground acceleration both in operational conditions and during seismic events. The positioning of the acceleration sensors on the elevation of the South-East was decided according to the results of the dynamic identification. Sensors were fixed orthogonally to the façade, following vertical and horizontal lines, on the internal and external façades, with an increased number of sensor at the second level (Fig. 5.39).



a) b) c)

Fig. 5.39 - a) Layout of the dynamic monitoring system installed on the South-East wing of the fortress. b) Acceleration transducers fixed at the base along two horizontal perpendicular direction to record ground motion. c) Acquisition unit

The selected monitoring strategy is similar to the monitoring system of the Arena of Verona (see §5.2.3.3 for more details). Dynamic data are collected both at fixed time intervals ("long" acquisitions, corresponding to 131'072 points, or to 21'51'' of record at a sampling frequency of 100 Hz, each 24 hours) to allow successive dynamic identification of the structure with different environmental conditions, and on a trigger basis (shorter records, 3'35'' at a sampling frequency of 100 Hz), when the signal, on one of the acceleration channels, exceeds the predefined threshold (meaningful event, e.g. earthquake), either in frequency or time domain.

5.7 Conclusions

In this chapter the general methodology for the application of SHM to cultural heritage structures and monuments, proposed in Chapter 4, is validated on selected case studies. The choice of specific pilot projects, among the 9 available monitored structures, is made according to some general principles and in particular considering the final aims for which monitoring systems were designed and implemented.

Following this approach the case of the Roman Arena of Verona was selected as an example of SHM applied to increase the knowledge level on a structure and assess possible needs of strengthening, avoiding in this case the execution of unnecessary and possibly invasive interventions.

In other cases SHM acts in the context of incremental application and the long-term validation of strengthening interventions, when their execution is indispensable to increase the safety level, especially against earthquake-induced risks. The case study of the Cansignorio stone tomb belong to this category.

Finally SHM plays a fundamental role in the management of the emergency activities in a post-seismic scenario and two emblematic cases (Civic tower and Spanish Fortress), heavily damaged by the 2009 l'Aquila earthquake are presented and discussed. Here the presence of a monitoring system guarantee the continuous control of the induced damage and particularly the validation of the effectiveness of the adopted provisional strengthening measures. Furthermore SHM help scheduling the execution of definitive interventions and act as a tool for the safety of workers employed in their execution.

In this framework, SHM case studies have been presented, focusing in particular on the importance of the execution of detailed historical research and the implementation of a series of inspections to increase the knowledge level on the building and define the layout of the monitoring system. To this aim, both detailed crack and damage pattern surveys as well as the implementation of NDT and MDT on the one hand and execution of dynamic identification tests to capture and understand the global dynamic response on the other hand were extensively applied. The final step is the definition of the architecture of the monitoring system, the selection of the optimum number and position of sensors and the choice of adequate monitoring strategies. Within this complex approach, it was possible to increase the performance and thus benefits of SHM applied to CH structures.

6 AUTOMATED ALGORITHMS FOR STATIC MONITORING

6.1 Introduction

The central core of each monitoring system is the capability to automatically extract information to increase the efficiency and provide almost real time data on the health state of the controlled structure. This chapter presents the development in MATLAB environment of an automatic computerized system for static monitoring data processing.

The idea is to create dedicated subroutines running on a on line basis on the central server of the University of Padova where monitoring data are continuously transmitted. The developed algorithms process static data automatically and send early warning messages in case of exceedance of predefined thresholds or in case of system's malfunctioning. A Graphical User Interface (GUI) was also developed to post-process the acquired data, analyzing and correcting possible errors and cross correlating the outputs with environmental parameters.

6.2 General framework of the algorithms

Each monitoring system is characterized by its own layout and number of sensors. Typically, static sensors usually control specific local damages or crack pattern or entire macroelements and collapse mechanisms, strictly related to the safety level of the building. Sensors implemented to measure those parameters are linear displacement transducers, inclinometers, pendulum, etc. In other cases the health state can be monitored through the control of working stresses and strains on specific points, structural or reinforcement elements (by means of strain gauges, fiber optic sensors, etc.). Also environmental parameters (especially temperature and relative humidity) are usually recorded by static systems.

For more details on specific static sensors and instrumentations traditionally implemented in monitoring systems the reader has to refer to §2.3.1.3.

In case of static systems a few measurements per hour are enough to characterize the behavior of the monitored parameters.

A typical acquisition file (Fig. 6.1) recorded by the central unit of the system is a matrix whose columns and rows are composed by static channels and records at specific sampling intervals (usually varying from 30 min to 6 hours) respectively.

Date and hours	P_1	P_2	P_3	P_4	P_5	P_6	P_7	P_8	T1	H1	T2	H2
------------------	mm	mm	mm	mm	mm	mm	mm	mm	°C	%	°C	%
2011/04/28 08:54:03	9,029	49,189	51,274	39,537	46,292	40,590	39,777	48,232	10,633	76,749	11,401	78,185
2011/04/28 09:54:03	9,012	49,194	51,258	39,539	46,295	40,598	37,010	48,239	10,859	75,302	12,290	71,822
2011/04/28 10:54:03	9,000	49,195	51,249	39,539	46,300	40,598	44,841	48,235	10,972	74,777	13,842	62,896
2011/04/28 11:54:03	8,971	49,196	51,233	39,539	46,244	40,589	36,983	48,234	11,135	73,176	14,933	58,776
2011/04/28 12:54:03	8,957	49,197	51,228	39,539	46,166	40,581	36,990	48,233	11,211	72,265	15,565	58,718
2011/04/28 13:54:03	8,952	49,200	51,221	39,537	46,178	40,589	37,000	48,230	11,425	71,399	15,403	58,862
2011/04/28 14:54:03	8,939	49,215	51,216	39,535	46,154	40,580	36,998	48,228	11,571	70,793	15,782	55,680
2011/04/28 15:54:03	8,935	49,214	51,239	39,534	46,179	40,588	37,007	48,227	11,576	71,993	14,989	62,034
2011/04/28 16:54:03	8,958	49,211	51,290	39,535	46,178	40,588	37,021	48,232	11,121	77,233	11,787	82,000
2011/04/28 17:54:03	8,963	49,209	51,290	39,535	46,175	40,590	37,015	48,233	11,122	79,339	10,852	88,893
2011/04/28 18:54:03	8,964	49,207	51,288	39,536	46,173	40,593	37,013	48,234	11,072	76,630	11,018	84,511
2011/04/28 19:54:03	8,967	49,205	51,290	39,535	46,166	40,593	37,011	48,235	11,021	78,664	11,069	86,049
2011/04/28 20:54:03	8,971	49,203	51,288	39,536	46,142	40,594	37,011	48,236	11,009	78,453	11,281	86,190
2011/04/28 21:54:03	8,974	49,202	51,290	39,536	46,129	40,595	37,011	48,236	10,971	79,390	11,133	86,646
2011/04/28 22:54:03	8,976	49,199	51,291	39,536	46,139	40,597	37,012	48,236	10,965	79,318	11,336	86,936
2011/04/28 23:54:03	8,982	49,199	51,296	39,537	46,135	40,598	37,016	48,236	10,967	79,133	10,499	88,849
2011/04/29 00:54:03	8,984	49,197	51,298	39,537	46,129	40,599	37,017	48,236	10,946	80,269	10,399	90,759
2011/04/29 01:54:03	8,985	49,196	51,297	39,538	46,126	40,600	37,018	48,236	10,902	80,902	10,652	89,988
2011/04/29 02:54:03	8,990	49,195	51,302	39,539	46,120	40,602	37,019	48,236	10,858	81,245	10,284	90,065
2011/04/29 03:54:03	8,995	49,194	51,303	39,539	46,119	40,603	37,021	48,235	10,807	80,840	9,500	92,837
2011/04/29 04:54:03	8,996	49,193	51,303	39,539	46,116	40,604	37,021	48,235	10,795	80,865	10,258	89,265
2011/04/29 05:54:03	9,002	49,192	51,303	39,540	46,109	40,606	37,020	48,235	10,732	80,285	9,683	89,997
2011/04/29 06:54:03	9,008	49,192	51,307	39,541	46,103	40,608	37,019	48,235	10,682	79,768	9,689	88,920

Fig. 6.1 - Typical acquisition file of static system: each column contains data from a single sensor, each row corresponds to a measurement at a predefined sampling interval

The automatic algorithm for static data treatment and analysis was developed in MATLAB environment. Each monitoring system has its own algorithm, designed and implemented according to the specification of the system, i.e. typologies and numbers of sensors.

The algorithm is composed by two distinct and separated procedures. The first one, fully automated, does not require user interaction and processes row data on arrival at the central server of university. The algorithm performs a preliminary check on data, sending early warning massages if one sensor exceeds the predefined safety threshold or if any kind of system's malfunctioning is recorded. This routine has specific characteristics for each monitoring system.

The second part of the procedure is based on the development of a graphical user interface (GUI), simple and universally applicable to every monitoring system. It allows to post-process the acquired data, analyzing and correcting possible errors, which require an expert judgment of the user.

6.3 Automated subroutine

The automatic subroutine, developed for raw data processing, is running directly on the central server of university, where static data are transmitted on a daily basis. Fig. 6.2 represents the flowchart of the algorithm that can be ideally subdivided into four different phases:

(i) Files selection

(ii) Sorting and correction
(iii) Treatment and analysis
(iv) Plot of results

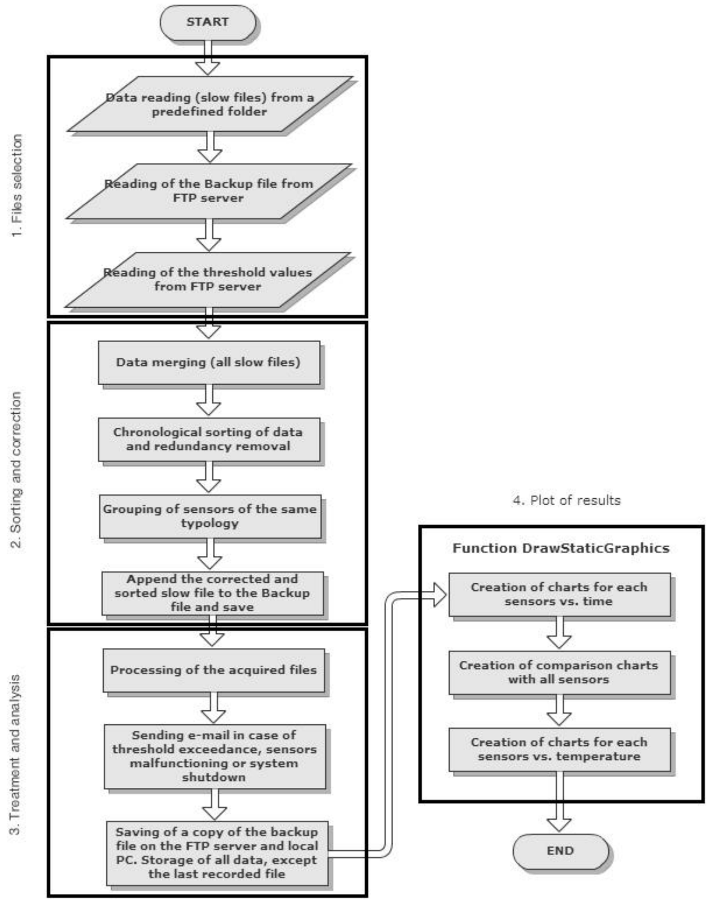


Fig. 6.2 - Flowchart of the automatic subroutine for static data processing

6.3.1 Files selection

The algorithm reads static data (`'_Slow.txt'` files) contained in the FTP server's folders and subfolders. The subroutine reads also the `Backup.txt` file, containing data already corrected and sorted by date, from the activation of the system until the last processing performed by the algorithm, and the `Threshold.txt` file, containing safety threshold values defined by the user for each sensor through the GUI.

Afterwards the subroutine appends all the new records to the *backup* file, creating the acquisition matrix, where columns are defined by sensors and rows by measurements. Finally it saves a temporary file (`Temp.txt`).

6.3.2 Sorting and correction

The algorithm sorts chronologically data of the `Temp.txt` file and reorders the columns of the acquisition matrix in order to group together sensors of the same typology. Following this procedure, for example, displacement transducers are moved to the first columns, followed by environmental sensors (temperature and relative humidity), and, if present, inclinometers, strain gauges and possible new type of sensors. This part of the algorithm is obviously specific and different for each system, since usually there is no uniformity in the number of sensors and in the positions within the file of records.

Once all the measurements are grouped and sorted correctly the routine checks and corrects possible redundancies (repetition of rows with same data) and saves the `Backup.txt` file containing the new acquisition matrix.

6.3.3 Treatment and analysis

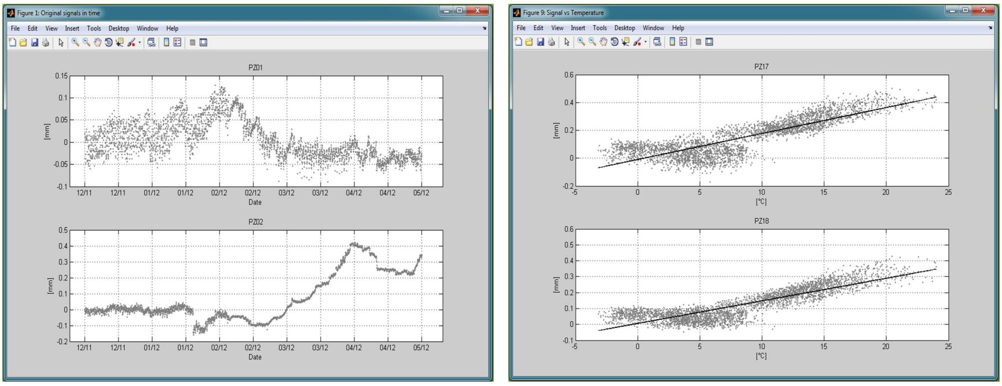
The algorithm sets to zero the initial values of each sensor (apart from temperature and relative humidity) and possibly converts the acquired signals from a value of electric potential difference (volt) into a measure of displacement (mm), angle (deg), strain (%), etc. The algorithm checks whether the recorded signals for two consecutive values exceed both the attention and alarm thresholds and if the monitoring system is working properly sending regular blocks of data.

In case the subroutine records some anomalies, an email is automatically generated, containing the early warning messages (threshold exceedance, system shutdown, sensor malfunctioning).

Finally the algorithm moves all the `'_Slow.txt'` files contained in the folder into an archive folder and overwrites the updated and corrected `Backup.txt` file, so that it serves as input for the subsequent openings of the automated subroutine.

6.3.4 Plot of results

The algorithm, through `DrawStaticGraphics` function generates automatically a set of graphs and charts containing the analysis results. Those graphs are saved directly in an archive folder of the server and can be sent by e-mail to the users, responsible of the monitoring system, or directly to persons or entities that are in charge of the management of the monitored structure or infrastructure.



a) *b)*

Fig. 6.3 - Automatic generation of graphs: a) signal vs. time and b) signal vs. temperature plots

Graphs automatically generated by the subroutines are listed hereafter:

(i) For each sensor: signal vs. time (Fig. 6.3a);
(ii) For each sensor: signal vs. temperature (or relative humidity) (Fig. 6.3b).
(iii) For group of sensor: signal vs. time (Fig. 6.4).

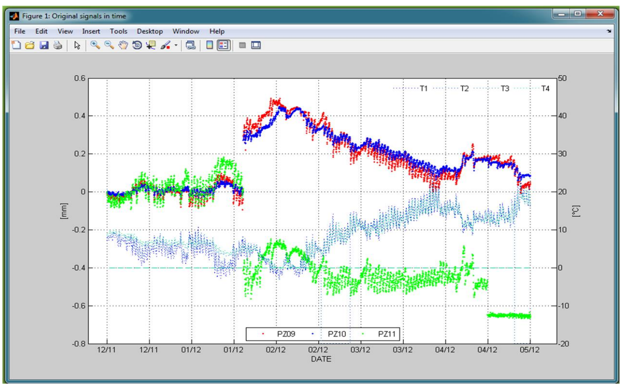


Fig. 6.4 - Automatic generation of graphs by group of sensors: signal vs. time plot

6.4 Graphical User Interface for static systems

To facilitate and reduce the time of data displaying and processing a graphical user interface (GUI) was designed and developed in MATLAB environment. This interface was conceived to fit any monitoring system through the input of the number of sensors and the choice of the reference channels for environmental parameters. The following paragraph describes the development of the GUI and the main features of the software.

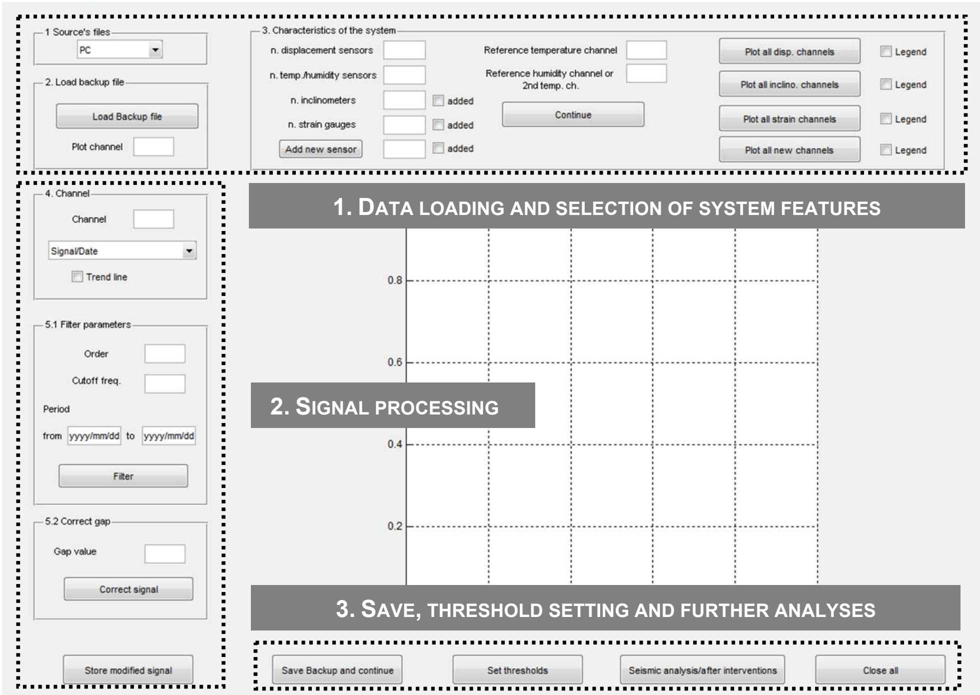
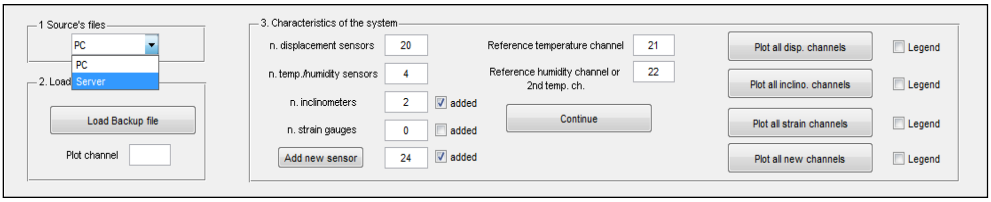


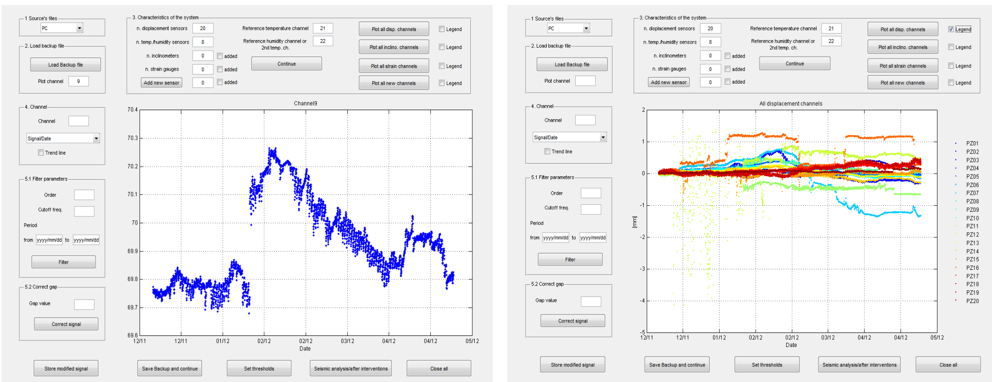
Fig. 6.5 - GUI for static systems: data reading, displaying and analysis

When the program is launched the main window of the software opens (Fig. 6.5). The top bar of the window is devoted to the loading of the backup file created by the automated subroutine (§6.3), displaying of raw data and insertion of the characteristics of the system (number and types of sensors). The reference channels of environmental parameters (one temperature and one relative humidity) are selected as well. In the central blank box data and graphs are displayed. On the left side of the window a specific section is dedicated to signal processing and data correction. The bottom bar contains the commands for saving, setting threshold values and the link to open additional interfaces for more detailed analyses.
The main capabilities and features of the GUI are reported hereinafter.

6.4.1 Data loading and system features definition



The user can upload static data from either the local PC where the algorithm is running or directly from the central server through an FTP connection (in the latter case host, username and password have to be inserted). The file to be uploaded is the *Backup* file previously generated and pre-processed by the automated subroutine. When the uploading phase is complete it is possible to display raw data records from each sensor (Fig. 6.6a).



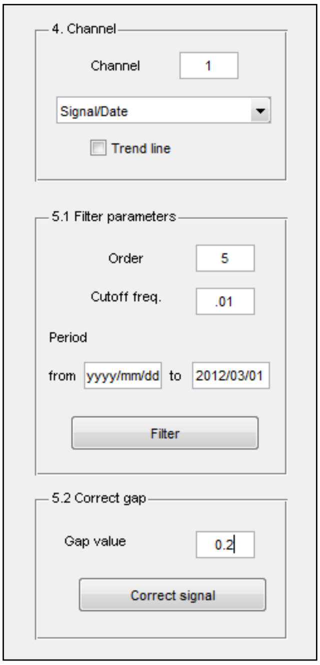
a) b)

Fig. 6.6 - a) Displaying of raw data records for single sensor; b) Plot of all channels subdivided by sensor typology

The specific characteristics of the monitoring system have to be defined (e.g. number of displacement transducers, inclinometers, strain gauges, etc.). If multiple temperature or relative humidity sensors are present, it is necessary to select one reference channels for each of them. It is also possible to add new sensors, specifying ID code and units.

Afterwards the user can plot the graphs by groups of sensors, displaying the results in the central box of the interface (Fig. 6.6b).

6.4.2 Signal processing



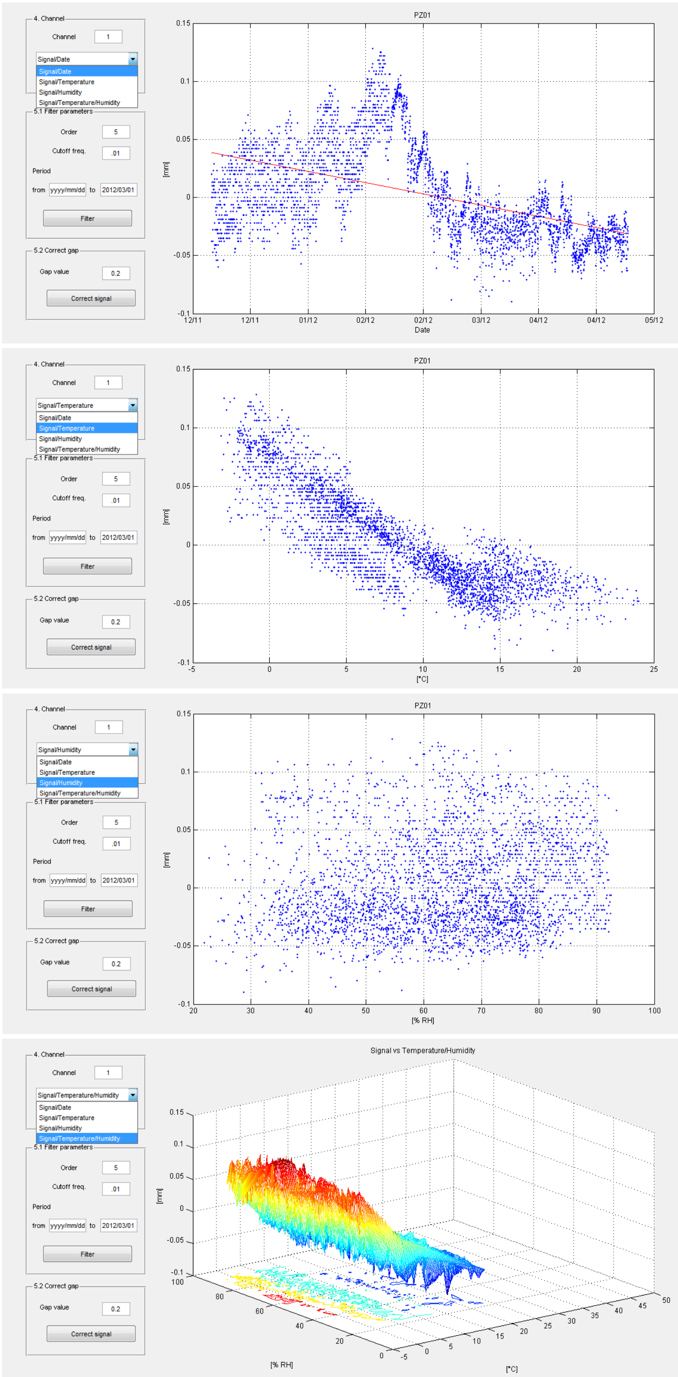
In the signal processing section the GUI allows modifying and post-processing the acquired data.

Firstly a series of graphs can be plotted for each sensor in the central box of the interface, selecting: (i) signal vs. time (Fig. 6.7a), (ii) signal vs. temperature (Fig. 6.7b), (iii) signal vs. relative humidity (Fig. 6.7c), (iv) 3D graphs of signal vs. temperature vs. relative humidity (Fig. 6.7d).

Selecting the option *Trend line* a regression curve that linearly interpolate plotted data is automatically calculated and displayed.

Another interesting function of the GUI is the possibility to filter the signals applying a Butterworth filter (Fig. 6.8a). Prefiltering data can help to remove high-frequency noise or low-frequency disturbances (drift) from the signal. Since the variation of static data over time can be considered as a dynamic system, the application of time-domain data filtering can reduce in some cases the noise produced during the acquisition phase. An alternative is to apply a filter to eliminate or at least reduce the effects of environmental parameters on the measured signals, smoothing the seasonal cyclic variations induced for example by temperature.

The last signal processing tool of the GUI allows the user to correct accidental errors and gaps generated by system's malfunctioning or human interference and not directly correlated to structural changes and/or damages (Fig. 6.8b). In this case the expert judgment of the user is essential to distinguish damaging phenomena from noise or errors that are normally present in the recorded signals.



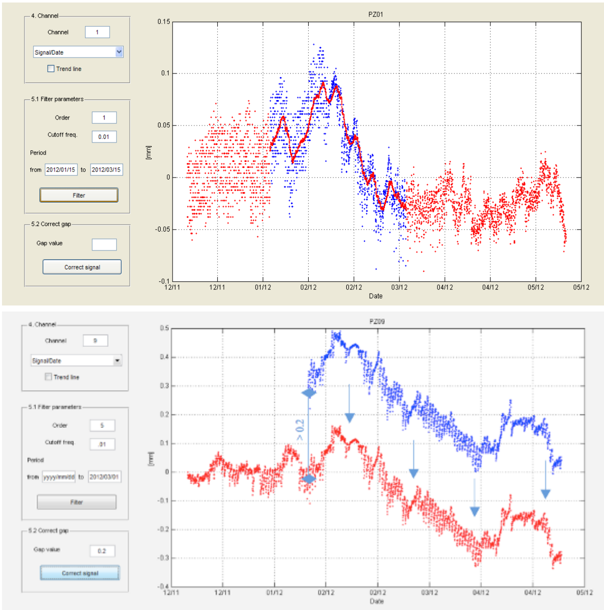
a)

b)

c)

d)

Fig. 6.7 - Plot of graphs for each sensor: a) signal vs. time; b) signal vs. temperature; c) signal vs. relative humidity; d) 3D plot signal vs. temperature vs. relative humidity



a)

b)

Fig. 6.8 - Signal processing tools: a) Time-domain data filtering; b) errors and gaps correction

6.4.3 Save, thresholds settings and further analyses

The function *Save backup and Continue* overwrites the `Backup.txt` file in the server's folder with a new file containing, possibly, the filtered and corrected signals. In this way the automated subroutine will start from the updated backup file to append new measurements.

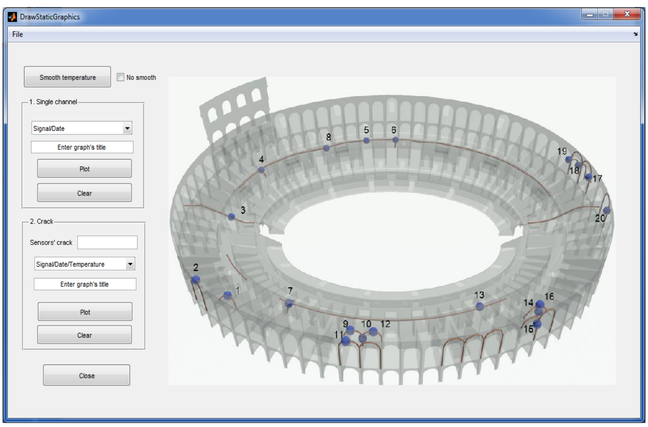


Fig. 6.9 - Interface with the monitoring layout and the position of sensors to plot the results

Once the updated and corrected `Backup.txt` file is saved and stored on the server's folder, an additional interface is displayed with a scheme of the static monitoring layout of the system under investigation. In this way the user can see directly on the screen the position of sensors on the monitored structure, allowing to easily plot the results sensor by sensor or displaying graphs by group of sensors (Fig. 6.9).

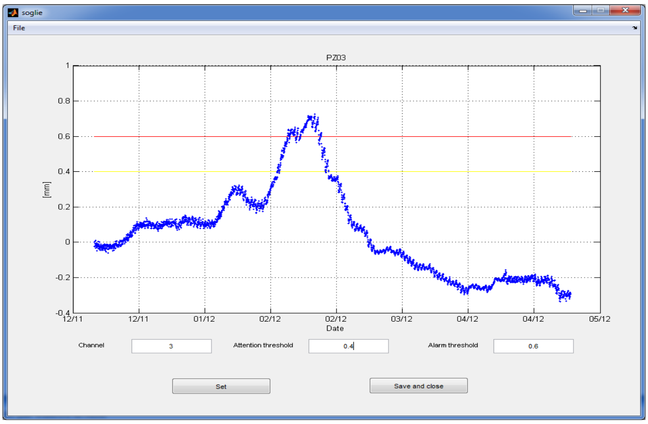


Fig. 6.10 - Interface to set attention (yellow line) or alarm (red line) thresholds

Set threshold button opens another interface with the possibility to set attention and alarm thresholds for each sensor: all information will be archived on the `Threshold.txt` file in the server's folder. The automated subroutine, based on the threshold values defined by the user, will check automatically if a sensor exceed the threshold, sending, in case, an early warning message.

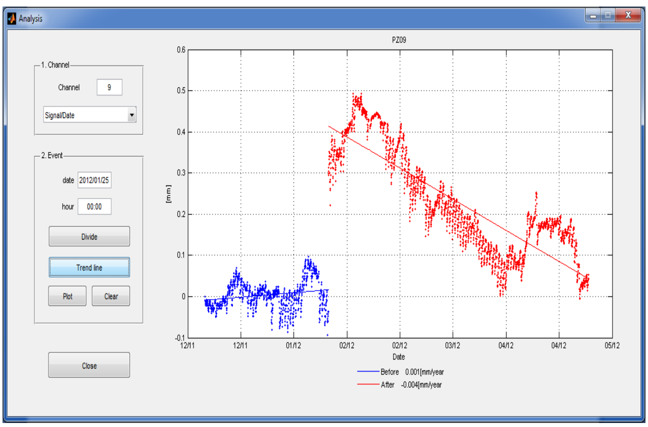


Fig. 6.11 - Post earthquake/intervention analysis: the signal is split into two part: before (blue) and after (red) the event

Finally the *Post earthquake/intervention analysis* function gives the possibility to perform particular data analyses in case of exceptional events, such as earthquakes or after the execution of strengthening interventions. This function split the signal into two parts (before and after the event) and allows detecting any changes in the structural behavior of the monitored parameter. Also in this case an interpolating trend line can be calculated and displayed.

6.5 Application and validation of the algorithms

Potentialities and performance of the algorithm for static monitoring data processing and analysis are tested and presented in this paragraph. The case of the Arena of Verona is taken as an example to show how static data can be processed and cross correlated with environmental factors.

The static system of the Arena is composed by 20 displacement transducers (for more details see §5.2.3), installed to control specific damages and crack pattern or entire macroelements (e.g. detachment of the perimeter stone wall from the radial walls). To simplify and make more clear the representation of results, displacement transducers are collected into 10 different groups according to their position (Fig. 6.12).

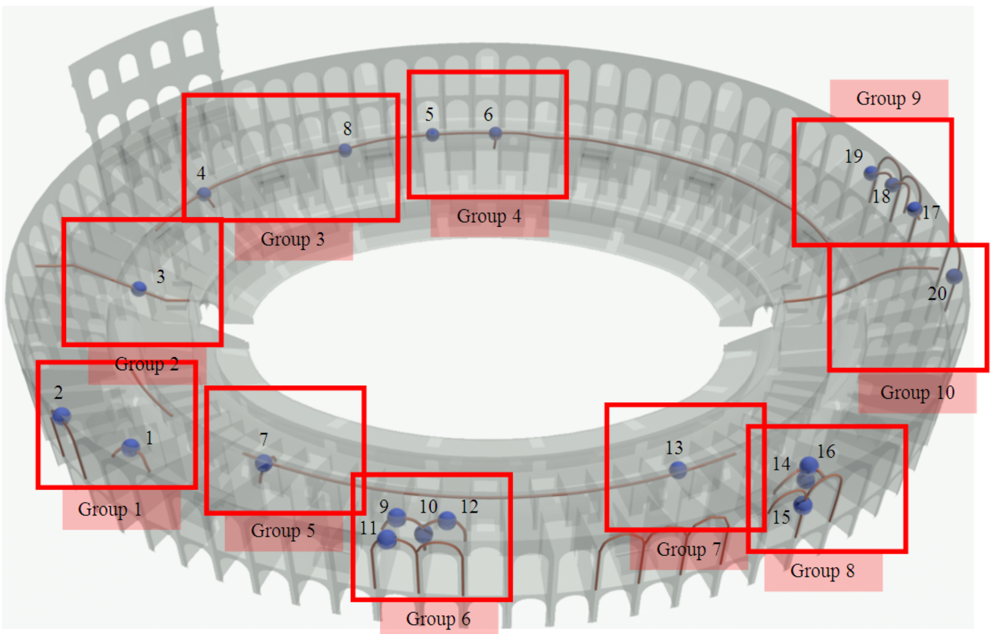


Fig. 6.12 - Grouping of the static sensors of the Arena of Verona

Within this paragraph only some representative results will be presented, corresponding to group #1, #6 and #9.
The idea is to demonstrate potentialities and performance of the dedicated software for monitoring data treatment and analysis, showing how it is possible to process raw static data, detect anomalies through the early warning tool and cross correlate outputs with environmental measurements.
The analysis concentrates on the first six months of monitoring, from December 2011 to May 2012. During this period in fact Northern Italy and the city of Verona were hit by a series of low/moderate seismic events, which culminated with the strong earthquakes of the 20th and 29th May 2012 in Emilia-Romagna region. The monitoring system recorded and captured all the seismic events and the dynamic behavior of the structure will be described in detail in Chapter 8 (§8.4). Here the static response is evaluated, with particular attention to the consequences induced by the earthquake of 25/01/2012, that induced the highest level of vibrations on the monument.

- Group #1

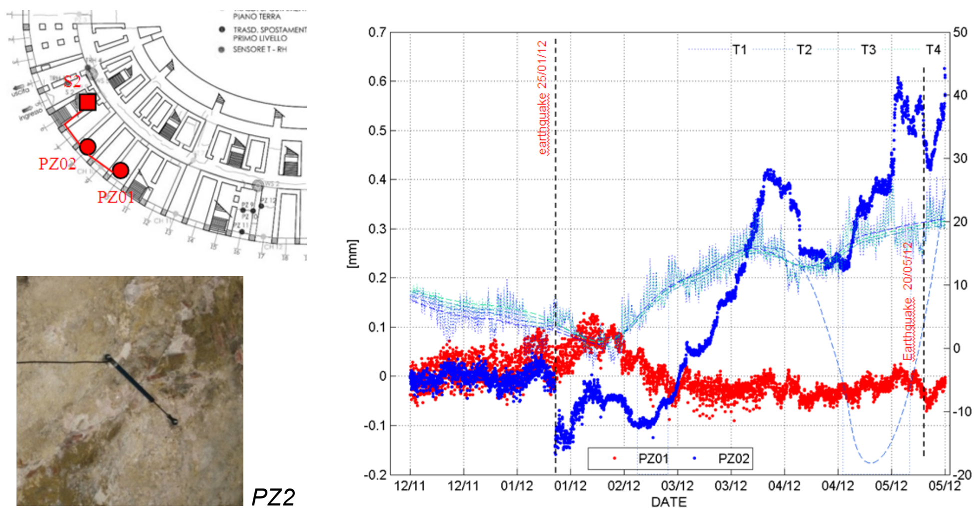
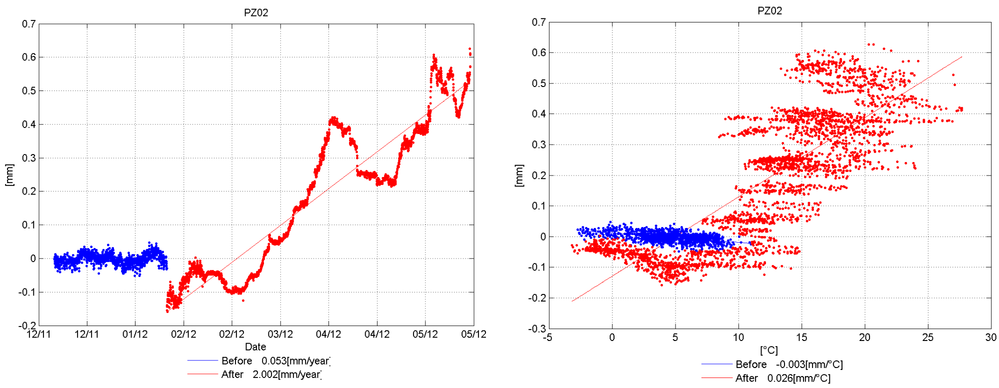


Fig. 6.13 - Sensor group #1: crack opening vs. time plot

As it can be noted in Fig. 6.13 the earthquake caused a sudden slight opening of the crack monitored by sensor PZ02, located in a vaulted niche ("arcovolo") at the second level. The recorded displacement is very small (about 0,1 mm) and certainly not alarming from a structural point of view. It can be noted in fact that temperature-induced variations are much more significant. On the contrary the crack monitored by the other transducer (PZ01) was not subjected to any change after the earthquake.

In this case the useful tool incorporated into the processing software for static monitoring gave the opportunity to analyze more in detail the behavior of the monitored lesion PZ02, splitting the trend of crack opening into two parts: before and after the seismic event (Fig. 6.14a). Combining this observation with environmental factors through a crack opening vs. temperature plot (Fig. 6.14b), it is possible to evaluate the slope of the interpolating curve before and after the event. In this case not enough data are available for describing the pre-event behavior. However if more information would be present, similar tendencies before and after the event meant that no permanent damage occurred. In general this tool can be exploited to this aim.



a) b)

Fig. 6.14 - PZ02: pre-post event analysis. crack opening vs. time (a) and crack opening vs. temperature (b) plots. In blue and red pre- and post-earthquake measurements respectively.

➢ Group #6

Similar considerations can be traced for the group of transducers #8. Sensors PZ09 and PZ10 monitor a partial macroelement interested by a passing through crack that cut into two portions two adjacent vaulted niches (“arcovoli”), at the second level of the Arena. PZ11 controls the detachment between the perimeter stone block layer from the radial walls of the monument. The earthquake induced a sudden opening of the cracks monitored by PZ09, PZ10 and a reclosing of the lesion controlled by PZ11 (Fig. 6.15). Also in this case the absolute displacements are very small and did not compromise the structural stability.

Again the post-earthquake analysis tool was successfully implemented splitting the response into pre- and post-event branches, comparing the behavior thanks to a crack opening vs. temperature plot.

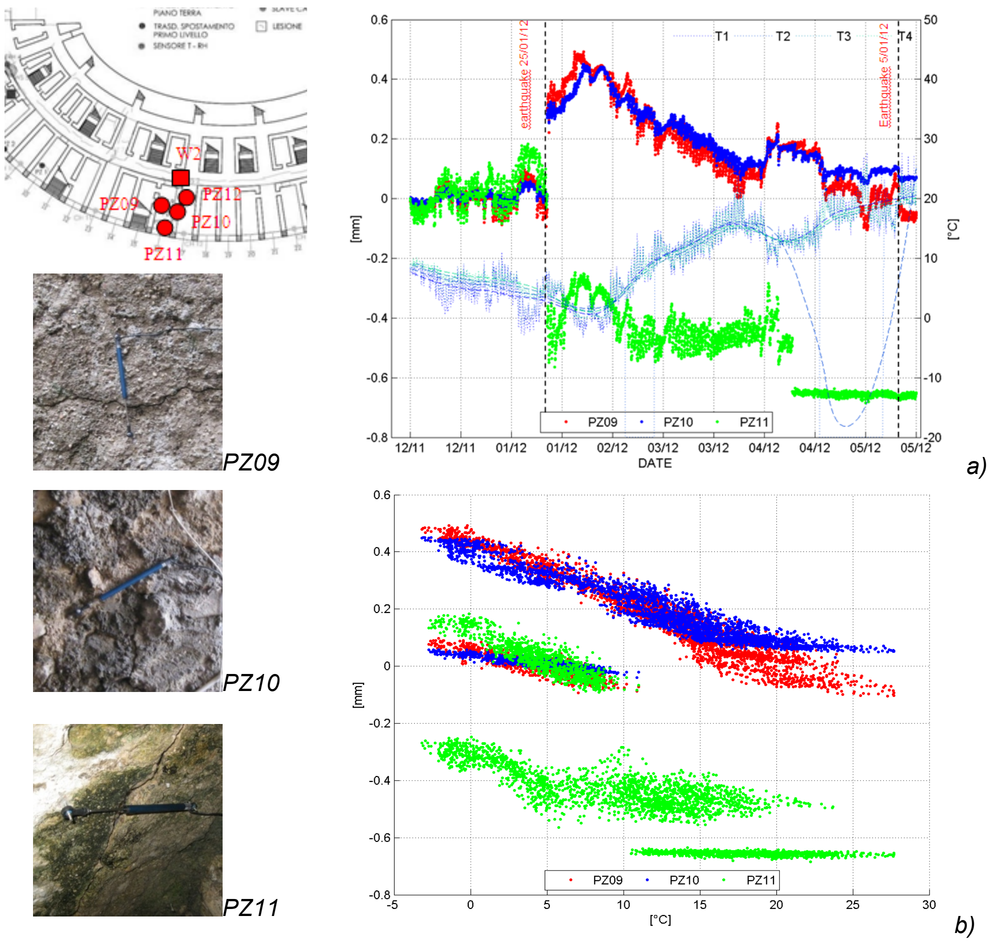


Fig. 6.15 - Sensor group #6: crack opening vs. time (a) and crack opening vs. temperature (b) plots

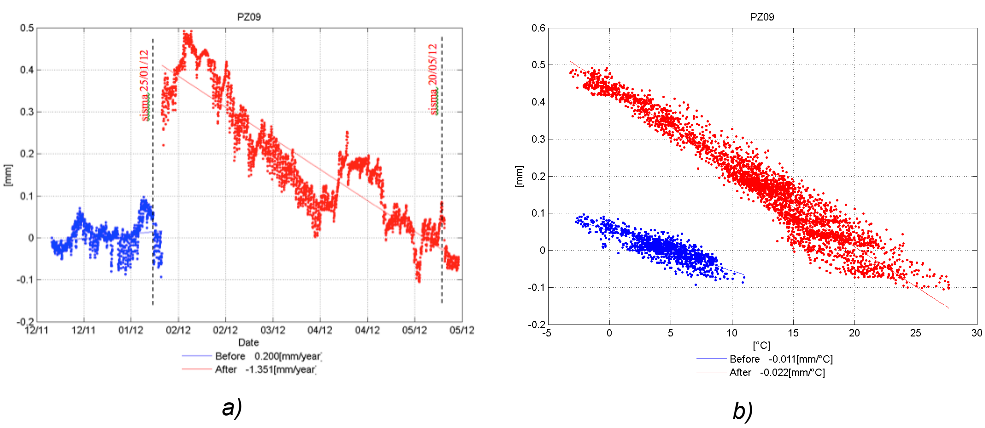


Fig. 6.16 - PZ09: pre-post event analysis. crack opening vs. time (a) and crack opening vs. temperature (b) plots. In blue and red pre- and post-earthquake measurements respectively

Here it is evident that the pre-earthquake behavior is very similar to the post-earthquake one since the interpolating curve of the crack opening vs. temperature diagram have the same slope. A preliminary conclusion, even if not enough data are available on the pre-earthquake condition, is that the earthquake did not caused a permanent damage on the structure.

➢ Group #9

Finally the group of transducers #9 is analyzed. These sensors monitor some local cracks on the vaulted niches (“arcovoli”) in the South-East part of the Arena.

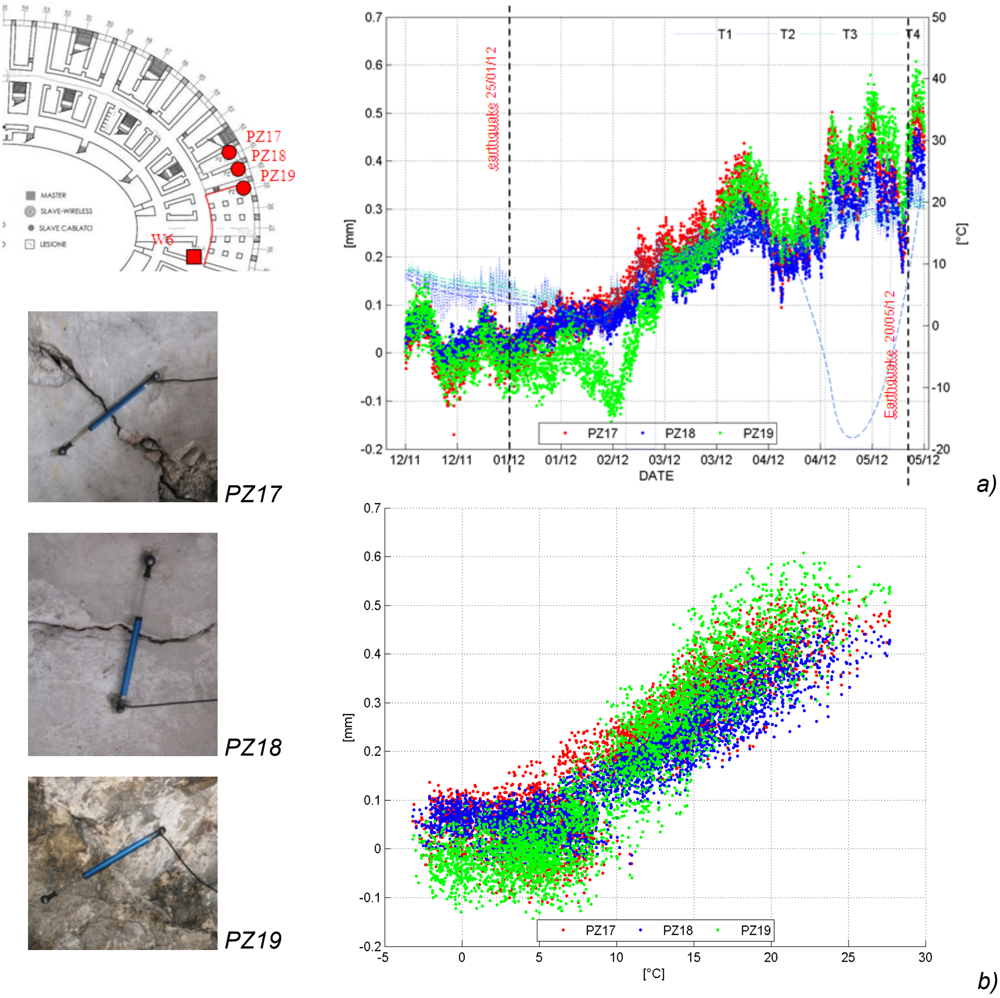


Fig. 6.17 - Sensor group #9: crack opening vs. time (a) and crack opening vs. temperature (b) plots

In this case the earthquake did not induced any anomalous displacement of the cracks' edges. The crack pattern is stable and follows almost perfectly temperature variations (Fig. 6.17a). Looking at the crack opening vs. temperature plot (Fig. 6.17b) it is in fact possible to appreciate the spectacular behavior, exactly alike of the three monitored lesions (PZ17, PZ 18, PZ19). Cracks tend to open during warm periods and are characterized by a linear dependency with temperature above 8°C and tend to reclose with low temperatures with an almost constant behavior in the range -3°C - 8°C. This clear bilinear behavior is visible in Fig. 6.17b.
The developed algorithm for data processing and analysis also in this case prove to be very useful to combine and cross correlate static monitoring outputs.
Moreover the sudden opening/reclosing of some monitored lesions (in groups #1 and #6) were recorded by the automatic subroutine that sent immediately by e-mail an early warning message, alerting about the anomalous behavior. Nevertheless an in-depth analysis of the post-earthquake response allowed excluding any permanent damages induced by the earthquake.

6.6 Conclusions

This chapter presented the development of automated subroutines and processing software for the analysis of data collected by static monitoring systems. The algorithms are composed by a fully automated script, running on the central server of the University of Padova, that process raw data on arrival. It is also integrated by an early warning system that alerts if one or more sensors exceed predefined safety thresholds or if a system's malfunctioning is recorded.
The automatic routines are then complemented with a GUI that allows post-processing the measured data. In particular it is possible to apply filtering algorithms and perform punctual corrections in case the user surveys anomalies and errors.
The software is conceived to combine and cross correlate static measurements with environmental factors, plotting the results using various available plot combinations. Another useful function was implemented to perform specific analysis on data after significant events, such as after an earthquake or the conclusion of strengthening interventions, comparing the static response of the monitored structure in different states.
Finally an example is provided with the application and validation of the developed automated algorithm for static monitoring to the case study of the Arena of Verona.

7 AUTOMATED ALGORITHMS FOR OPERATIONAL MODAL ANALYSIS

7.1 Introduction

This chapter integrates the previous one presenting the development of a processing software for the online automatic identification of modal parameters, in terms of natural frequencies, mode shapes and damping ratios. The algorithm exploits the parametric frequency domain identification method p-LSCF, implemented in the MATLAB toolbox MACEC 3.2, complemented with a new procedure, based on a hierarchical clustering algorithm, developed for the automatic analysis of the stabilization diagrams provided by the p-LSCF technique.
A Graphical User Interface (GUI) was also developed to post-process the acquired data, correlating the outputs with environmental parameters.

7.2 General framework of the algorithms

The implementation of algorithms for the automatic identification of modal parameters is a fundamental tool for dynamic monitoring systems, since the accuracy of the estimates strongly influences the exploitation of results to apply successfully damage detection algorithms or model updating procedures of reference behavioral models.
Time series produced by dynamic monitoring systems are stored as `'_Fast.txt'` files in the acquisition unit with a predefined sampling rate and length. Usually the sampling frequency vary from 80 to 100 Hz with a length of about 20 min and records every 12 or 24 hours for trigger-based systems. In case of continuous monitoring, dynamic information are continuously acquired and records are split into files of 1 hour length at a specific sampling period.
The stored files are then transmitted through an Internet connection to the Department of Civil, Architectural and Environmental Engineering of the University of Padova and automatically processed on arrival.

Also in this case the procedure for the automated extraction of modal parameters is split into two phases: a subroutine is running on the central server and is devoted to the processing of recorded time series and the automatic identification of modal parameters in terms of natural frequencies, mode shapes and damping ratios. Then a Graphical User Interface (GUI) was developed in order to plot the results, create graphs and correlate the extracted modal parameters with environmental factors.

7.3 Subroutine for the automatic modal identification

The processing software was developed in MATLAB environment and includes the automatic identification of the structural modal parameters from the measured responses. The proposed methodology uses the poly-reference Least Square Complex Frequency Domain method (p-LSCF) (see §3.3.3 for more details), implemented in the MATLAB toolbox MACEC 3.2 (Reynders *et al.* 2011) and it is then complemented by a new procedure developed for the automatic analysis of the stabilization diagrams provided by the p-LSCF method, based on a hierarchical clustering algorithm. The application of hierarchical clustering in stabilization diagrams is not a novel idea and the proposed methodology is partially based on similar approaches developed by Magalhães *et al.* 2008 and Reynders & De Roeck 2011 for modal identification of bridges.

Within the present study the identification problem is complicated by the fact that the developed automated techniques are applied to large (and usually massive) masonry buildings, where generally the level of ambient vibrations is not always sufficiently high to excite uniformly the frequency range of interest, especially the higher modes. Moreover in some cases the level of existing damage is pretty high and thus structures do not show a unitary dynamic behavior and their response is characterized by several local modes. A great research effort has been performed in order to overcome these drawbacks.

Fig. 7.1 shows the flowchart of the developed algorithm for automatic identification of modal parameters starting from time series acquired by dynamic monitoring systems.

The automated routine includes the execution of the following tasks:

- Creation of a database with the original data that can be later used to test alternative processing methodologies;
- Pre-processing of data to eliminate the offset and decimate the sampling frequency in the range of interest (usually 0-20 Hz);

- System identification using the p-LSCF method and creation of stabilization diagrams;
- Analysis of the obtained stabilization diagrams through a hierarchical clustering algorithm and automatic extraction of modal parameters;
- Creation of a database with the results of the processing;
- Display of plots with the most relevant results using the developed graphical user interface.

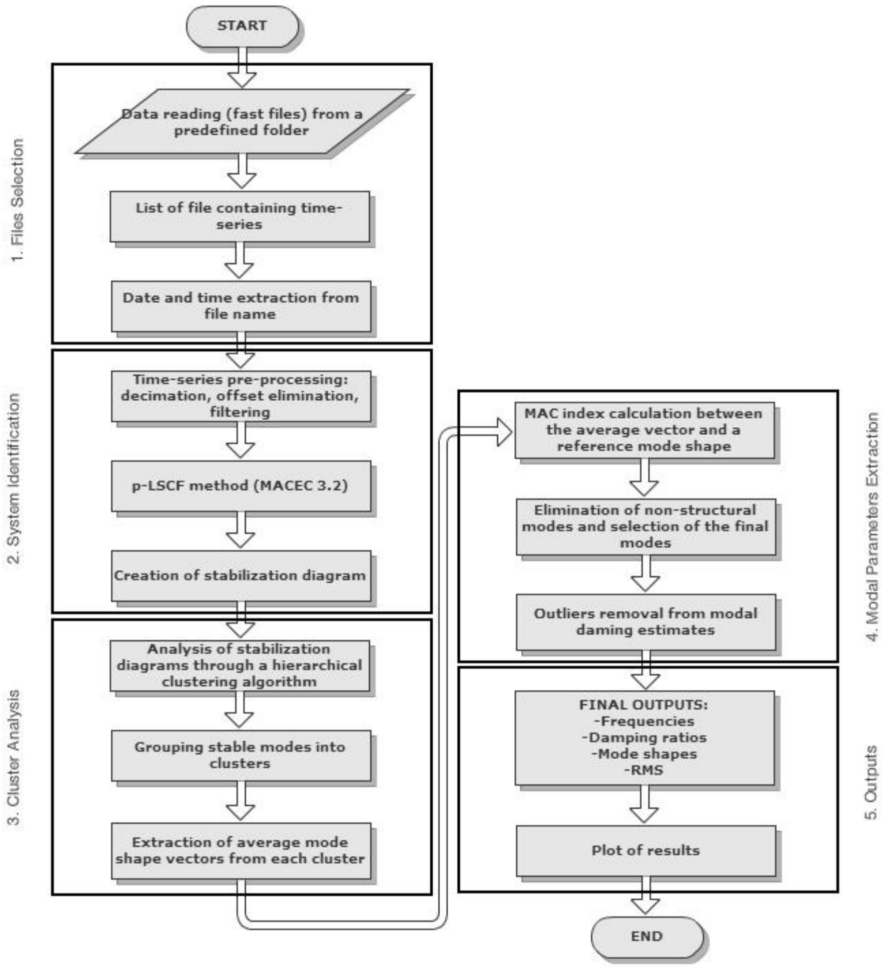


Fig. 7.1 - Flowchart of the subroutine for automatic modal parameters identification

7.3.1 Preliminary remarks

In order to perform automated identification of structural modal parameters using parametric algorithms three complementary aspects has to be considered: (i)

selection of identification algorithms that can provide clearer stabilization diagrams (choice of the optimal OMA identification technique), (ii) study of additional parameters to characterize the modes estimates in order to make a more well-founded selection of the stable modes and (iii) application of automated procedures to analyze and extract information usually presented in stabilization diagrams.
Regarding the first aspect within the present research the frequency-domain p-LSCF method (implemented in MACEC 3.2) was chosen as the best parametric algorithm, able to provide extremely clear stabilization diagrams, facilitating the selection of the physical modes estimates.

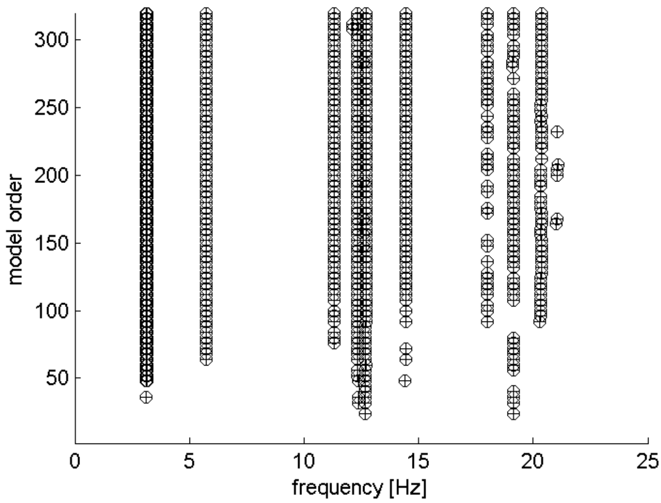


Fig. 7.2 - Typical stabilization diagram obtained through the p-LSCF method in the application to a time series recorded by the monitoring system of the Cansignorio stone tomb in Verona

The stabilization diagram (Fig. 7.2) plots the modal parameters estimate for an increasing model order n which ideally equals twice the number of natural frequencies (i.e. the number of degrees of freedom) of the system. Since the model order is generally unknown, the standard approach is to perform the identification for a wide range of model orders, and to plot all modes in an eigenfrequency vs. model order diagram, i.e. the stabilization diagram.
Thanks to stabilization diagrams it is possible to eliminate poles related to mathematical or spurious modes by means of the application of stabilization parameters. In other words modes that appear in most of the model orders, with consistent frequency, mode shape and damping ratio are classified as stable, whereas modes that only appear in a few orders are considered spurious.
For all monitoring systems analyzed and studied within this work, the same stabilization parameters have been chosen, since this selection proved to be

effective in the extraction of physical modes. The following limits were used: natural frequency variation < 1%; modal damping coefficient < 5%; mode shape stabilization criteria < 1%; modal transfer norm criterion 100%.

Obviously the quality of the stabilization diagrams depends not only from the choice of the identification method, but also from the level of noise contained in the time series under analysis.

Although stabilization diagrams have become a key tool in modal testing, the selection of physical modes as columns in the diagram is often not straightforward, the results may depend on the judgment of the analyst, and possible additional validation criteria may be needed (Reynders & De Roeck 2011).

There is in fact the need for a procedure that allows grouping together the estimates related to the same physical mode. The idea is to develop an algorithm that could reproduce decisions and judgments that an expert takes during the analysis of stabilization diagrams.

The solution adopted here is the use of cluster analysis to directly interpret the information contained in the stabilization diagram. In the next section some basic concepts of cluster analysis are summarized and discussed.

7.3.2 Cluster analysis

Cluster analysis, also called *segmentation analysis* or *taxonomy analysis,* groups data object based only on information found in the data that describes the objects and their relationships. The goal is that objects within a group are similar (or related) to one another and different from (or unrelated to) objects in other groups. The greater the similarity (or homogeneity) within a group and the greater the difference between groups, the better or more distinct the clustering.

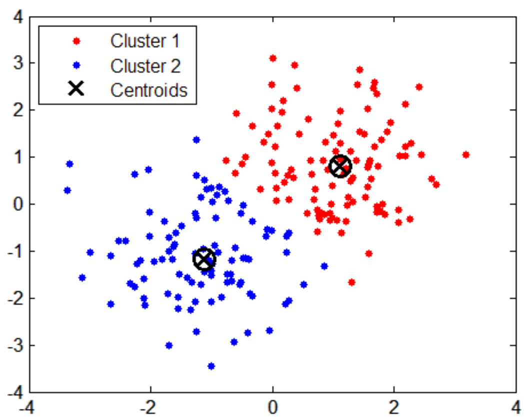


Fig. 7.3 - K-Means clustering procedure: iterative partitioning of the points into k=2 clusters (Matlab 2010)

Most commonly used clustering algorithms can be classified into two general categories: hierarchical and partitional (or non-hierarchical).

A partitional clustering, also known as K-Means clustering, is simply a division of the set of data object into non-overlapping subsets (clusters) such that each data object is in exactly one subset (Fig. 7.3).

These algorithms partition data into *k* mutually exclusive clusters, and returns the index of the cluster to which it has assigned each observation. Unlike hierarchical clustering, *k*-means clustering operates on actual observations (rather than the larger set of dissimilarity measures), and creates a single level of clusters. The distinctions mean that *k*-means clustering is often more suitable than hierarchical clustering for large amounts of data.

The major problems faced by all non-hierarchical clustering procedures are the need to previously define the number of clusters and the requirement to select the clusters seeds. The second drawback is usually overcome by a random selection of the seeds. However, this leads to another problem, that is the not deterministic nature of the solution, as different runs can produce different solutions (Magalhães *et al.* 2008).

Hierarchical clustering algorithms groups data over a variety of scales by creating a cluster tree or dendrogram (Fig. 7.4). The tree is not a single set of clusters, but rather a multilevel hierarchy, where clusters at one level are joined as clusters at the next level. This allows you to decide the level or scale of clustering that is most appropriate for your application.

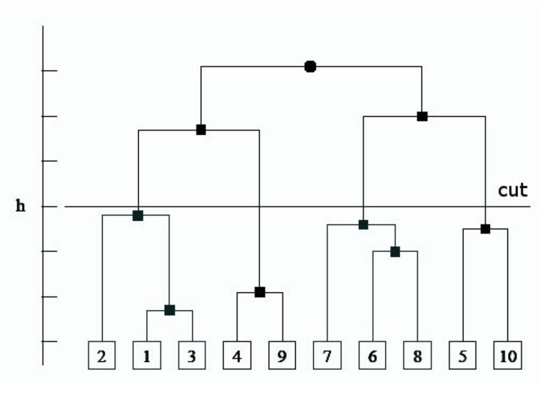


Fig. 7.4 - Hierarchical clustering: cluster tree or dendrogram cut to define 4 clusters

The implementation of the hierarchical algorithms is composed of the following main steps: calculation of the similarity between every pair of objects in the dataset, linking of the objects in a hierarchical tree and, finally, the definition of a rule to cut the hierarchical tree at a certain level, assigning all the objects of each branch to a single cluster (Magalhães *et al.* 2008). Depending on the way used to compute the distance between clusters, there are different hierarchical algorithms: single linkage, complete linkage, average linkage, Ward's method and centroid method (Hair *et al.* 1998).

Theories of cluster analysis can be exploited and applied to the interpretation of stabilization diagrams in order to group together the estimated poles that represent the same physical mode. Following this approach stable poles arranged along vertical lines in the stabilization diagram of Fig. 7.2 can be organized in groups (or cluster) applying a cluster analysis through the procedure described in the following paragraph.

7.3.3 Description of the procedure

The procedure proposed for the automated identification of modal parameters uses a hierarchical clustering algorithm for the analysis of stabilization diagrams obtained by the p-LSCF method.

Once the spurious modes are eliminated from the stabilization diagram, applying the stabilization parameters defined in §7.3.1, N stable poles are left ($\oplus$ denoted a stable pole in the stabilization diagram of Fig. 7.2)., each of one characterized by a frequency f and damping values ξ and by its eigenvector $\mathbf{\Phi}$.

The clustering analysis aims at grouping the N stable poles into homogeneous set (clusters) so that each cluster contains only the poles related to the same structural mode.

To do so a similarity between all the pairs of stable poles has to be calculated through the Euclidian distance between the objects to be clustered. Following this approach the distance between two modes (i and j) is calculated with the formula:

$$d(i,j) = \left|\frac{f_i - f_j}{f_j}\right| + \left[1 - \mathrm{MAC}(\mathbf{\Phi}_i, \mathbf{\Phi}_j)\right] \tag{7.1}$$

where f_i and f_j are the eigenvalues of modes i and j respectively; $\mathbf{\Phi}_i$ and $\mathbf{\Phi}_j$ are their respective mode shapes and MAC denotes the modal assurance criterion. If the distance between two modes is short, it means that both estimates have similar frequency and modes shape and thus they have to be included in the same cluster.

The dimension of the cluster is the total number of modes contained in it.

The sets of consistent frequencies, damping ratios and mode shapes of the poles contained in the generic cluster $\mathbf{C}_i$ are denoted as $\mathbf{C}_{i,f}$, $\mathbf{C}_{i,\xi}$, $\mathbf{C}_{i,\Phi}$ respectively. For example the following cluster

$$\mathbf{C}_i = \{a_1\ a_2\ \dots\ a_n\} \tag{7.2}$$

has dimension $D_{Ci} = n$ and it is composed by poles having the following frequencies, damping ratios and mode shapes:

$$\mathbf{C}_{i,f} = \{f_{a1}\ f_{a2} \dots f_{an}\} \quad \mathbf{C}_{i,\xi} = \{\xi_{a1}\ \xi_{a2} \dots \xi_{an}\} \quad \mathbf{C}_{i,\Phi} = \{\mathbf{\Phi}_{a1}\ \mathbf{\Phi}_{a2} \dots \mathbf{\Phi}_{an}\} \tag{7.3}$$

The distance between two clusters $d(\mathbf{C}_i, \mathbf{C}_j)$ is calculated again with Eq. (7.1) and it is equal to the shortest distances between any point in the cluster $\mathbf{C}_i$ and any point in the cluster $\mathbf{C}_j$.

This procedure allows the construction of the hierarchical cluster tree or dendrogram. Afterwards it is necessary to define the tree cut level, according to the number of clusters to be expected (i.e. the number of physical modes in the stabilization diagram). However since some clusters may contain still spurious modes this number cannot be a priori defined. The solution to this not trivial problem is found in the imposition of a maximum limit for the distance between the poles in the same cluster: the lower is the limit, the higher is the number of resulting clusters and so poles associated to the same physical mode might be split in several clusters. In general, according to Eq. (7.1), a value lower than 1 might avoid the inclusion in one cluster of estimated poles related to different physical modes, since the MAC values, in this case, is close to zero. Obviously this condition is satisfied only if a sufficiently high number of sensors is installed on the monitored structure as for example, especially in case of symmetric structure and closely spaced modes, bending and torsional modes could have very similar mode shapes, difficult to distinguish and separate.

In the present application, good results were obtained with a limit distance of 0,2. When the quality of collected data is higher and the level of noise is lower (this is not the case of our structures, as discussed in the introduction of this paragraph) this limit can be further reduced.

Fig. 7.5 shows the results of cluster analysis with the application of the presented methodology to the stable poles of the stabilization diagram of Fig. 7.2, related to a time series recorded by the monitoring system of the Cansignorio stone tomb in Verona. Each cluster $\mathbf{C}_i$ is represented by an histogram, whose width depends on the variation (or dispersion) of values around the mean frequency calculated among all the n poles contained in the cluster. The height of the histogram corresponds to the dimension $D_{Ci} = n$ of the cluster.

In this case 10 clusters have been identified in the frequency range 0-22 Hz, characterized by a relatively high number of elements in each cluster. However it is noted that some clusters still contained spurious modes induced either by noise contained in the recorded time series or by the overestimation of the system model order. There is also the possibility that some modes corresponds to physical local modes. These problems have been surveyed in the analysis of data of all dynamic monitoring studied within this research.

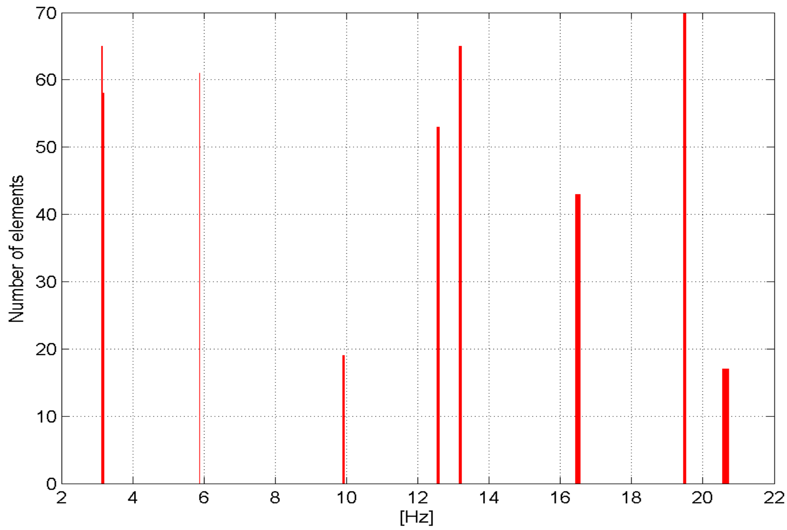


Fig. 7.5 - Cluster analysis results: each cluster is represented by an histogram, whose width depends on the dispersion of values around the mean frequency and whose height corresponds to the number of stable modes contained in it. The six higher histograms (clusters) represent the structural modes.

In order to match modal parameters identified during the preliminary OMA investigations (see Chapter 5 for more details), where a large number of sensors were implemented and thus more reliable results are expected, the next step is the introduction of a further control in order to make the overall procedure much more robust and effective.

The idea is to calculate for each component of the cluster $\mathbf{C}_i$, reported in Eq. (7.3), mean values of frequency $\bar{f}_{Ci}$ and damping ratios $\bar{\xi}_{Ci}$, as well as an average mode shape vector $\bar{\boldsymbol{\Phi}}_{Ci}$.

Then the average mode shape vector $\bar{\boldsymbol{\Phi}}_{Ci}$ of each cluster is compared, through the MAC index, with reference sets of mode shape vectors $\boldsymbol{\Phi}_{rt}$, initially selected by the user and then automatically extracted by the algorithm.

The reference set of mode shape vectors is defined as follow:

$$\mathbf{C}_{r,\Phi} = \{\boldsymbol{\Phi}_{r1}\ \ \boldsymbol{\Phi}_{r2} \dots \boldsymbol{\Phi}_{rt}\} \tag{7.4}$$

with $t = 1, 2, \dots, n$ equal to the total number of structural modes of the system, extracted and identified through the initial identification.

If

$$MAC(\bar{\boldsymbol{\Phi}}_{Ci}, \boldsymbol{\Phi}_{rt}) > 0{,}7 \tag{7.5}$$

it means that the structural mode associated to the cluster $\mathbf{C}_i$ correspond to the mode related to the mode shape vector $\mathbf{\Phi}_{rt}$, otherwise the two modes shapes are not correlated and refer to different modes.

Following this iterative procedure the algorithm match automatically each reference structural mode with the sets of clusters $\mathbf{C}_i$ previously defined.

As previously introduced the reference group of mode shape vector $\mathbf{C}_{r,\Phi}$ is defined by the user at the beginning and extracted by the algorithm in the successive phases. This aspect was introduced to avoid the problem that mode shapes might be subjected to changes during the monitoring period, induced, for example, by structural damages. Thus the algorithm save the last ten modal parameter identifications, selecting the median mode and extracting the reference set of mode shape vector $\mathbf{C}_{r,\Phi}$.

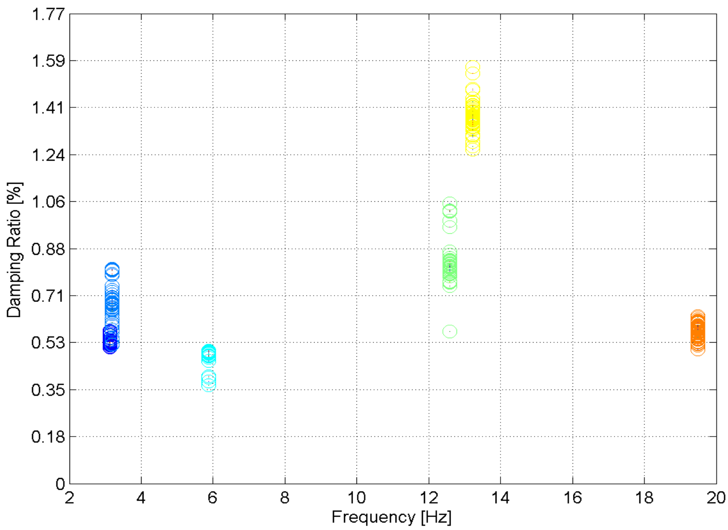


Fig. 7.6 - Modal damping ratio vs. frequency plot of the mode estimates that correspond to the clusters of greater dimension, before the outliers removal

In Fig. 7.6 the estimates of the clusters in Fig. 7.5 are represented into a damping vs. frequency diagram. Each colored vertical line represent an identified structural mode of the Cansignorio stone tomb, selected as reference case to explain the proposed methodology. In particular the first two vertical line (dark blue and blue) around 3.2 Hz corresponds to the first two closely spaced bending modes of the structure, the third line (light blue) around 6 Hz to the first torsional mode, the two green and yellow dots around 12-13 Hz to the second two bending modes and finally the last vertical line (orange) to the second torsional mode. It can be noted a considerable dispersion of the modal damping ratios, especially for the higher modes, with extreme values that distort the mean values.

To overcome this problem an outlier analysis is performed by the algorithm within each component $\mathbf{C}_{i,\xi} = \{\xi_{a1}\ \ \xi_{a2} \dots \xi_{an}\}$ that contain the estimated damping ratios of the cluster $\mathbf{C}_i$ in order to remove the extreme values. To do that a well-establish method, commonly used in descriptive statistics, is applied to remove the outliers, based on the calculation of the lower (Q_1) and the upper (Q_2) quartiles and the interquartile range (IQR). The quartiles of a set of values are the three points that divide the data set into four equal groups, each representing a fourth of the population being sampled:

$Q_1 = 1/4\,(n+1)th$ value, with n the number of data in the cluster

(7.6)

$Q_3 = 3/4\,(n+1)th$ value, with n the number of data in the cluster

The interquartile range is defined as the spread of the middle 50% of the data values n: $IQR = Q_3 - Q_1$.
Once the lower and upper quartiles and the IQR is calculated for each vector $\mathbf{C}_{i,\xi}$ contained in the cluster, the lower and upper bounds (LB and UB) are calculated through the formula:

$$LB = Q_1 - 1{,}5(IQR)$$

(7.7)

$$UB = Q_3 + 1{,}5(IQR)$$

Data lying outside the lower and upper bounds are considered outliers and removed.
After the application of the outlier analysis the extreme values contained in $\mathbf{C}_{i,\xi}$ are removed, allowing a better estimation of the modal damping ratio, as shown in Fig. 7.7.

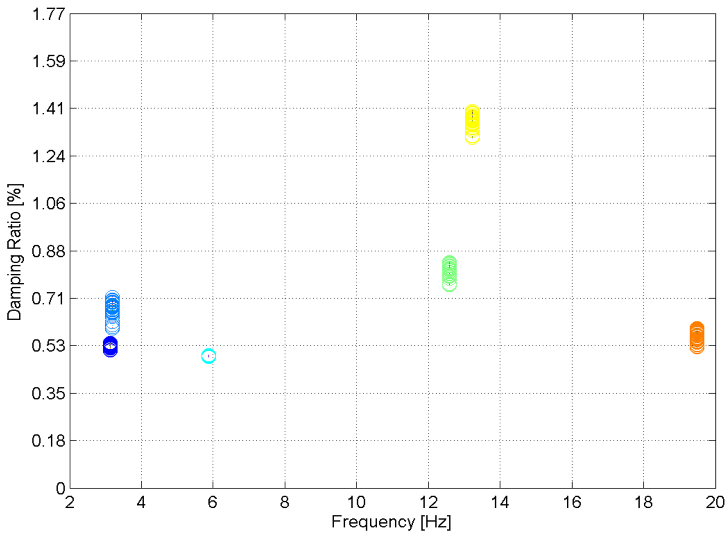


Fig. 7.7 - Modal damping ratio vs. frequency plot of the same mode estimate, after the outliers removal

The final outputs of the proposed procedure for each cluster containing a structural mode are:

- Mean value of natural frequency $\bar{f}_{Ci}$
- Mean value of modal damping ratio $\bar{\xi}_{Ci}$, calculated after the outliers removal
- Average mode shape vector $\bar{\boldsymbol{\Phi}}_{Ci}$
- MAC index calculated between the average mode shape vector $\bar{\boldsymbol{\Phi}}_{Ci}$ of the cluster $\mathbf{C}_i$ and the reference mode shape vector, iteratively estimated by the algorithm every 10 identifications
- Root Mean Square (RMS) of the acceleration values contained in the recorded time series.

The outputs of the automated procedure for modal parameters identification are then analyzed and correlated with the environmental factors (temperature and relative humidity) thanks to the Graphical User Interface (GUI), designed and developed for this purpose and presented in the following paragraph.

7.4 Graphical User Interface for dynamic systems

In parallel to the development of the GUI for static systems, presented in §6.4., it was decided to create a similar tool to display and post-process output data of the algorithm for automated OMA previously described. The interface is conceived to fit output data of any dynamic monitoring system through the input of reference environmental parameters and the features extracted by the automated subroutine.

The following paragraph describes the development of the GUI and the main features of the software.

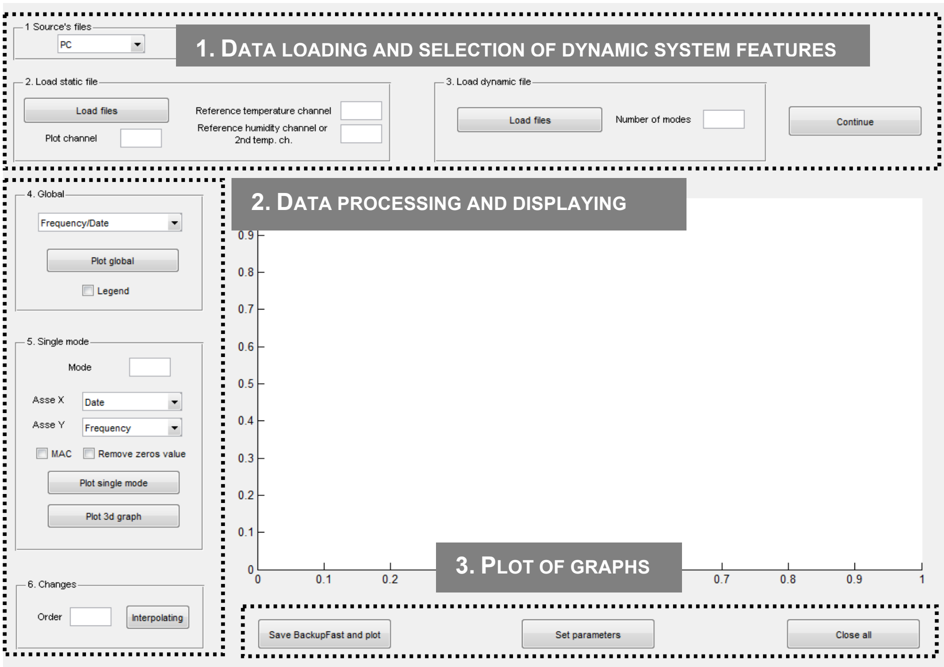
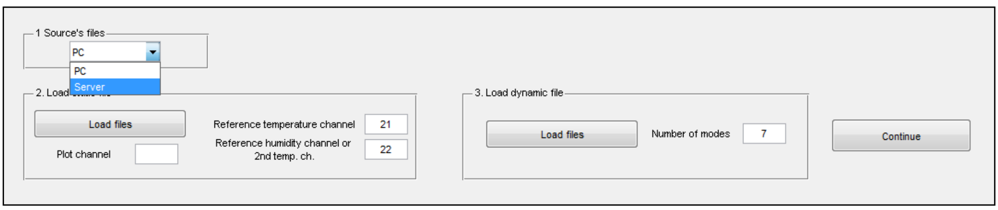
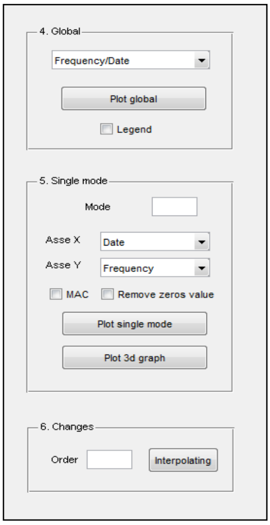


Fig. 7.8 - GUI for dynamic systems: post-processing and displaying of the outputs of the algorithm for automated OMA



When the program is launched the main window of the software opens (Fig. 7.8). The top bar of the window is devoted to the loading of the results obtained from the routine for automated OMA. Then dynamic properties of the analyzed structure in terms of number of structural modes need to be specified as well as the selection of one temperature and one relative humidity channel. The latter step allows cross correlating and plotting graphs of modal parameters in function of the environmental factors.

The user can upload dynamic data from either the local PC where the algorithm is running or directly from the central server through an FTP connection (in the latter case host, username and password have to be inserted).



The left bar of the GUI window can be exploited by the user to display dynamic monitoring results and plot graphs. It is possible to select a global display of the modal parameters of the structure or plot results by single mode. A large variety of options can be chosen, correlating modal parameters and environmental factors:

(i) Frequencies vs. time (Fig. 7.9)
(ii) Modal damping ratios vs. time
(iii) Frequencies vs. temperature (or humidity)
(iv) Damping ratios vs. temperature (or humidity)

It is also possible to display the MAC values (Fig. 7.10) calculated, as previously described, between the average mode shape vector $\bar{\mathbf{\Phi}}_{Ci}$ of the cluster $\mathbf{C}_i$ and the reference mode shape vector, iteratively estimated by the algorithm every 10 identifications. MAC indexes are represented in a second y-axis through histograms. Finally the algorithm, through a curve fitting procedure, calculate and plot a polynomial interpolating curve that fit data and give a first indication on the variation of the displayed parameters. The user can specify the order of the interpolating curve.

Graphs can be plotted and save using the commands in the bottom bar of the window.

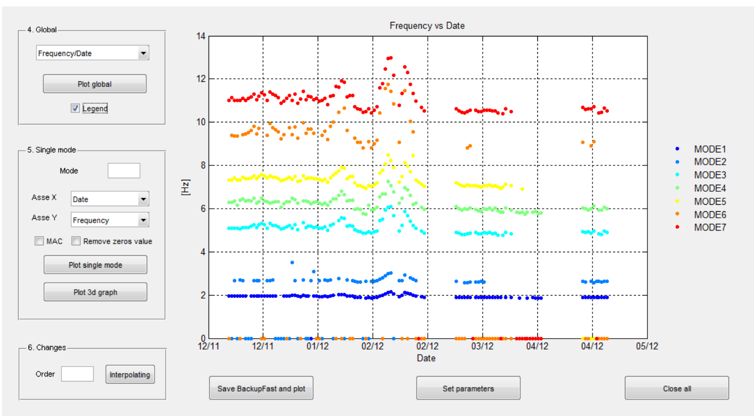


Fig. 7.9 - Display of results in the GUI for dynamic systems: global visualization of all structural modes in a frequency vs. time representation

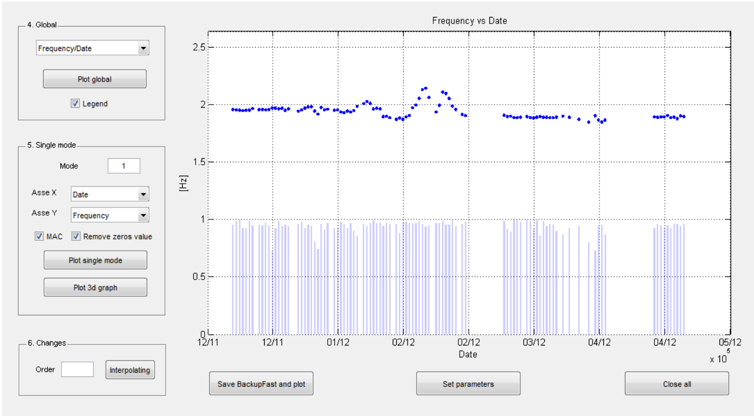


Fig. 7.10 - Display in the GUI of the frequency variation over time of a single mode and variation of the MAC index vs. time given by histograms

7.5 Application and validation of the automated algorithm for OMA

The methodology for the automatic extraction of modal parameters described in §7.3 was successfully implemented in a subroutine running on the central server of the University of Padova, that performs a real-time processing of data continuously collected by several monitoring systems (for more detail on the characteristics and layout of the active monitoring systems the reader has to refer to Chapter 5).

This paragraph is devoted to the validation of the developed algorithm in the application to real case studies, evaluating the performance of the subroutine and comparing the outputs of the algorithm with modal parameters manually extracted by means of a dedicated software for OMA, i.e. ARTeMIS Extractor Pro (SVS 2006).

Case studies considered within the validation process of the algorithm are those described in Chapter 5:

- Roman Arena of Verona
- Canisgnorio stone tomb
- Civic Tower
- Spanish Fortress

In the following section main results of the analyses will be presented and discussed.

7.5.1 Arena of Verona

The SHM system of the Arena of Verona, described in §5.2.3, is composed by 18 static sensors and 16 acceleration transducers implemented in a trigger-based monitoring. Time series recorded each 24 hours by the 6 accelerometers installed on the freestanding wall (i.e. the wing) of the monument represent the inputs of the automated subroutine. The analysis is focused in the frequency range 0-11 Hz, where 7 structural modes are expected (Fig. 7.11).

The developed automated algorithm for OMA was applied and tested on time series acquired by the system from 5 December 2011 (date of system activation) to 20 January 2013.

MODE 1	**MODE 2**	**MODE 3**	**MODE 4**	**MODE 5**	**MODE 6**	**MODE 7**
1,93 Hz	2,64 Hz	5,08 Hz	5,88 Hz	7,30 Hz	9,30 Hz	10,94 Hz

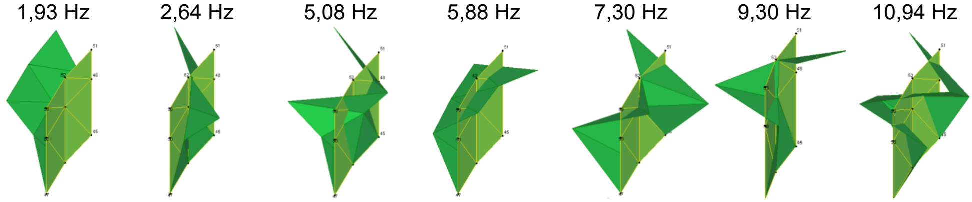


Fig. 7.11 - Natural frequencies and associated mode shapes of the wing of the Arena identified during preliminary AVT

The automatic extraction of modal parameters is based on two successive steps: (i) application of the cluster analysis to stabilization diagrams obtained from the system identification and (ii) comparison of the average mode shape vectors calculated for each cluster with a reference mode shape, using MAC index as selection criterion.

Tab. 7.1 - Results of automated modal analysis for the dynamic monitoring of the Arena during the period 05/12/2011 - 20/01/2013

Mode	Success Rate [%]	f_{mean} [Hz]	f_{std} [Hz]	ξ_{mean} [%]	ξ_{std} [%]	MAC_{mean} [%]	MAC_{min} [%]
1	86%	1,902	0,051	0,977	0,359	90,66	70,30
2	38%	2,621	0,097	0,903	0,326	89,10	70,55
3	94%	4,888	0,240	1,037	0,226	94,15	70,98
4	98%	6,016	0,232	5,247	1,527	96,62	74,25
5	99%	7,091	0,253	1,933	0,772	94,25	70,11
6	96%	9,028	0,575	0,961	0,365	86,93	70,01
7	26%	10,555	0,384	1,119	0,229	94,91	70,03

Tab. 7.1 summarizes the results of the application of the automated subroutine for the identification of the vibration modes of the structure.

The success rate parameter expresses the percentage of successful identifications of a certain mode and gives an indication on the performance of the algorithm during the validation period. It is possible to note that in general the automated algorithm presents a satisfying performance with success rate higher than 80% for almost all modes. Some problems have been recorded in the identification of mode #2 and #7 characterized by rather low success rates, 38% and 26% respectively. This is probably due to the fact that during the monitoring period a couple of accelerometers did not work properly: their positions coincide with the fundamental nodes (DOFs) associated to the eigenvectors of those vibration modes.

In the other columns mean values and standard deviation of the natural frequencies (f) and modal damping ratios (ξ) are presented.

The values of the standard deviation of the natural frequencies show that environmental conditions have a strong influence on these modal parameters, especially for modes #2-to-#7: the coefficient of variation, defined as $CV = f_{std}/f_{mean}$ vary in fact from 3,5% to 6,4%. Mode #1 seems to be less influenced by environmental factors with a CV of about 2,7%.

Looking at the mean values of modal damping ratio it is possible to note that mode #4 is characterized by high damping, confirming that dissipation mechanisms occur mainly where such mode shape present the maximum bending, i.e. just in correspondence of the anchorage of post-tensioned steel cables inserted along the massive pillars of the wing (for more details see §5.2.2 and §5.2.3.2)

The mean values of the MAC coefficients, reported in the last column of the table, are rather high (generally higher than 90%), meaning that mode shapes are not sensitive to environmental changes and that the procedure leads always to high-quality estimates. The minimum MAC indexes are close to 70%, as this was the selected lower bound to accept estimates for a certain reference mode.

Fig. 7.12 presents the evolution of the 7 identified natural frequencies during the analyzed period and Fig. 7.13 shows a zoom on two modes of the structure.

In general it is possible to note a slight trend of decrement of all natural frequencies with the approach of the hot season, demonstrating a certain correlation between variation of natural frequencies and environmental parameters. During the month of February 2012 it has been recorded a marked increment of frequency in correspondence to very cold days.

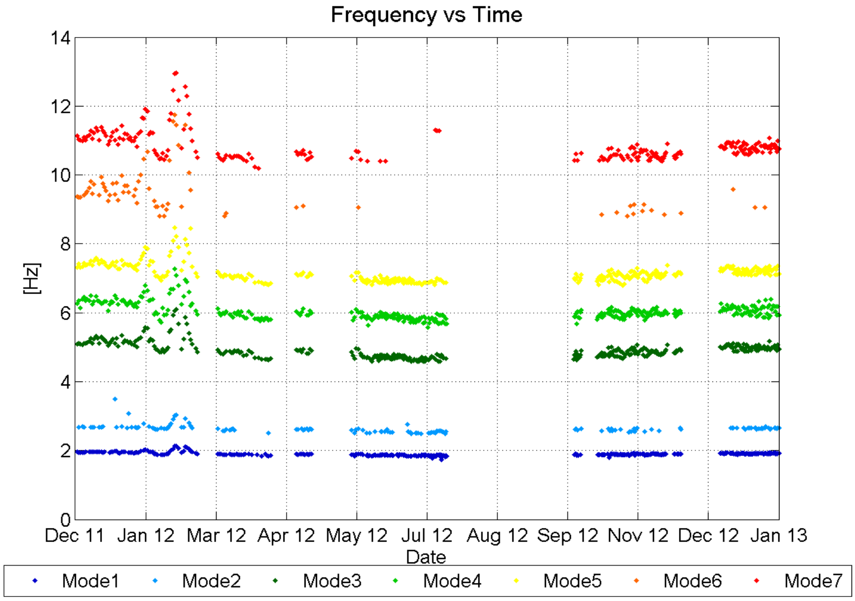
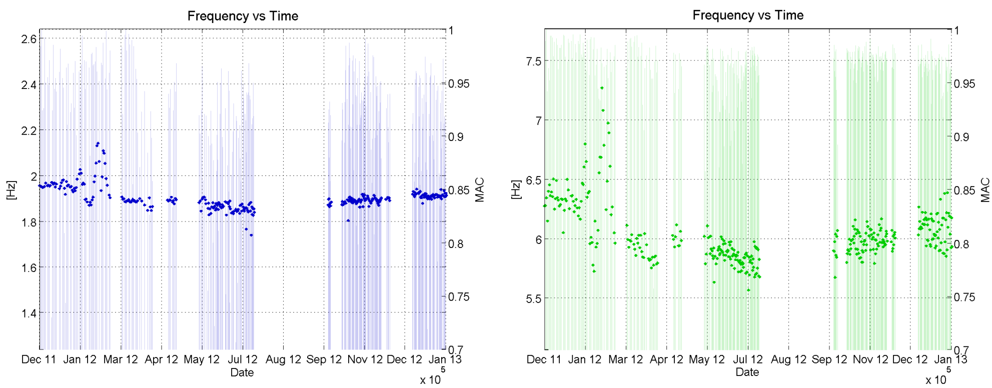


Fig. 7.12 - Evolution of the natural frequencies of the Arena's wing automatically identified by the algorithm in the period 05/12/11 - 31/12/12



a) *b)*

Fig. 7.13 - Evolution of the natural frequencies during the analyzed period: zoom on the two first out-of-plane bending modes: a) mode #1 and b) mode #4. On the second y-axis the evolution of MAC index is reported through a histogram representation.

This phenomenon can be noted also from the analysis of the recorded data in a frequency vs. temperature plot. Above a temperature of 2-3°C the behavior is quasi-perfectly linear and constant, whereas below there is a clear increment of frequency showing a rough bilinear behaviour.

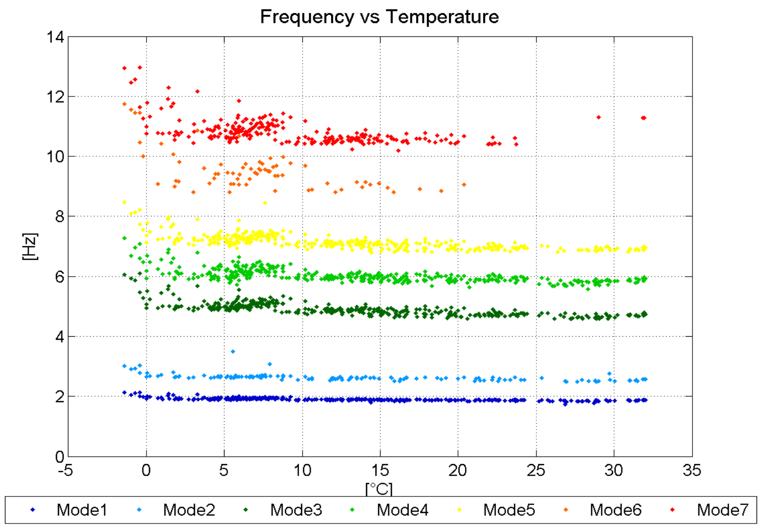


Fig. 7.14 - Natural frequency vs. temperature plot of the 7 modes of the structure

The variation of the modal damping ratios of the 7 identified structural modes in function of the temperature is reported in Fig. 7.15. It can be observed that damping present a high scatter due to the fact that its estimation is always affected by some uncertainty. However, variation of modal damping ratios of mode #4 and #5 (yellow and green dots respectively) seems to be somehow correlated with temperature, with a clear tendency to increase with low temperatures. Damping coefficients for the other modes are not influenced by temperature and keep constant around the mean value of 1%.

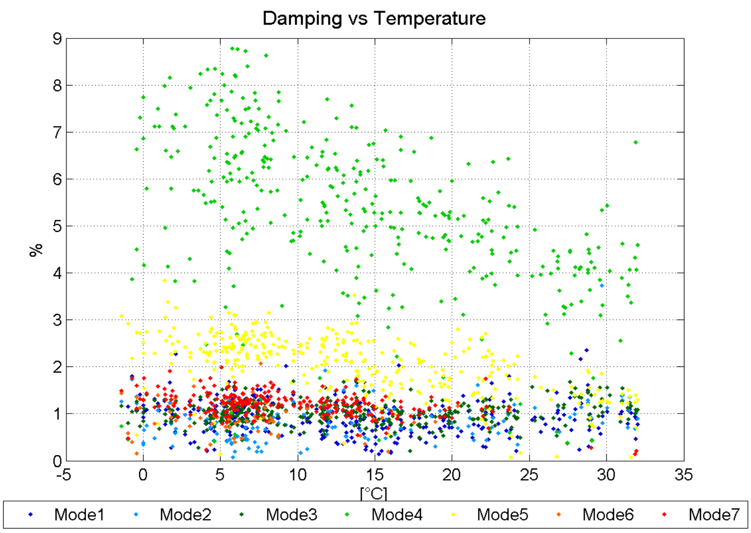


Fig. 7.15 - Modal damping ratios vs. temperature of the first 7 modes of the structure

7.5.2 Cansignorio stone tomb

The SHM system of the stone tomb of Cansignorio in Verona, described in §5.3.3, is composed by 2 static sensors and 6 acceleration transducers implemented in a trigger-based monitoring. Time series recorded each 24 hours by the 6 accelerometers represent the inputs of the automated subroutine. The analysis is focused in the frequency range 0-20 Hz, where 6 structural modes are expected (Fig. 7.16).

The developed automated algorithm for OMA was applied and tested on time series acquired by the system from 22 December 2006 (date of system activation) to 31 August 2012.

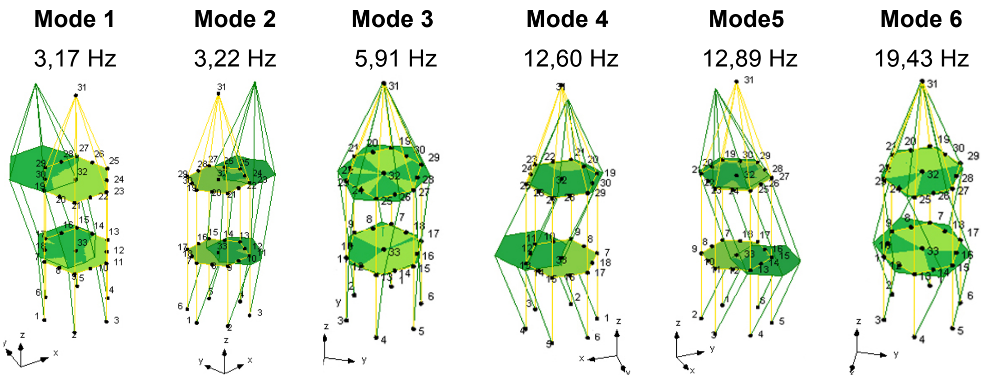


Fig. 7.16 - Natural frequencies and associated mode shapes of the Cansignorio stone tomb identified during preliminary AVT

Tab. 7.2 summarizes the results of the application of the automated subroutine for the identification of the vibration modes of the structure.

Tab. 7.2 - Results of automated modal analysis for the dynamic monitoring of the Cansignorio stone tomb during the period 22/12/2006 - 31/08/2012

Mode	Success Rate [%]	f_{mean} [Hz]	f_{std} [Hz]	ξ_{mean} [%]	ξ_{std} [%]	MAC_{mean} [%]	MAC_{min} [%]
1	97%	3,135	0,067	0,598	0,126	96,19	73,94
2	99%	3,186	0,068	0,634	0,167	95,60	75,09
3	99%	5,824	0,159	0,541	0,146	98,61	80,46
4	72%	12,549	0,284	0,945	0,200	91,92	70,54
5	83%	12,861	0,260	0,832	0,246	91,13	70,58
6	85%	19,415	0,444	0,494	0,138	96,89	73,91

The success rate of the developed algorithm during the validation period (data over almost six years of monitoring) in the identification of the first 6 modes of the structure is quantified. Excellent results have been obtained with success rates

higher than 83% for all modes a part from the 4th mode (72% success rate), which is likely not always well excited by ambient vibrations and thus its frequency estimates in all data sets are more difficult.
The reliability of the developed algorithm is confirmed together with its capability to clearly separate also very closely spaced modes. Mode #1 and #2 correspond in fact to the two first order bending modes along orthogonal directions (given the high symmetry of the structure) and are comprised in a range of about 0,05 Hz. The algorithm is able to separate and identify perfectly both modes with a success rate of almost 100% (97% and 99% respectively).
In the other columns mean values and standard deviation of the natural frequencies (f) and modal damping ratios (ξ) are presented. The values of the standard deviation of the natural frequencies show that environmental conditions have a certain influence on each identified mode (coefficient of variation CV from 2% to 3% for all modes).
The mean values of modal damping ratios are almost uniform for all structural modes, ranging from 0,5% to 0,9%. However also in this case difficulties and uncertainties in the estimation of this modal parameter are present. Damping estimates are characterized by a high scatter (CV varying from 21% (for mode #1) to 29% (for mode #5)).
The mean values of the MAC coefficients, reported in the last column of the table, are considerably high (always more than 90%), meaning that mode shapes are not sensitive to environmental changes and that the procedure leads always to high-quality estimates. The minimum MAC indexes are close to 70%, as this was the selected lower bound to accept estimates for a certain reference mode.
The developed algorithm was then validated through a direct comparison of the outputs in terms of natural frequency with the same modal parameters extracted manually by means of the FDD identification method implemented in the commercial software ARTeMIS Extractor Pro. Fig. 7.17 shows the comparison of the performed analysis for the following modes:

a) bending mode #1 around the mean frequency of 3,135 Hz and
b) bending mode #2 around the mean frequency of 3,186 Hz
c) torsional mode #3 around the mean frequency of 5,824 Hz
d) torsional mode #6 around the mean frequency of 19,415 Hz

Red dots indicate the outputs of the automated subroutine, whereas blue dots represents the results of manual identifications performed through FDD method in the period 22/12/2006 - 31/08/2011. It is possible to note the quasi-perfect match of results, demonstrating the reliability and effectiveness of the algorithm.

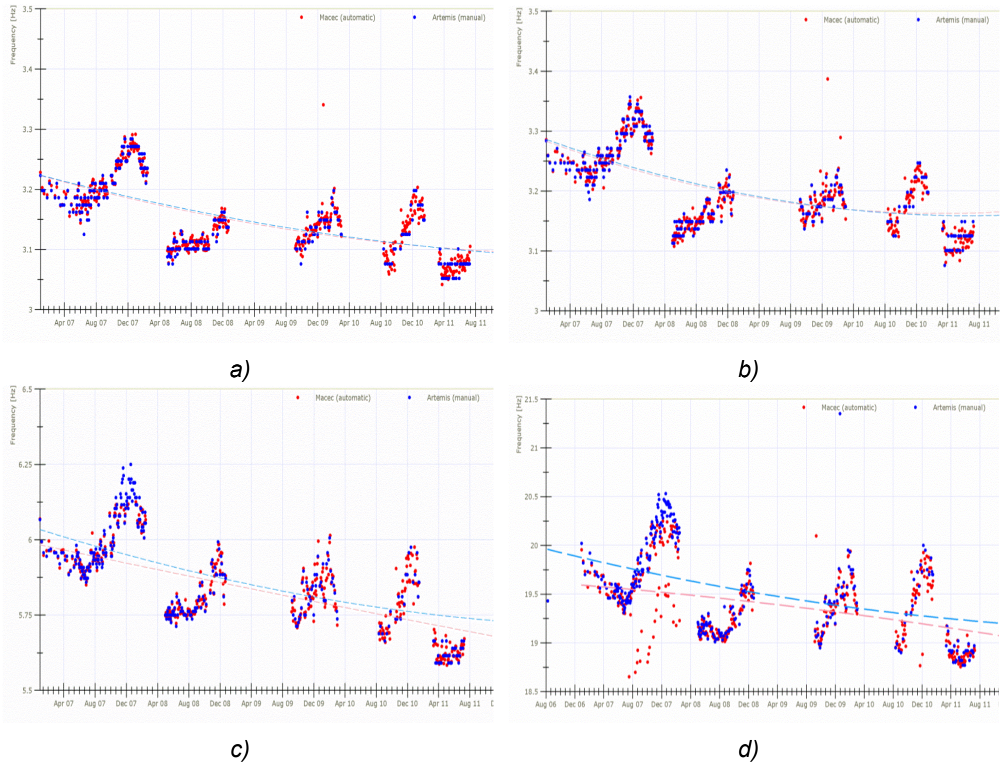


Fig. 7.17 - Comparison charts between outputs of the automated algorithm (in red) and manual identification of natural frequencies through FDD method (in blue) for two bending modes (a and b) and two torsional modes (c and d)

Fig. 7.18 shows the evolution of the 6 identified natural frequencies during the analyzed period. In spite of the availability of fragmented data due to some shutdowns and malfunctioning of the system during the monitoring period, it is possible to provide some conclusions based on the variation of natural frequencies over almost six years. It is necessary to point out that the system was installed before the execution of strengthening interventions performed in the period 2006-2008 and kept active during their implementations and for four years after their conclusion.

It can be noted, after the conclusion of the interventions in March 2008 (first dashed vertical line in the graph of Fig. 7.18) and the removal of the scaffolding that incorporated the whole structure, a significant decrement frequencies. This variation is recorded for the frequencies of all six vibration modes of the structure (Fig. 7.19 shows the frequency variation for the first bending mode a) and the first torsional mode b)). Since no damages were recorded on the structure, the frequency shift can be explained by the change of the boundary environmental conditions, in terms of solar radiation, ventilation and thus temperature and relative humidity, of the structure after the removal of the scaffolding at the end of the intervention works. A second, even if slighter decrement of natural frequencies is noted at the end of 2010 (second vertical dashed line in the graph of Fig. 7.18). Also in this case no damages are recorded on the structure and thus it is necessary to evaluate and investigate various hypotheses before reaching a rigorous explanation.

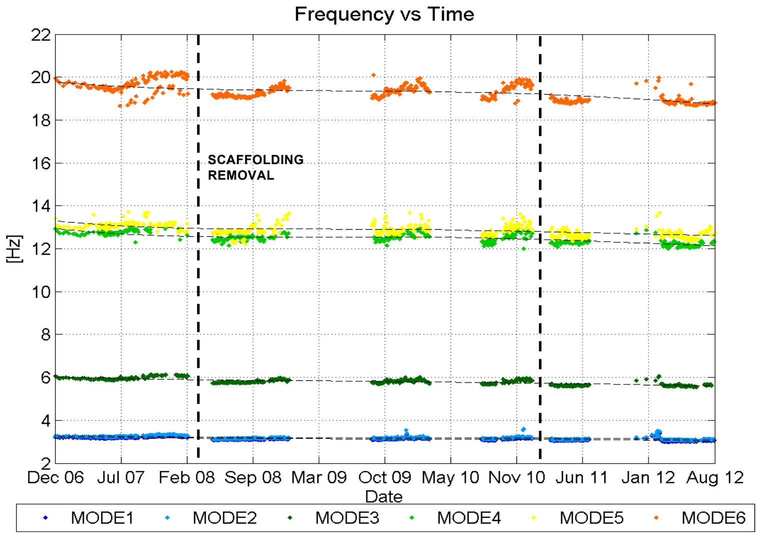
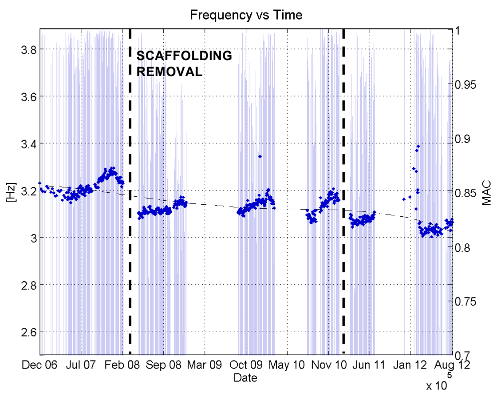
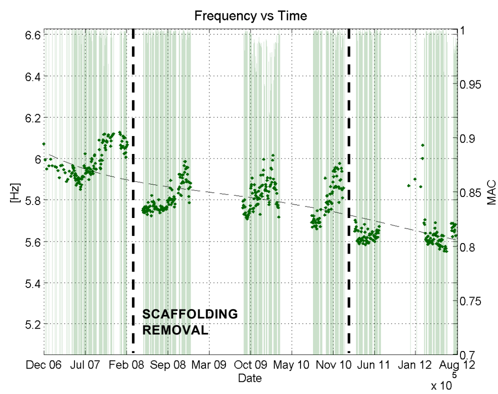


Fig. 7.18 - Evolution of the natural frequencies of the stone tomb automatically identified by the algorithm in the period 22/12/2006 - 31/08/2012



a)



b)

Fig. 7.19 - Zoom on two structural modes: a) mode #1: first bending and b) mode #4: first torsional. It can be noted a marked decrement of frequency in correspondence of the two vertical dashed lines.

Fig. 7.20 shows the variation of natural frequencies in function of the recorded external temperature. Thanks to the zoom on the first two closely spaced bending modes of the structure it is possible to appreciate a clear dependency of frequency from environmental parameters. The scattered values of Fig. 7.20b and the distinction of two groups of frequency values can be justified by the removal of the scaffolding after the interventions and the presence of two completely different environmental conditions during the analyzed monitoring period.

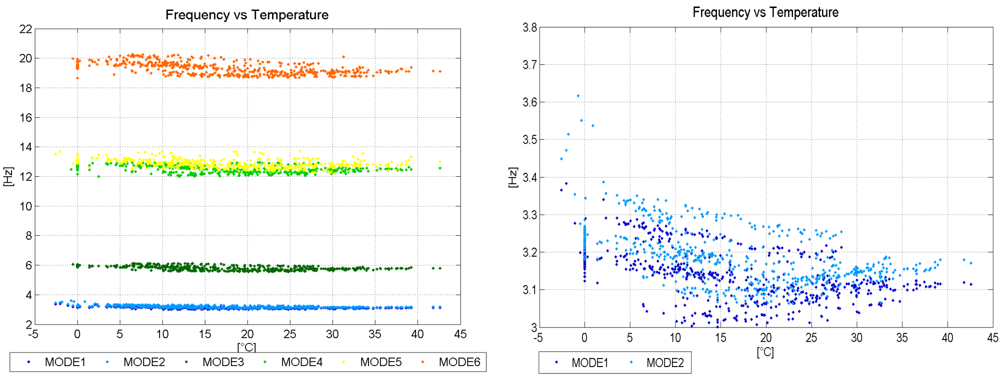


Fig. 7.20 - a) Natural frequency vs. temperature plot of the first six structural modes; b) zoom on the first two bending modes.

7.5.3 Civic tower

The SHM system of the Civic Tower in l'Aquila, described in §5.5.4, is composed by 18 static sensors and 8 acceleration transducers implemented in continuous monitoring. Time series, continuously recorded by 8 accelerometers, represent the inputs of the automated subroutine. The analysis is focused in the frequency range 0-7 Hz, where 7 structural modes are expected (Fig. 7.21).

The developed automated algorithm for OMA was applied and tested on time series acquired by the system from 22 July 2010 (date of system activation) to 9 January 2013.

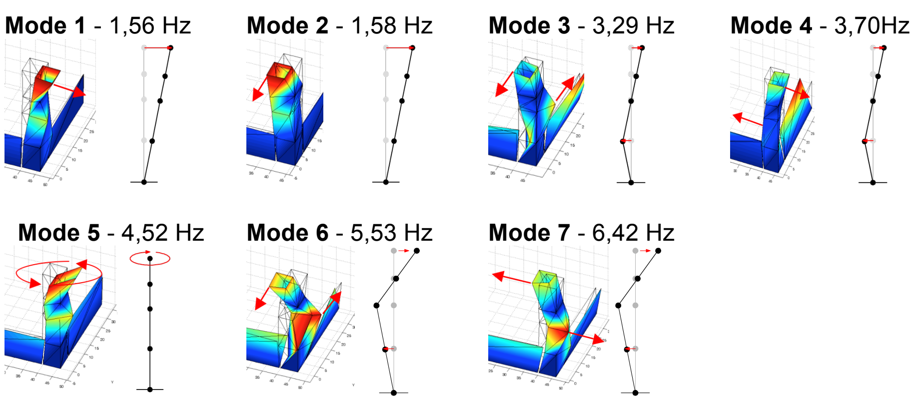


Fig. 7.21 - Natural frequencies and associated mode shapes of the Civic tower

Tab. 7.3 summarizes the results of the application of the automated subroutine for the identification of the vibration modes of the structure.

Tab. 7.3 - Results of automated modal analysis for the dynamic monitoring of the Civic tower during the period 22/07/2010 - 09/01/2013

Mode	Success Rate [%]	f_{mean} [Hz]	f_{std} [Hz]	ξ_{mean} [%]	ξ_{std} [%]	MAC_{mean} [%]	MAC_{min} [%]
1	80%	1,624	0,068	0,58	0,24	89,61	70,16
2	90%	1,640	0,080	0,56	0,20	90,30	70,16
3	81%	3,148	0,083	1,07	0,21	92,41	77,43
4	69%	3,669	0,177	0,94	0,25	89,46	71,38
5	100%	4,774	0,231	1,05	0,39	95,53	70,86
6	96%	5,579	0,158	1,43	0,39	94,05	71,55
7	52%	6,279	0,197	1,80	0,65	89,09	70,22

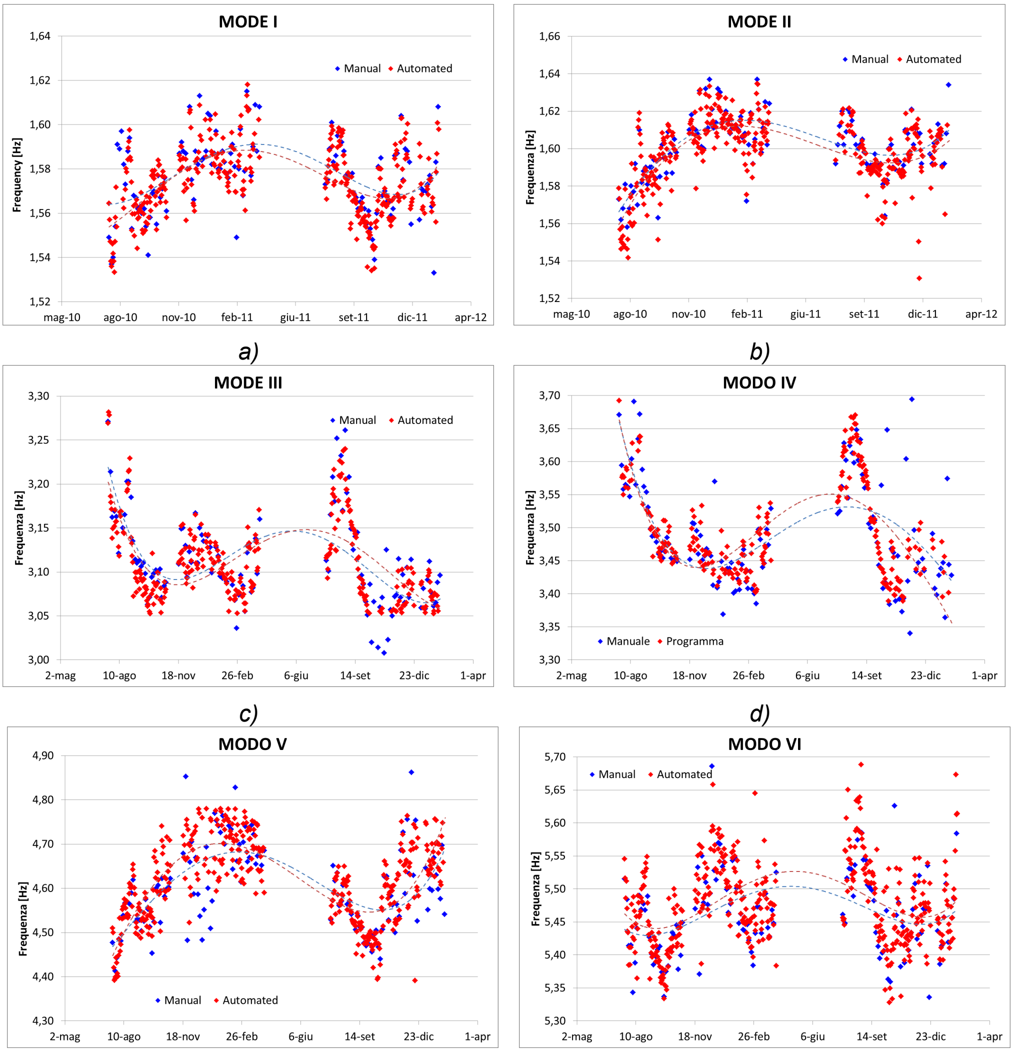
The developed automated algorithm proved to be rather efficient in the identification of the first 7 vibration modes of the structure. Very good results have been obtained with success rates higher than 80% for all modes a part from modes #4 (69% success rate) and #7 (52%). The lower success rate can be explained by the fact that those modes, that corresponds to the second and third order bending modes along the same E-W direction, are not always well excited by ambient vibrations. It is necessary to point out that the structure is characterized by high levels of damage induced by the earthquake and thus the dynamic response is generally more difficult to identify and modal parameters estimations less precise.

However the reliability of the developed algorithm is confirmed together with its capability to clearly separate also very closely spaced modes. Mode #1 and #2 correspond in fact to the two first order bending modes along orthogonal directions (given the high symmetry of the structure) and are comprised in a range of about 0,02 Hz. The algorithm is able to separate and identify both modes with a success rate of almost 80% and 90% respectively.

In the other columns mean values and standard deviation of the natural frequencies (f) and modal damping ratios (ξ) are presented.

A more detailed analysis on the variation of modal parameters over the entire monitoring period will be presented and deepened in the next chapter, where statistical black box models starting from the identified natural frequencies and temperature measurements will be constructed (see §8.2.2)

The developed algorithm was then validated through a direct comparison of the outputs in terms of natural frequency with the same modal parameters extracted manually by means of the FDD identification method implemented in the commercial software ARTeMIS Extractor Pro.



e) *f)*

Fig. 7.22 - Comparison charts between outputs of the automated algorithm (in red) and manual identification of natural frequencies through FDD techniques for the two first order bending modes (a and b) and two second order bending modes (c and d), the torsional mode (e) and the third order bending mode (f).

Fig. 7.22 shows the comparison of the performed analysis for the first six structural modes of the tower.

Red dots indicate the outputs of the automated subroutine, whereas blue dots represents the results of manual identifications performed through FDD method in the period 22/07/2010 - 09/01/2013. Manual identification are performed once a week during the validation period, whereas the automatic modal analysis algorithm is launched, on a daily basis, on the whole recorded time series. It is possible to note also in this case the quasi-perfect match of results, demonstrating the reliability and effectiveness of the algorithm.

Fig. 7.23 shows the evolution of the 7 identified natural frequencies during the analyzed period, whereas Fig. 7.24 shows the variation of natural frequencies in function of the recorded external temperature. As clarified before a more exhaustive representation of the automated subroutine outcomes will be provided in the next chapter.

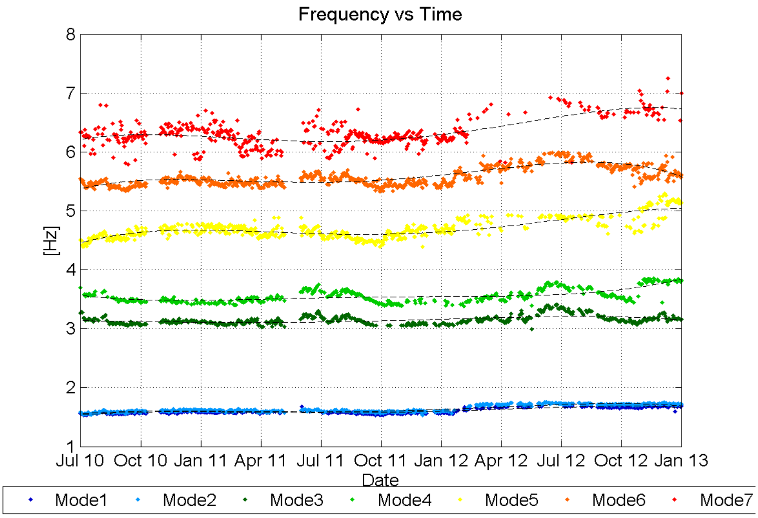


Fig. 7.23 - Natural frequencies variation of the 7 structural modes of the Civic tower

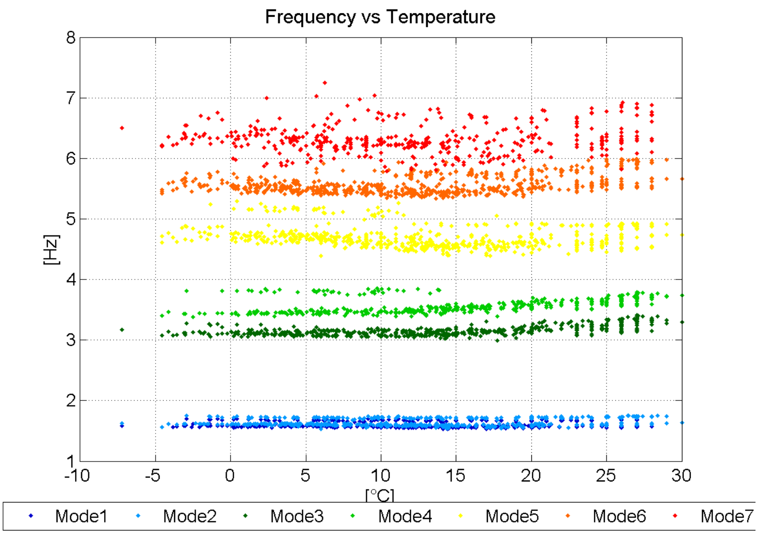


Fig. 7.24 - Natural frequency vs. temperature plot of the 7 modes of the Civic tower

7.5.4 Spanish fortress

The SHM system of the Spanish Fortress in l'Aquila, described in §5.6.4, is composed by 8 acceleration transducers implemented in trigger-based monitoring. Time series recorded each 24 hours by the 8 accelerometers represent the inputs of the automated subroutine. The analysis is focused in the frequency range 0-9 Hz, where 5 structural modes are expected (Fig. 7.25).
The developed automated algorithm for OMA was applied and tested on time series acquired by the system from 20 December 2009 (date of system activation) to 22 January 2013.

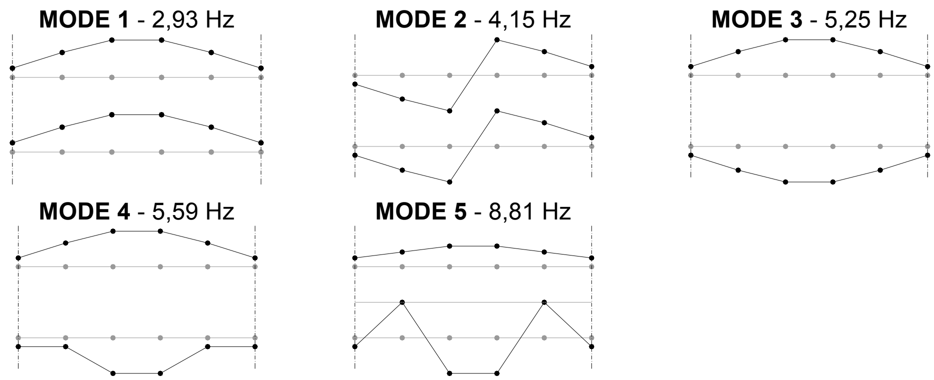


Fig. 7.25 - Natural frequencies and associated mode shapes of the Spanish Fortress: top view scheme

Tab. 7.4 summarizes the results of the application of the automated subroutine for the identification of the vibration modes of the structure.

Tab. 7.4 - Results of automated modal analysis for the dynamic monitoring of the Spanish Fortress during the period 20/12/2009 - 22/01/2013

Mode	Success Rate [%]	f_{mean} [Hz]	f_{std} [Hz]	ξ_{mean} [%]	ξ_{std} [%]	MAC_{mean} [%]	MAC_{min} [%]
1	100%	2,990	0,094	1,101	0,325	97,85	72,04
2	80%	4,289	0,171	0,810	0,339	90,09	70,88
3	91%	5,456	0,285	1,230	0,517	87,01	70,04
4	99%	5,847	0,143	1,163	0,410	94,64	71,42
5	97%	8,694	0,523	0,742	0,318	91,74	70,01

The success rate of the developed algorithm during the validation period (data over more than 3 years of monitoring) in the identification of the first 5 modes of the structure is quantified. Excellent results have been obtained with success rates higher than 90% for all modes a part from the 2nd mode (80% success rate), which is likely not well excited by ambient vibrations and thus its estimation in all data sets is more difficult.

In the other columns, as always, mean values and standard deviation of the natural frequencies (f) and modal damping ratios (ξ) are presented.

As for the Civic tower, a more detailed analysis on the variation of modal parameters over the entire monitoring period will be presented and deepened in the next chapter. Just a few comments on damping estimation are reported here. The mean values of modal damping ratios are almost uniform for all structural modes around 1% (varying from 0,7% of the 5th mode to 1,1% of the 1st and 4th mode). However also in this case difficulties and uncertainties in the estimation of this modal parameter are present. Damping estimates are characterized by a high scatter (CV coefficient varies from 29% (for mode #1) to 43% (for mode #5)).

The developed algorithm was then validated through a direct comparison of the outputs in terms of natural frequency with the same modal parameters extracted manually by means of the FDD identification method implemented in the commercial software ARTeMIS Extractor Pro.

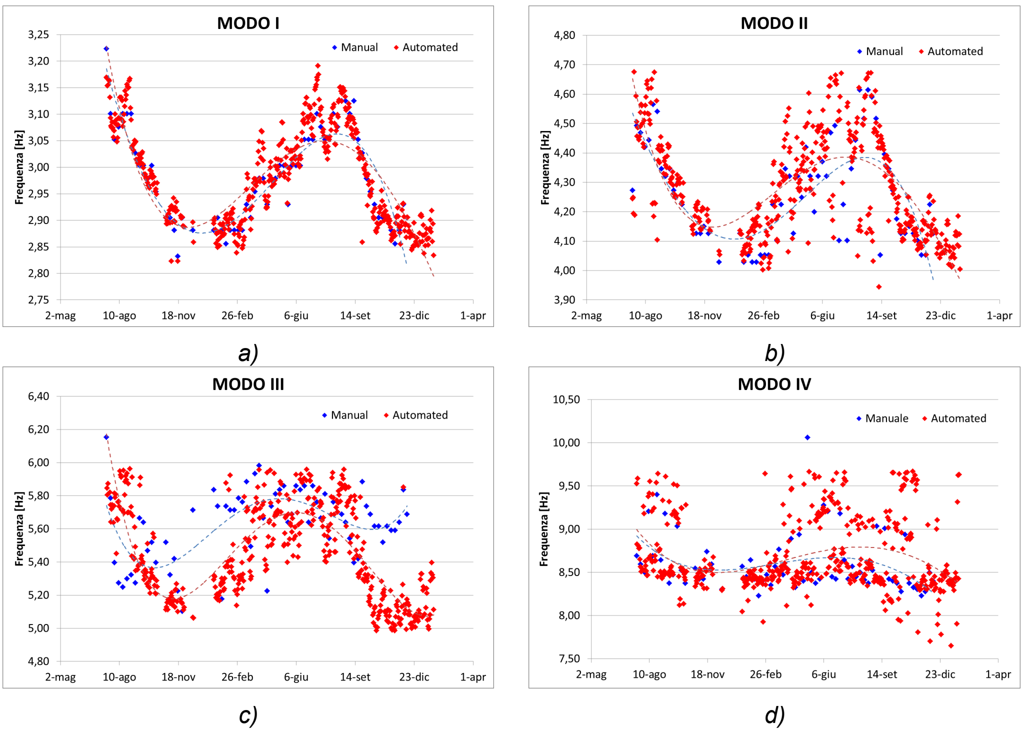


Fig. 7.26 - Comparison charts between outputs of the automated algorithm (in red) and manual identification of natural frequencies through FDD techniques for the two first order bending modes (a and b) and two second order bending modes (c and d), the torsional mode (e) and the third order bending mode (f).

Fig. 7.26 shows the comparison of the performed analysis for the first four structural modes of the fortress.

Red dots indicate the outputs of the automated subroutine, whereas blue dots represents the results of manual identifications performed through FDD method in the period 19/07/2010 - 25/01/2011. Manual identification are performed once a week during the validation period, whereas the automatic modal analysis algorithm is launched, on a daily basis, on the whole recorded time series.

It is possible to note also in this case the a very good match of results, demonstrating the reliability and effectiveness of the algorithm.

Fig. 7.27 shows the evolution of the 5 identified natural frequencies during the analyzed period. The fifth frequency is characterized by scattered values, meaning that its estimation presents some uncertainties due to the fact that most likely this frequency is not sufficiently well-excited by ambient vibrations and its tracking during the monitoring period becomes much more complicated. Similar difficulties have been recorded also during manual identifications performed by an expert user.

It is possible to note an almost perfect cyclic variation of natural frequencies during the monitoring period that demonstrated the strict correlation between frequencies and environmental parameters.

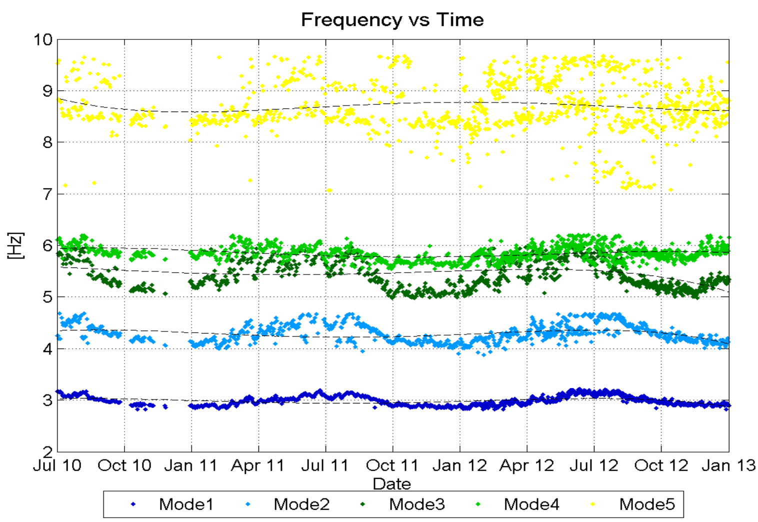


Fig. 7.27 - Evolution of the first natural frequencies of the Spanish Fortress automatically identified by the algorithm in the period 20/12/2009 - 22/01/2013

This correlation is better visible thanks to the natural frequencies vs. temperature plot, reported in

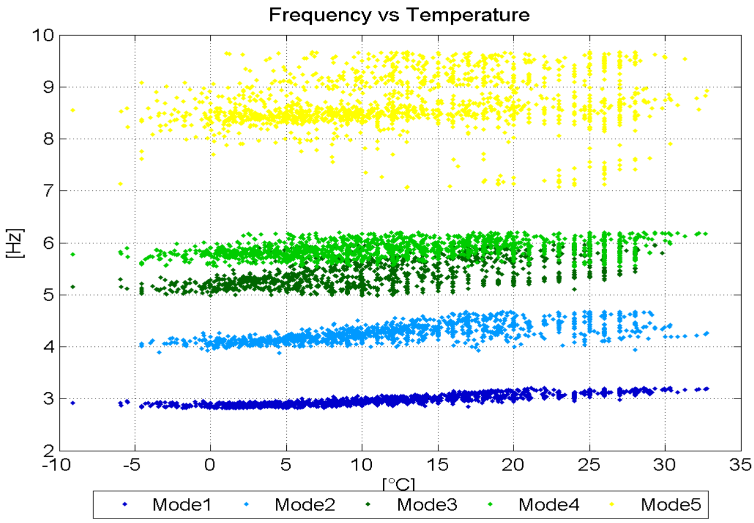


Fig. 7.28 - Natural frequency vs. temperature plot of the 5 modes of the Spanish fortress

Based on these observations in the next chapter some statistical models will be developed and described and a tentative application of damage detection algorithms on the extracted modal parameters will be presented.

7.6 Conclusions

This chapter reports the development of automated algorithms for modal parameters identification. A fully automated subroutine runs on the central server of the University of Padova, providing almost real time information on the dynamic response of the monitored structures. The algorithms are devoted to the processing of recorded time series and the automatic extraction of modal parameters in terms of natural frequencies, mode shapes and damping ratios. The proposed methodology exploits the parametric frequency domain identification method p-LSCF (poly-reference Least Square Complex Frequency Domain), implemented in the MATLAB toolbox MACEC 3.2 and it is then complemented by a new procedure developed for the automatic analysis of the stabilization diagrams, based on a hierarchical clustering algorithm. A specific part of the algorithm is devoted to the refinement of the modal damping estimation, which constitutes a challenging issue for OMA since it presents always high scattered values, difficult to assess.
The final outputs of the proposed procedure for each recorded time series are:

- Mean value of natural frequency $\bar{f}_{Ci}$
- Mean value of modal damping ratio $\bar{\xi}_{Ci}$, calculated after the outliers removal

- Average mode shape vector $\bar{\mathbf{\Phi}}_{Ci}$
- MAC index calculated between the average mode shape vector $\bar{\mathbf{\Phi}}_{Ci}$ of the cluster $\mathbf{C}_i$ and the reference mode shape vector, iteratively estimated by the algorithm every 10 identifications
- Root Mean Square (RMS) of the acceleration values contained in the recorded time series.

Then a Graphical User Interface (GUI) was developed in order to plot the extracted features, create graphs and correlate the automatically identified modal parameters with environmental factors.

Finally applications and validations of the automated algorithm for OMA are presented, showing its performance and evaluating its effectiveness on selected case studies: (i) Arena of Verona, (ii) Cansignorio stone tomb, (iii) Civic tower, (iv) Spanish fortress.

For each application an index showing the success rate of the algorithm during the automatic extraction of modal parameters is reported, together with some statistical results of the identification and plots of the principle outcomes, combined with the measurements of ambient variables.

8 SHM FOR THE ASSESSMENT OF CH BUILDINGS

8.1 Introduction

This chapter develops and defines some procedures to interpret, post-process and exploit the results of monitoring in the general framework of increasing the knowledge level and assess the structural conditions of CH buildings.
SHM outcomes of the case studies presented in Chapter 5 are used here on the one hand to build advanced statistical models and implement damage detection algorithms following a data-driven approach, and on the other hand to update and validate reference numerical models in a model-driven approach. The analysis of monitoring results takes into account different but interconnected conditions: (i) control the structural behavior of the monitored building under operational states; (ii) develop and calibrate reference behavioral models; (iii) study and characterize the structural response in case of exceptional events.

8.2 SHM under operational conditions

The application of OMA techniques into continuous or trigger-based dynamic monitoring on the one hand and the control of static parameters, crack and damage patterns through static monitoring on the other hand allows characterizing the structural response of a building subjected to normal or operational conditions.
Those parameters are strictly correlated to the change of environmental factors and loading conditions, whose control and quantification become essential before any attempt to identify the presence of damage.
In fact once the environmental effects are filtered out from recorded data and extracted features, it is possible to accurately decompose the measurements into their reversible and irreversible components, the latter being associated to active deteriorating processes.
Different methods can be applied to remove the effects of environmental or operational factors on the extracted parameters (mainly natural frequencies for dynamic monitoring, evolution of crack opening, inclination, etc. for static data).

One approach is to create specific models able to capture and represent the physical phenomena behind the parameter changes. Although this procedure is very interesting in order to explain the principal factors that may influence the observed static or dynamic behavior of the system, it is practically not applicable in the context of SHM because of the extreme complexity of such models. Moreover even if these models were constructed and implemented, in any case some effects would not be correctly represented.

In order to overcome these limitations a second approach can be applied using regression analysis and black-box models, whose parameters are tuned exploiting a large number of observations to establish relations between extracted or recorded parameters (e.g. natural frequencies, crack opening) and the factors that may influence them.

8.2.1 Modeling environmental effects through black-box models

Regression analysis is able to establish relationships between the observed environmental factors and the estimated natural frequencies or other static monitored parameters. It analyses the relation between a single dependent variable and several independent (predictor) variables. This relationship is described through a statistical model that can be exploited, in an initial phase, to understand the influence of each predictor (input of the model) on the dependent variable (output of the model) and then, to predict future values of the response when only the predictors are known. In the context of SHM, a first set of data (usually referred as estimation phase) is used to construct the model and afterwards, the developed model is used to predict the outputs (natural frequencies, crack opening, etc.) taking into account the measured independent variables (temperature, relative humidity, loading conditions, etc.)

The predicted outputs (usually referred as validation phase) are subsequently compared with the values directly estimated from acquired time series or recorded static data, calculating the residuals between actual data and model predictions. In this way it is possible to remove the environmental factors and provide a preliminary judgement on the health conditions of the structure.

In the framework of SHM, where several parameters are continuously monitored, a regression model for each one has to be built. Data over at least one year have to be collected in order to construct a reliable statistical model, able to characterize the influence of environmental factors on the monitored parameters, considering a large range of variation, with data from summer and winter periods.

Regression models can be classified into static or dynamic. Static models explain the values of the output variables at a certain time instant $\boldsymbol{y}_k = y(t_k)$ using only the observations of the predictors associated with the same time instant $\boldsymbol{u}_k = u(t_k)$. Dynamic models assume that the values of the dependent variables at a certain time instant can also be influenced by the values of the model inputs at previous time instants (Magalhães *et al.* 2011). These models have the advantage to be able to take into account the thermal inertia of the structure, including some dynamics: the current output and input are related to outputs and inputs at previous time instants.

One example of dynamic model is the so called ARX model (Ljung 1999) that comprehends an Auto-Regressive output and an eXogeneous input part, already described in §2.3.3.2. This is ideal for representing monitored parameters when they depend (linearly) on the rate of change or trend in temperature as well as the present temperature.

As previously reported in §2.3.3.2 the multivariable ARX model with n inputs u and one output y is presented by:

$$A_q y_k = B_q u_{k-nk}^{env} + e_k \tag{8.1}$$

where A_q is a scalar with the delay operator q^{-1}, B_q is a matrix $1{\times}n$, and e_k is the unknown residual, a white noise term indicating that the input-output relation is not perfect. The ARX model is characterized by 3 numbers: n_a, the auto-regressive order, n_b the exogenous order and n_k the pure time delay between input and output. The coefficients of operator polynomials A_q and B_q can be estimated with the simple linear least square method. ARX models are commonly indicated by the orders n_a, n_b and n_k, e.g. ARX142 with $[n_a, n_b, n_k] = [1,4,2]$.

In the next section the application of ARX model to the features extracted from SHM systems of two selected CH structures will be presented, following a similar approach used by Peeters & De Roeck (2000) and Ramos (2007), with the appropriate modifications for the present case studies.

8.2.2 Applications to l'Aquila case studies

In the present research ARX models with single input and single outputs (SISO models) is applied to simulate and predict the response of some parameters related to the SHM of two case studies in l'Aquila. This choice was made considering that the monitoring systems of both the Civic tower and the Spanish fortress have been active for 2,5 and 3 years respectively, without any significant system malfunctioning and loss of data. Such monitoring periods are long enough to collect data related to a large range of variation. In particular for both case studies features extracted from the first year of monitoring are implemented to establish the best statistical model of the considered parameters, i.e. the model that better described the influence of the independent inputs (environmental factors) on the recorded outputs. Afterwards, in the second step, the constructed model is exploited to predict the outputs in the following years, comparing the predicted response with the actual measured parameters. The final phase of the proposed procedure consists in the application of a residual analysis, able to detect the outliers and provide the identification of possible damages.

8.2.2.1 Spanish fortress

The monitoring system of the Spanish Fortress is essentially composed by 8 acceleration transducers that collect time series each 24 hours as described in §5.6.4. Data recorded by the system are continuously transmitted to the central server of the University of Padova and automatically processed at arrival by the algorithm for automated OMA. Feature extracted by the subroutine from time histories are natural frequencies, modal damping ratios and mode shapes. The dynamic system has been working since the 20th of December 2009. Therefore, a database with the variation of the fortress modal parameters identified by the automated algorithm during more than 3 years is available. In the present chapter, the period from 20/12/2009 to 22/01/2013 is analyzed. In §7.5.4 the evolution of the fortress first 5 natural frequencies was presented and briefly discussed. Here a more detailed analysis of the variation of natural frequencies and their strict correlation with environmental parameters is reported.

Fig. 8.1 shows a zoom on the evolution over 3 years of monitoring of the first 4 natural frequencies and the dependency on environmental factors through frequency vs. temperature plots.

It is possible to note that all the analyzed frequencies are strictly correlated with temperature changes: in particular frequencies decrease during winter and cold periods and increase during summer. It is relevant to observe that changes are significant: 0,4 Hz for mode #1, 0,8 Hz for mode #2, 1 Hz for mode #3 and 0,7 Hz for mode #4. The comparison of the annual evolution of modal parameters with the annual variation of temperature demonstrates that the temperature is the major factor that influence the annual fluctuations.

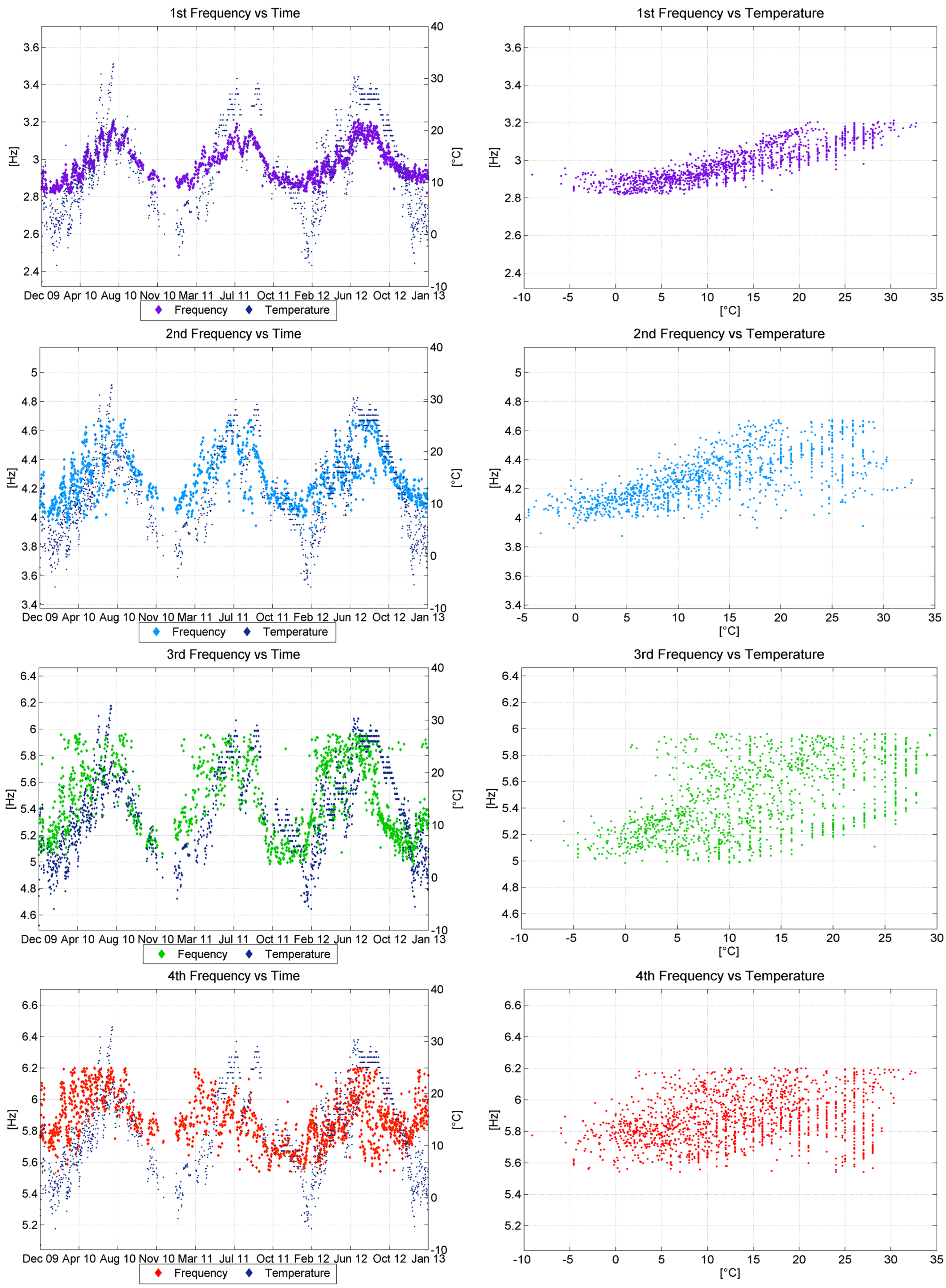


Fig. 8.1 - First four vibration modes of the Spanish Fortress. Left column: frequency vs. time plots. Right column: frequency vs. temperature plots.

The statistical results of the natural frequencies are presented in Tab. 8.1. The average values of the first five natural frequencies together with the frequency change, standard deviation and coefficient of variation CV (i.e. relative standard deviation) are reported.

Tab. 8.1 - Statistical results of the first 5 natural frequencies of the Spanish Fortress

Mode	f_{max} [Hz]	f_{min} [Hz]	f_{mean} [Hz]	f_{change} [%]	f_{std} [Hz]	f_{CV} [%]
1	3,213	2,819	2,982	13,98	0,097	3,26
2	4,675	3,876	4,274	20,60	0,172	4,02
3	5,963	4,985	5,457	19,63	0,278	5,10
4	6,201	5,542	5,872	11,91	0,153	2,61
5	9,668	7,069	8,963	36,77	0,505	5,80

Starting from these considerations, it is possible to develop robust statistical models able to remove (or at least minimize) the effects of environmental factors on the natural frequencies of the fortress. This step is fundamental before any attempt to apply damage identification algorithms since environmental or operational conditions might mask the change of frequency possibly induced by an actual damaging process active on the structure.

In the present application, as continuous temperature measurements are available, ARX models are tested. Data collected during the first year of monitoring (from 20/12/2009 to 20/12/2010) are used to build the regression models and then data collected during the remaining 2 years of monitoring (from 21/12/2010 to 22/01/2013) were used to validate the quality of the forecasts provided by the models, trying to detect through residual analysis any changes in the response, possibly linked to damage.

In this procedure it is assumed that during the first period (estimation phase) the damage pattern induced by the earthquake to the building is stable, also thanks to the provisional strengthening interventions performed by fireman during the emergency phase. The idea is to validate the long-term effectiveness of the performed interventions.

In a first step the predictors of the ARX models have to be selected. Since the monitoring system is not equipped with environmental sensors, temperature records are collected from monitoring systems installed on near buildings, i.e. the Civic Tower (with 6 thermal sensors from $T1$ to $T6$ controlling both air and walls temperatures) and the Church of S. Marco (with an integrated temperature/relative humidity sensor $T7$) in the historic city center of l'Aquila.

A correlation analysis is performed with the objective of identifying the temperature time series presenting higher correlation coefficients (Hair *et al.* 1998) with the time series containing the evolution of the automatically identified natural frequencies.

Through this type of analysis it is possible to study the correlation between environmental effects $\boldsymbol{x}_k$ (in this case temperatures) and measured outputs $\boldsymbol{y}_k$ (in this case natural frequencies), calculating for each temperature-frequency pair the so called correlation coefficient $\boldsymbol{r}_{xy}$ which represents the normalized measure of the strength of linear relationship between variables, given by:

$$\hat{\boldsymbol{r}}_{xy} = \frac{cov(\boldsymbol{x}_k, \boldsymbol{y}_k)}{\sigma_x \sigma_y} \tag{8.2}$$

where $cov(\boldsymbol{x}_k, \boldsymbol{y}_k)$ is the estimated covariance:

$$cov(\boldsymbol{x}_k, \boldsymbol{y}_k) = \frac{1}{N-1} \sum_{k=1}^{N} (x_k - \bar{x})(y_k - \bar{y}) \tag{8.3}$$

and σ_x and σ_y are the estimated standard deviation:

$$\sigma_x = \sqrt{\frac{1}{N-1} \sum_{k=1}^{N} (x_k - \bar{x})^2} \quad \text{and} \quad \sigma_y = \sqrt{\frac{1}{N-1} \sum_{k=1}^{N} (y_k - \bar{y})^2} \tag{8.4}$$

The absolute value of the correlation coefficient varies from zero to one, indicating respectively a weak or strong correlation between the two variables.

Tab. 8.2 - Correlation coefficients between all the frequency-temperature pairs of the Spanish Fortress. (in bold the best correlation corresponding to the selected predictors).

CORRELATION COEFFICIENTS					
	$f1$	$f2$	$f3$	$f4$	$f5$
$T1$	0.88	0.79	0.61	0.63	0.40
$T2$	0.89	0.79	0.61	0.63	0.40
$T3$	0.88	0.78	0.60	0.63	0.40
$T4$	0.88	0.78	0.60	0.62	0.40
$T5$	0.88	0.78	0.61	0.63	0.40
$T6$	0.90	**0.81**	0.64	0.66	0.41
$T7$	**0.90**	0.80	**0.64**	**0.66**	0.41

Tab. 8.2 presents the calculated correlation coefficients between all the frequency-temperature pairs, considering 1 year of observations (from 20/12/2009 to 20/12/2010). This procedure allows selecting for each time series of the frequencies, temperature records characterized by higher correlation coefficients and thus data that better describe the dependency of the two variables. The results of the analysis demonstrate that the variation of the first frequency is highly

correlated with temperature outputs, meaning that temperature fully describes the evolution of the modal parameter during the monitoring period.

This correlation is very well fulfilled until the fourth frequency (with maximum correlation coefficients equal to 0,90 for mode #1, 0,81 for mode #2 and 0,64 for mode #3 and 0,66 for mode #4). The higher frequencies present correlation coefficients significantly lower with the measured temperatures (0,41 for mode #5). Those results can be explained by the fact that other environmental parameters or loading conditions have a stronger influence on the frequency change or by the poor quality of the estimates. The latter reason is certainly true for the 5th frequency as can be note by the scattered values of this modal parameters during the monitoring period, clearly showed by Fig. 7.27.

Following this analysis it was decided to develop statistical ARX models only for the first four natural frequencies of the fortress, that are in any case the modes that characterized more the dynamic response of the structure. The statistical procedures applied here give also the possibility to select $T6$ and $T7$ as representative temperatures due to their higher correlation with all the selected frequencies.

After this preliminary selection of the predictor variables base on correlation coefficients, we are ready to create an input-output model. As clarified before the choice fell on the implementation of a dynamic regression model, the ARX model (see §2.3.3.2). This is one of the simplest model of its kind described in the system identification literature.

In this case a SISO model was selected considering as input data the measured temperature $T6$ or $T7$ and as output data the time series of the frequencies ($f1$, $f2$, $f3$, $f4$) automatically identified.

In the first step the means are removed from the input ($\boldsymbol{x}_{\mathrm{k}}$) and output ($\boldsymbol{y}_{\mathrm{k}}$) data (data normalization):

$$x_{\mathrm{k,norm}} = \frac{x_k^m - \bar{x}_k}{\sigma_{x_k}} \qquad\qquad y_{\mathrm{k,norm}} = \frac{y_k^m - \bar{y}_k}{\sigma_{y_k}} \tag{8.5}$$

where x_k^m and y_k^m are the measured inputs and outputs, $\bar{x}_k$ and $\bar{y}_k$ are the mean values and σ_{x_k} and σ_{y_k} are the standard deviations.

There are many different ARX models, defined in Eq.(8.1), that can be fitted to data, according to the choice of the orders n_a, n_b and n_k. Some quality criteria, as proposed by Peeters & De Roeck (2000) can be used to assess and compare the quality of models: the value of the loss function λ_0 and the Akaike's Final Prediction Error (*FPE*) (Ljung 1999) defined as:

$$\lambda_0 = \frac{1}{N}\sum_{k=1}^{N} e_k^2 \qquad FPE = \lambda_0 \frac{1 + d/N}{1 - d/N} \tag{8.6}$$

where e_k is the residual error calculated between measured data and the ARX model of eq. (8.1), d is the number of estimated parameters. Another quality criterion used is the coefficient of determination R^2 defined as the ratio between the variances of the fitted values of the model ($\hat{y}_k$) and the measured values of the dependent variable (y_k^m):

$$R^2 = \frac{\sum(\hat{y}_k - \bar{y})^2}{\sum(y_k^m - \bar{y})^2} \tag{8.7}$$

The strategy for the selection of the best ARX model that can be fitted to the automatically identified natural frequency is the following. A MATLAB subroutine was implemented in order to calculate ARX models for increasing order of n_a, n_b and n_k. The maximum order was fixed at 10: ARX$[n_a, n_b, n_k] = [1{:}\,10, 1{:}\,10, 1{:}\,10]$.
Among the 10^3 possible ARX models obtained, the best one was selected according to the previously defined quality criteria. In particular the ARX model with the lower λ_0, the lower *FPE* and the higher R^2 was finally selected as the 'best' model able to explain and fit better the measured data (i.e. natural frequencies).
The results are presented in Tab. 8.3 where the best fitting ARX models for the first four natural frequencies of the fortress are presented, compared with the results of the corresponding static regression models which are a particular case of ARX model with $n_a = 0,\ n_b = 1,\ n_k = 0$.

Tab. 8.3 - Comparison between ARX and static regression SISO models of the first four natural frequencies of the fortress

Mode	ARX models						Static regression models					
	n_a	n_b	n_k	λ_0	FPE	R^2	n_a	n_b	n_k	λ_0	FPE	R^2
1	9	7	0	0.0004	0.0004	0.85	0	1	0	0.0021	0.0021	0.81
2	4	10	0	0.0075	0.0081	0.68	0	1	0	0.0101	0.0102	0.63
3	10	5	0	0.0130	0.0141	0.70	0	1	0	0.0453	0.0456	0.40
4	10	8	0	0.0036	0.0039	0.73	0	1	0	0.0165	0.0166	0.43

It is possible to observe from the analysis of the selected quality criteria the improvement of an ARX model over a simple static regression model.
Another quality criterion proposed by Peeters & De Roeck (2000) to assess the quality of the identified ARX model is to investigate the auto-correlation function of its prediction error e_k due to the fact that the prediction error should be zero-mean

white noise in the case that a good ARX model is obtained. The auto-correlation function of the prediction error is estimated as:

$$\lambda_i = \frac{1}{N}\sum_{k=1}^{N} e_{k+i}e_k \tag{8.8}$$

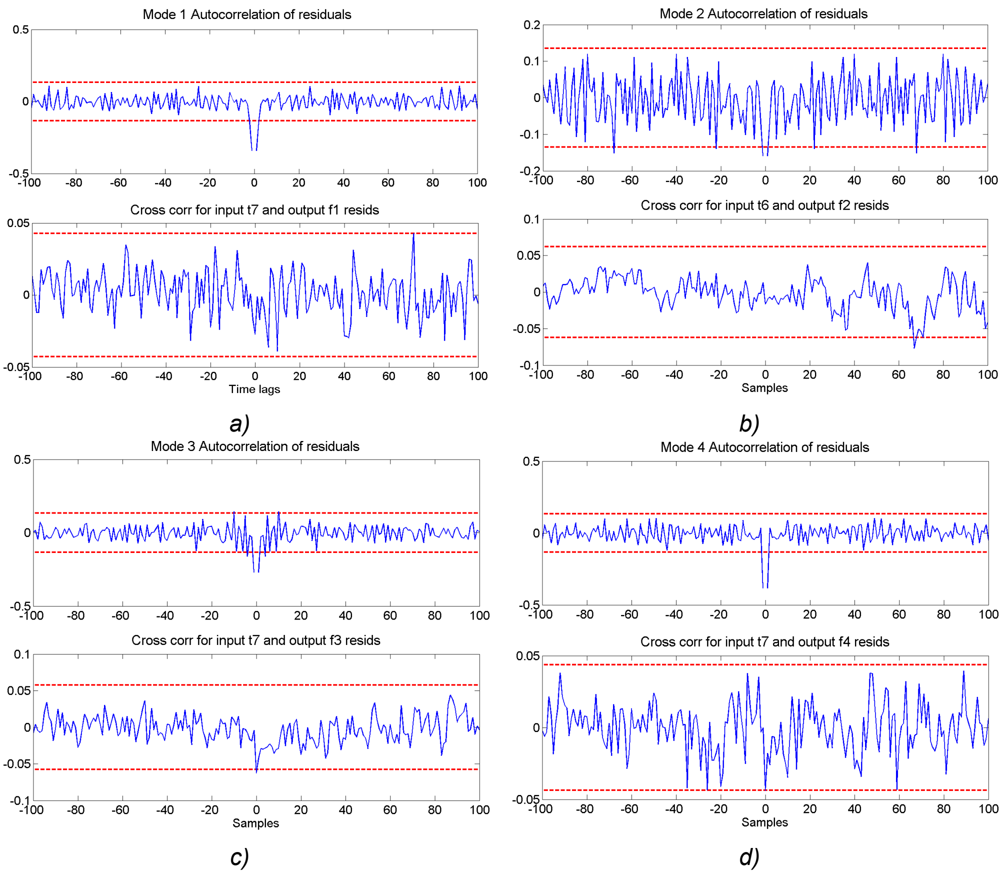


Fig. 8.2 - Normalized auto-correlation of residuals e_k for outputs and cross-correlations for input and output residuals with 95% confidence intervals of the ARX models identified for the first (a), second (b), third (c) and fourth (d) natural frequencies of the fortress.

In Fig. 8.2 the autocorrelation functions of prediction errors for the four ARX models corresponding to natural frequencies of the identified vibration modes of the fortress are plotted together with the 95% confidence intervals. Also the cross-correlation functions of residuals between input and output are reported always with 95% confidence intervals. Since both the autocorrelation and the cross-correlation functions of residuals are included within the confidence intervals, one can conclude that the residuals are white noise and no further information can be extracted from

data. It is therefore possible to state that the identified ARX SISO models for each frequency fit the measured data rather well.
Once good ARX models representing the identified natural frequencies as a function of measured environmental parameters are obtained using the data collected during the first period of monitoring (from 20/12/2009 to 20/12/2010), which represents the baseline state of the structure with the assumption that the damage pattern induced by the earthquake is stable, they can be used to simulate the natural frequencies based on the new measured environmental parameters (fresh data) during the so called validation period (from 21/12/2010 to 22/01/2013). From the comparison of these simulated natural frequencies and their identified counterparts, the changes in identified natural frequencies caused by structural damage can be distinguished from those caused by varying environmental conditions.
Following this procedure new environmental data are fed to the built ARX models, which are able to predict the corresponding eigenfrequencies (outputs) and also the standard deviation of these frequencies. The standard deviations can be used to establish confidence intervals around the predicted values. For instance, if $\hat{y}$ is the predicted output and $\hat{\sigma}_y$ the estimated standard deviation on a new observation, the $(100-\alpha)\%$ confidence interval on $\hat{y}$ is given by:

$$\left[\hat{y} - t_{\alpha/2,\nu}\hat{\sigma}_y \quad , \quad \hat{y} - t_{\alpha/2,\nu}\hat{\sigma}_y\right] \tag{8.9}$$

where the value $t_{\alpha/2,\nu}$ is found from a statistical table of the t-Student distribution and for a large number of data (as in this case) and $\alpha = 0{,}05\%$ (leading to 95% confidence intervals), we have $t_{\alpha/2,\nu} = 1{,}96$.
The confidence intervals defined in equation (8.9) can be used as an objective criterion to detect damage under the varying environmental conditions.
In the case that the natural frequency (automatically identified from time series recorded by the dynamic system) is significantly different from its ARX simulated counterpart and lies outside the confidence interval, then the variation of the identified natural frequency is caused not only by variations in environmental conditions, but also by a possible worsening of the damage pattern.

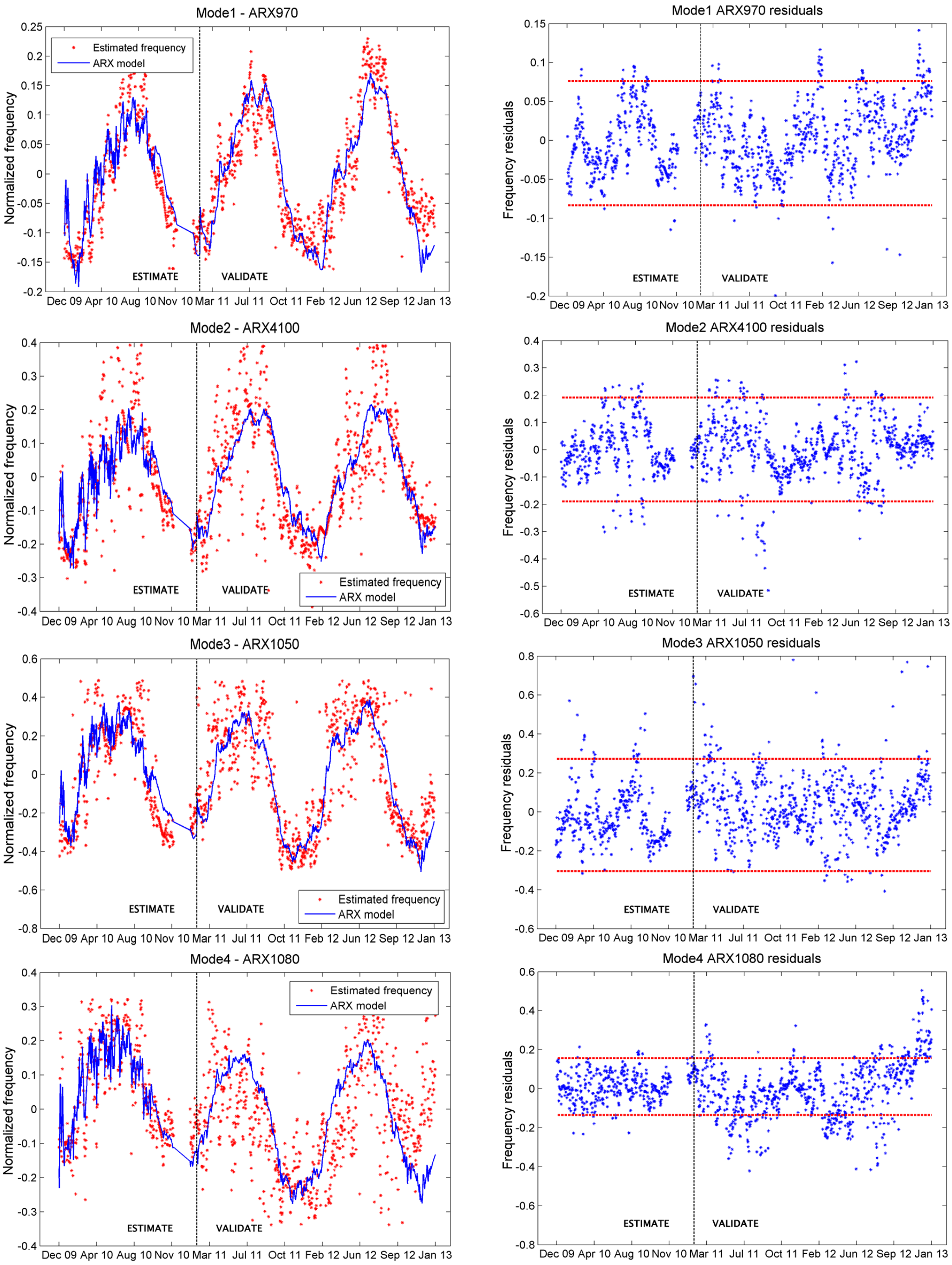


Fig. 8.3 - Left side: comparison between identified (red dots) and simulated (continuous blue line) frequencies for the first four vibration modes of the fortress. Right side: analysis of residuals (blue dots) between measured and predicted values of the same modal parameters with 95% confidence intervals

Fig. 8.3 (left side) shows the comparison between the estimated frequencies, automatically identified by the subroutine, and the same modal parameters simulated by the ARX model. The vertical line split data into two parts: an estimation period, where both input and output data (measurements) are used to estimate the statistical model and the validation period, where fresh input data related to environmental parameters are used to predict the response in terms of natural frequencies.

In Fig. 8.3 (right side) also the residuals, defined as the measured values minus the predicted values, with the 95% confidence intervals, are given. If the measured natural frequencies stand constantly outside the confidence intervals, it means that other parameters (in addition to temperatures) are influencing their behavior or it is likely that the structure has been damaged.

Based on these figures, it is observed that the identified natural frequencies are included, in general, within the confidence intervals, meaning that no further damages occurred on the structure during the validation period. This conclusion is consistent with visual inspections performed on the Spanish Fortress that confirm that damage and crack patterns induced by the earthquake are rather stable also thanks to the provisional strengthening measures implemented immediately after the seismic event.

However, it is also observed that the identified natural frequencies during some short periods move out of the bounds of the confidence intervals. This could be due to the fact that during these short periods temperatures of the validation phase are different from the ones used to estimate the ARX models. A further explanation is that other parameters influence natural frequencies in those periods, e.g. loading conditions. Therefore these ARX models are not capable of correlating well the natural frequencies with the measured environmental parameters during some days of the estimation period. The ARX models identified in this study need to be improved and validated by means the introduction of other input parameters that might influence the structural response, implementing for example Multiple Input-Single Output (MISO) models.

8.2.2.2 Civic tower

In this paragraph the same procedure for the analysis of the influence of environmental parameters on the dynamic characteristics of the Civic tower is presented. Before implementing statistical models to simulate and predict the response of the monitored structure, main results of the static monitoring are discussed, as they help understanding the outcomes of the identified modal parameters during more than 2,5 years of structural controls.

The static system of the tower, is composed by 3 displacements transducers (PZ1 to PZ3) installed on representative cracks in the lower part of the tower; 2 displacements transducers (PZ4 and PZ5) installed on the on the big cracks at the tower-palace interface; 1 inclinometer to control the displacement of the tower's top in two in-plane orthogonal directions (IN1 and IN2); strain gauges (ST1 TO ST6) on the existing metal ties and 6 thermocouples (T1 to T6). A detailed overview of the SHM system layout is reported in §5.5.4.

During the first 1,5 year of monitoring the crack and damage pattern of the tower was kept rather stable, also thanks to the provisional interventions implemented immediately after the earthquake. The strengthening measures concentrated in particular on the shaft of the tower, where a system of metal and wooden frames was installed to create a strong confinement along the entire height of the structural element. Additional ties were inserted to improve the tower-palace connection.

Fig. 8.4a presents the variation during the monitoring period (from 22/07/2010 to 09/01/2013) of the opening of three monitored cracks placed at the base of the tower, where earthquake-induced stresses caused the formation of a diffused crack pattern and the re-opening of some existing lesions. It can be noted that the damage pattern controlled by those sensors is stable, as demonstrated by the regular cyclic variation of cracks, that follow temperature changes in phase opposition. This correlation represents generally an index of reversible phenomena, i.e. the edges of the monitored lesions "open" and "reclose" cyclically, without accumulative phenomena typical of the progressive structural damages. The quasi-linear dependency of crack opening from environmental factors can be better appreciate through a crack opening vs. temperature plot (Fig. 8.4b/c). It can be observed that the lesions' edges tend to open with low temperatures and close with high temperatures: this phenomenon is quite common especially for structures characterized by high levels of hyperstaticity.

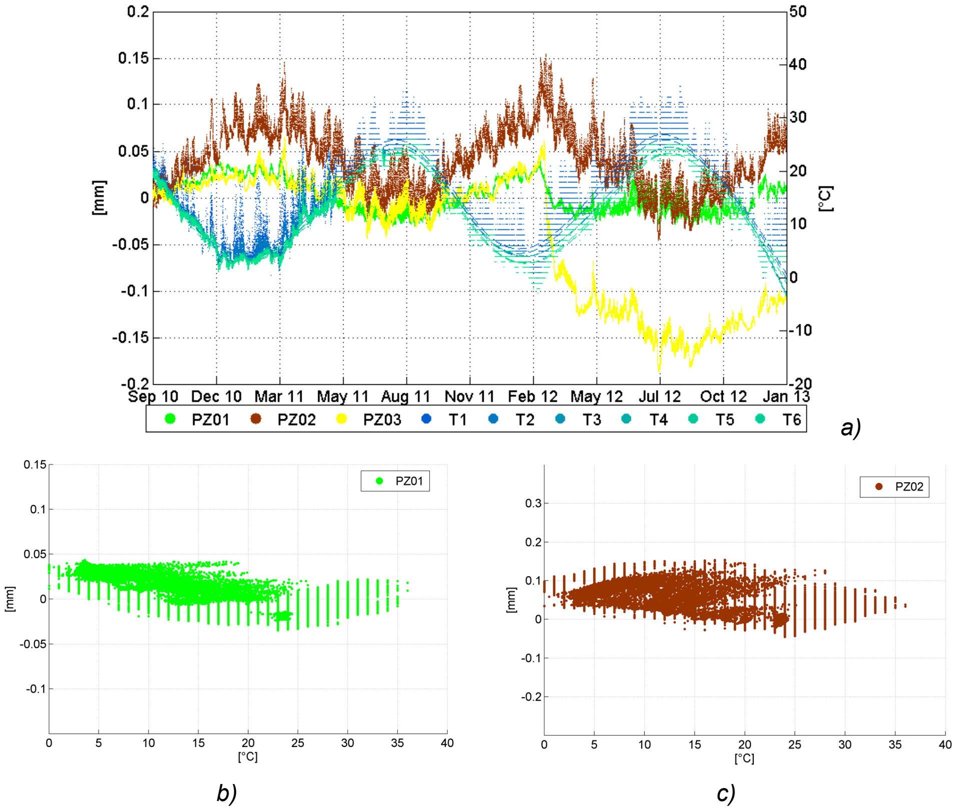


Fig. 8.4 - Variation of the crack opening of three monitored lesion over the monitoring period (a). Crack opening vs. temperature plots (b and c)

Analyzing also static data recorded by other sensors it is possible to state that until February 2012 the damage pattern induced by the earthquake on the civic tower was stable.

During February-March 2012 the equilibrium conditions of the tower underwent a significant change due to a slight rotation/displacement of the tower toward the palace, as recorded both by the inclinometer and by the displacement transducers placed at the tower-palace interface, in correspondence to the big lesion that separate the two structures (Fig. 8.5b).

Fig. 8.5a clearly shows the displacement of the tower toward the palace starting from Feb 2012. In particular the North-South translation of the top of the tower (IN1 -blue) is quantified in about 12mm, which corresponds to a re-closing of the crack between the tower and the palace of about 1mm (PZ4-orange) (see also Fig. 8.5c). Those values are consistent considering the different heights at which the displacements are recorded. The East-West displacement recorded by the inclinometer (IN2-red) is about 8mm and to this movement did not correspond a consequent re-closing of the crack monitored by PZ5 (see also Fig. 8.5d).

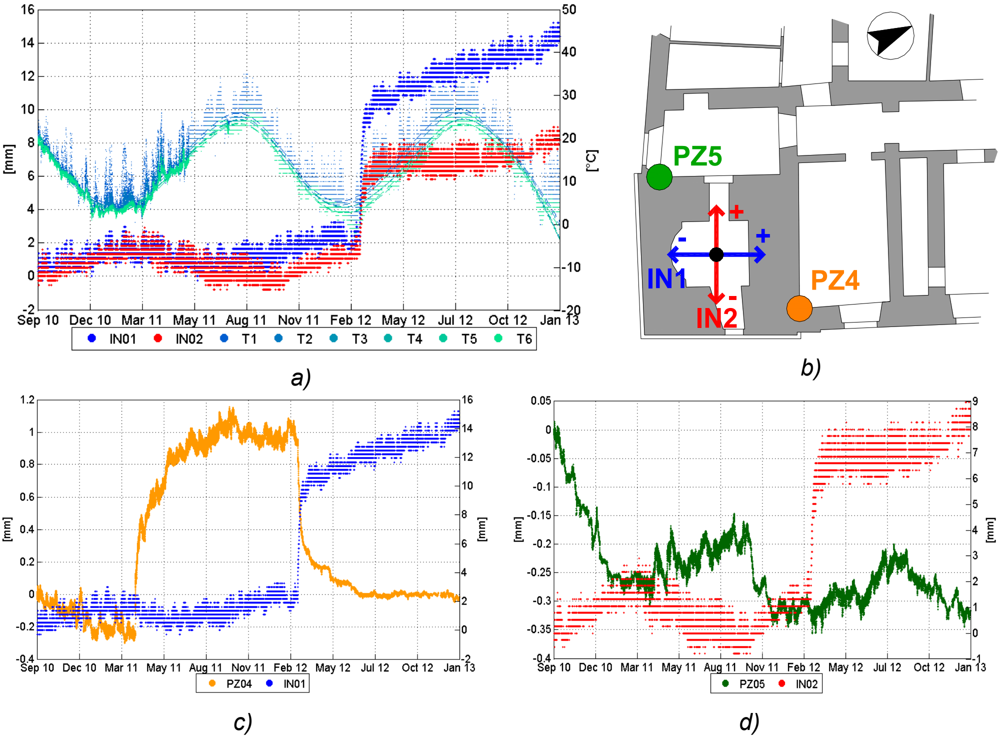
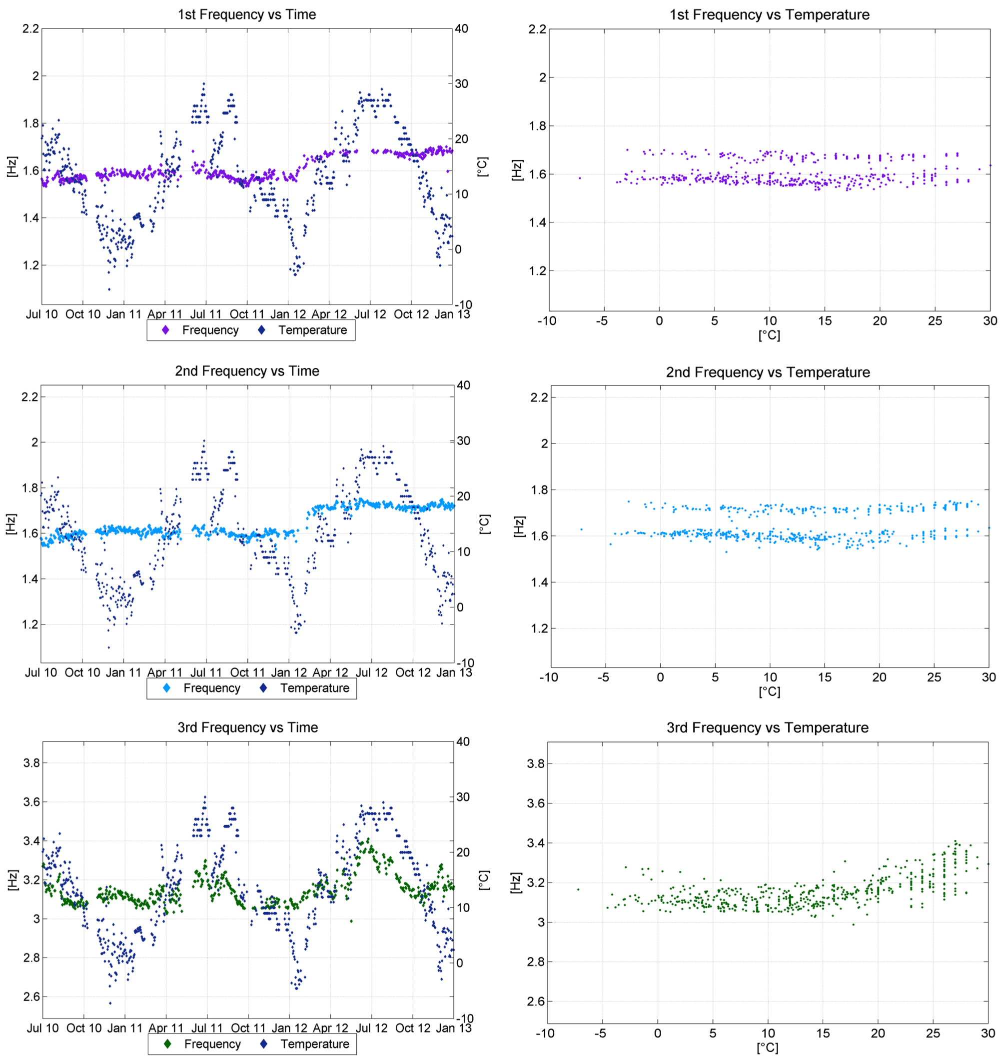


Fig. 8.5 - Static monitoring of the Civic tower: a) displacements of the tower's top recorded by the inclinometer along North-South (IN1-blue) and East-West (IN2-red) directions; b) Sensors layout: inclinometer (In1 and In2) and displacement transducers (PZ4 and PZ5) at the tower-palace interface. c) and d) comparison of displacements recorded by transducers and inclinometer along the same directions (displacement of PZ4 and PZ5 on the left y-axis, displacements of the inclinometer on the right y-axis).

In the next section information extracted from the recorded time series of the dynamic monitoring of the tower is analyzed in detail, applying the same procedure developed for the previous case study to show how the change in the equilibrium conditions of the structure influence also its dynamic response.

The dynamic monitoring system of the Civic Tower in l'Aquila is composed by 8 acceleration transducers that continuously collect dynamic information, as described in §5.6.4. Data recorded by the system are transmitted to the central server of the University of Padova and automatically processed at arrival by the algorithm for automated OMA. Feature extracted by the subroutine from time series are, as usual, natural frequencies, modal damping ratios and mode shapes. The dynamic system has been working since the 22nd of July 2010. Therefore, a database with the variation of the tower modal parameters identified by the automated algorithm from 22/07/2010 to 09/01/2013 is analyzed. In §7.5.3 the evolution of the tower first 7 natural frequencies, as a result of the automated algorithm, was presented and briefly discussed. Here a more detailed analysis of the variation of natural frequencies and their strict correlation with environmental parameters is reported.

Fig. 8.6 shows a zoom on the evolution over 2,5 years of monitoring of the first 7 natural frequencies of the tower and the dependency on environmental factors through frequency vs. temperature plots.



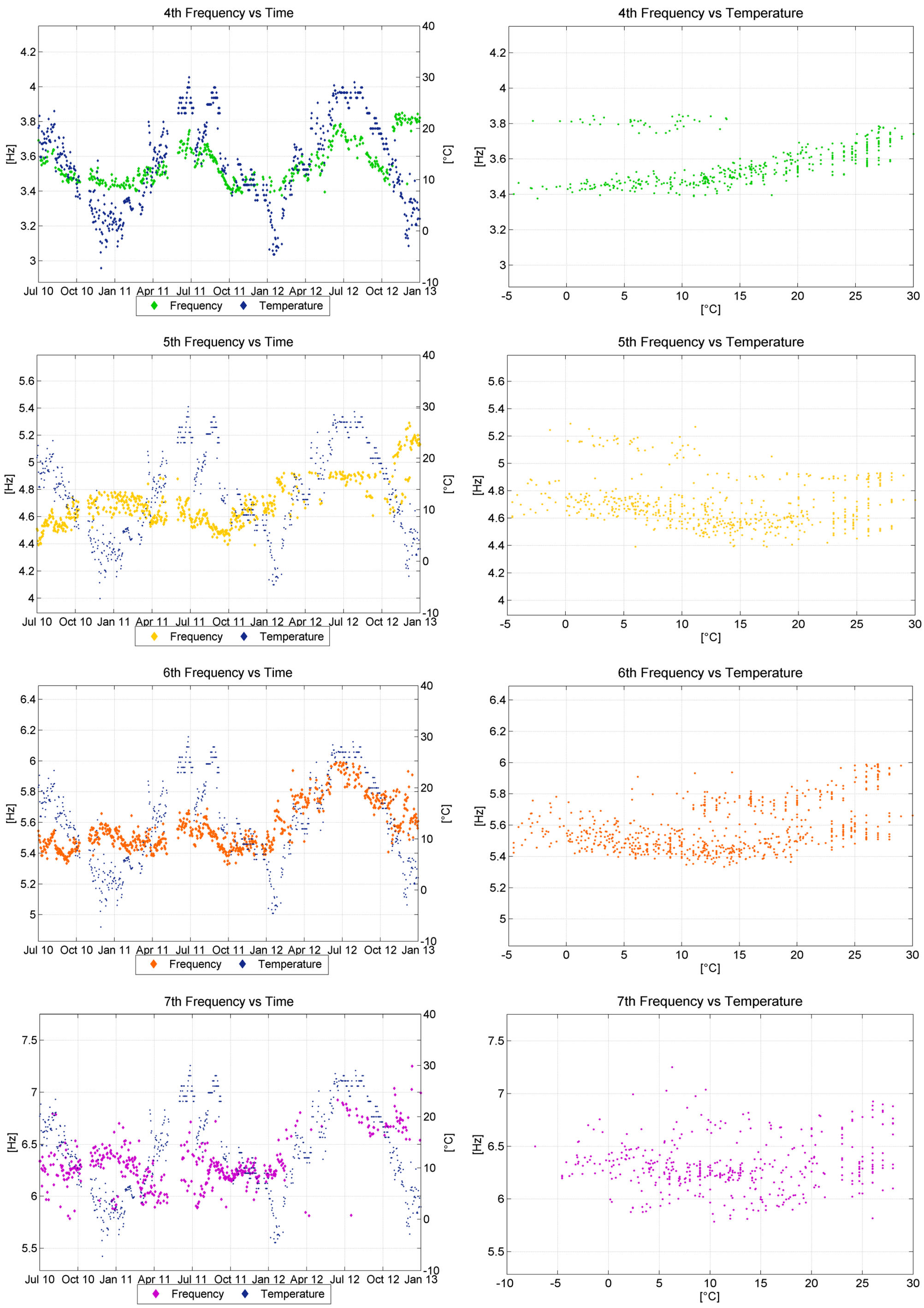


Fig. 8.6 - Zoom on the first seven vibration modes of the Civic tower. Left column: frequency vs. time plots. Right column: frequency vs. temperature plots

Analyzing the graphs in Fig. 8.6 (left column) one can note that starting from February 2012 all the natural frequencies of the tower tend to slightly increase, exactly in the same period when the anomalous static displacements of the structure were recorded. The frequency change can be appreciated also looking at frequency vs. temperature plots Fig. 8.6 (right column), where for almost all modes it is possible to distinguish two parallel clouds of points, demonstrating the frequency shift not correlated with ambient factors. This phenomenon will be better studied and explained through the construction of statistical models and the application of residual analysis, that allow filtering out the environmental effects.
The statistical results of the evolution of natural frequencies over the entire period of monitoring are presented in Tab. 8.4.

Tab. 8.4 - Statistical results of the first 5 natural frequencies of the Civic tower

Mode	f_{max} [Hz]	f_{min} [Hz]	f_{mean} [Hz]	f_{change} [%]	f_{std} [Hz]	f_{CV} [%]
1	1,701	1,533	1,604	10,92	0,047	2,93
2	1,752	1,531	1,642	14,44	0,060	3,64
3	3,410	2,988	3,150	14,09	0,076	2,42
4	3,849	3,377	3,558	14,00	0,118	3,32
5	5,291	4,391	4,692	20,48	0,173	3,69
6	5,989	5,328	5,566	12,41	0,152	2,73
7	7,251	5,786	6,305	25,32	0,232	3,68

Also in the present application, as continuous temperature measurements are available, ARX models are tested. Data collected during the first year of monitoring (from 22/07/2010 to 22/07/2011) were used to build the regression models and then data collected during the remaining 1,5 year of monitoring (from 23/07/2010 to 09/01/2013) were used to validate the quality of the forecasts provided by the models, trying to detect through residual analysis any changes in the response, possibly linked to damage.
In a first step a correlation analysis between all recorded temperature measurements and estimated natural frequencies is performed to select the predictors of the ARX models. In this case temperature records collected by 6 thermal sensors (T1 to T6) able to measure both air and wall temperature are analyzed.
Tab. 8.5 presents the calculated correlation coefficients between all the frequency-temperature pairs, considering 1 year of observations.

Tab. 8.5 - Correlation coefficients between all the frequency-temperature pairs of the Civic tower (in bold the best correlation corresponding to the selected predictor)

CORRELATION COEFFICIENTS							
	$f1$	$f2$	$f3$	$f4$	$f5$	$f6$	$f7$
$T1$	**0,23**	0,20	**0,50**	**0,84**	0,41	**0,13**	0,14
$T2$	0,15	0,27	0,49	0,81	0,48	0,10	0,14
$T3$	0,05	0,35	0,44	0,76	0,54	0,04	0,14
$T4$	0,04	0,35	0,44	0,75	0,54	0,04	0,13
$T5$	0,03	**0,36**	0,43	0,75	**0,55**	0,04	0,13
$T6$	0,09	0,33	0,46	0,78	0,53	0,05	**0,15**

The results of the correlation analysis demonstrate that the variation of the first two frequency (first order bending modes) is poorly correlated with temperature outputs, meaning that temperature do not fully describe the evolution of the modal parameters. Natural frequencies are much better correlated to temperatures for modes #3 and #4 (second order bending modes) and for the torsional mode (#5). The last two eigenfrequencies (third order bending) present the lowest correlation coefficients and thus are excluded from the analysis. The main reason can be attributed again by the poor quality of the estimates for the higher frequencies.

Following this analysis it was decided to develop statistical ARX models only for the first five natural frequencies of the tower. The statistical procedures applied here give also the possibility to select $T1$ and $T5$ as the best predictors.

Applying the same procedure described for the previous case study SISO ARX models have been then constructed, selecting the best model according to the following quality criteria:

(i) Loss function λ_0 defined by Eq. (8.6)
(ii) Akaike's Final Prediction Error (*FPE*) defined by Eq. (8.6)
(iii) Coefficient of determination R^2 defined by Eq. (8.7)
(iv) Auto-correlation and cross correlation functions of the prediction error defined by Eq.(8.8)

The results are presented in Tab. 8.6 where the best fitting ARX models for the selected five natural frequencies of the tower are presented, compared with the results of the corresponding static regression models.

The analysis of results indicates acceptable quality of the developed statistical models, especially for modes #3, #4 and #5. As can be expected the quality of the ARX models of the first two frequencies is lower due to the poor correlation between inputs and outputs (as demonstrated in Tab. 8.5).

Tab. 8.6 - Comparison between ARX and static regression SISO models of the first five natural frequencies of the tower

Mode	ARX models						Static regression models					
	n_a	n_b	n_k	λ_0	FPE	R^2	n_a	n_b	n_k	λ_0	FPE	R^2
1	7	10	0	0,0001	0,0001	0,52	0	1	0	0,0003	0,0003	0,23
2	6	10	0	0,0001	0,0001	0,38	0	1	0	0,0002	0,0002	0,36
3	5	9	0	0,0004	0,0004	0,54	0	1	0	0,0018	0,0018	0,50
4	9	10	0	0,0004	0,0005	0,85	0	1	0	0,0016	0,0016	0,84
5	0	10	0	0,0051	0,0054	0,54	0	1	0	0,0055	0,0056	0,55

In Fig. 8.7 the autocorrelation and cross correlation functions of prediction errors for the first four ARX models are plotted together with the 95% confidence intervals. Based on these figures, it is possible to state that the identified ARX SISO models fit the measured data quite well.

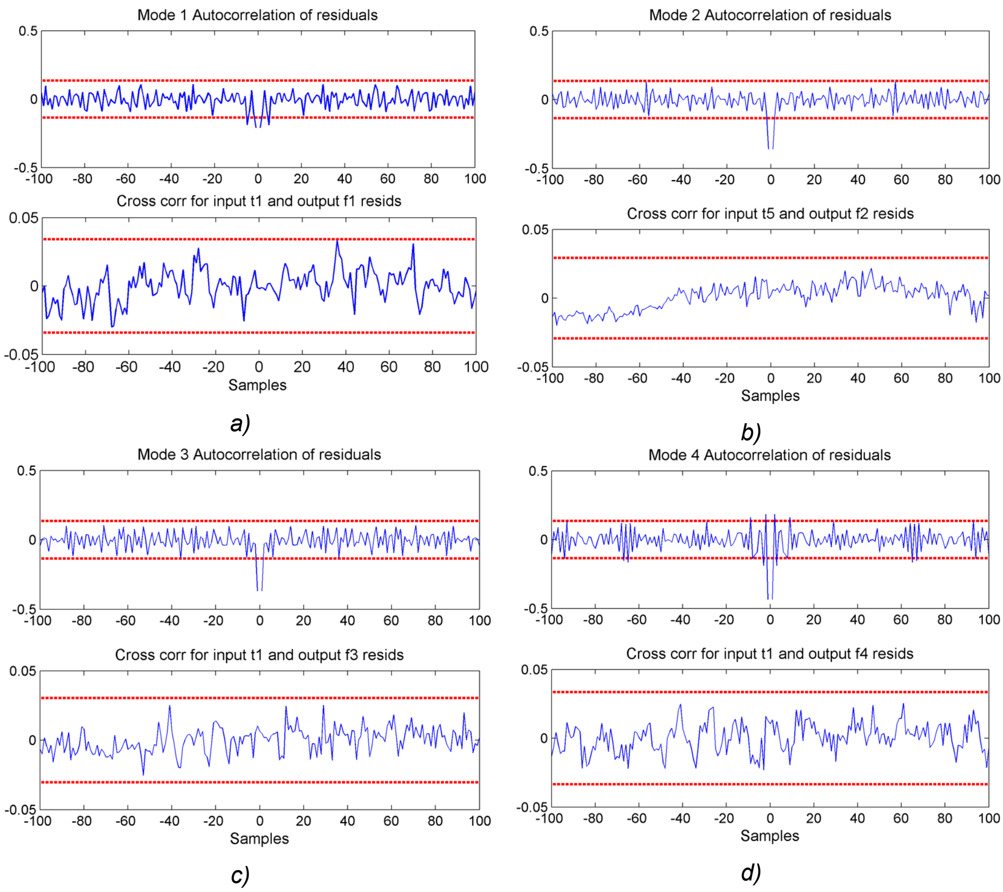
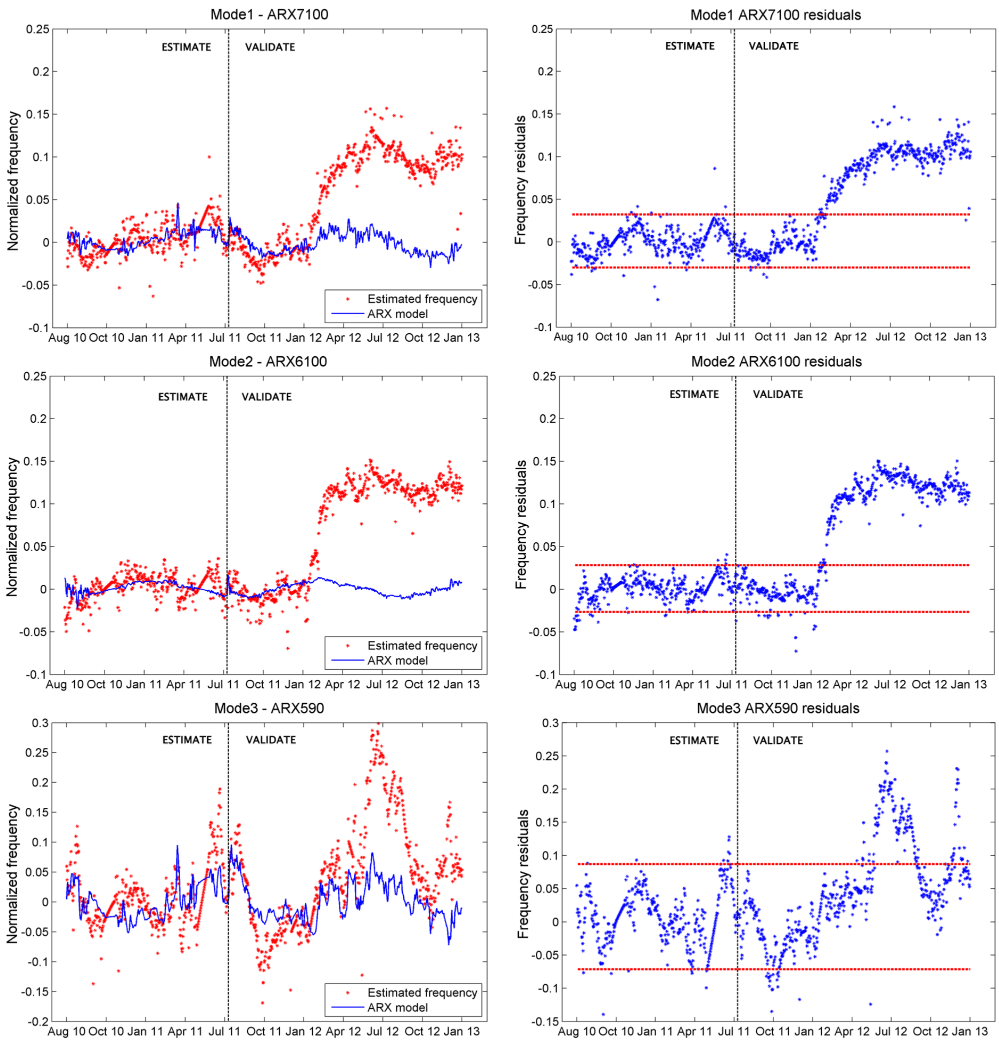


Fig. 8.7 - Normalized auto-correlation of residuals e_k for outputs and cross-correlations for input and output residuals with 95% confidence intervals of the ARX models identified for the first (a), second (b), third (c) and fourth (d) natural frequency of the civic tower

Once good ARX models representing the identified natural frequencies as a function of measured environmental parameters are obtained using data collected during the first year of monitoring (baseline state of the structure with the assumption that the damage pattern induced by the earthquake is stable), they can be used for prediction. To this aim natural frequencies are predicted on the basis of new measured environmental parameters (fresh data) during the so called validation period (23/07/2011 to 09/01/2013). From the comparison of the simulated natural frequencies and their identified counterparts, changes in identified natural frequencies caused by structural damages can be distinguished from those caused by varying environmental conditions.



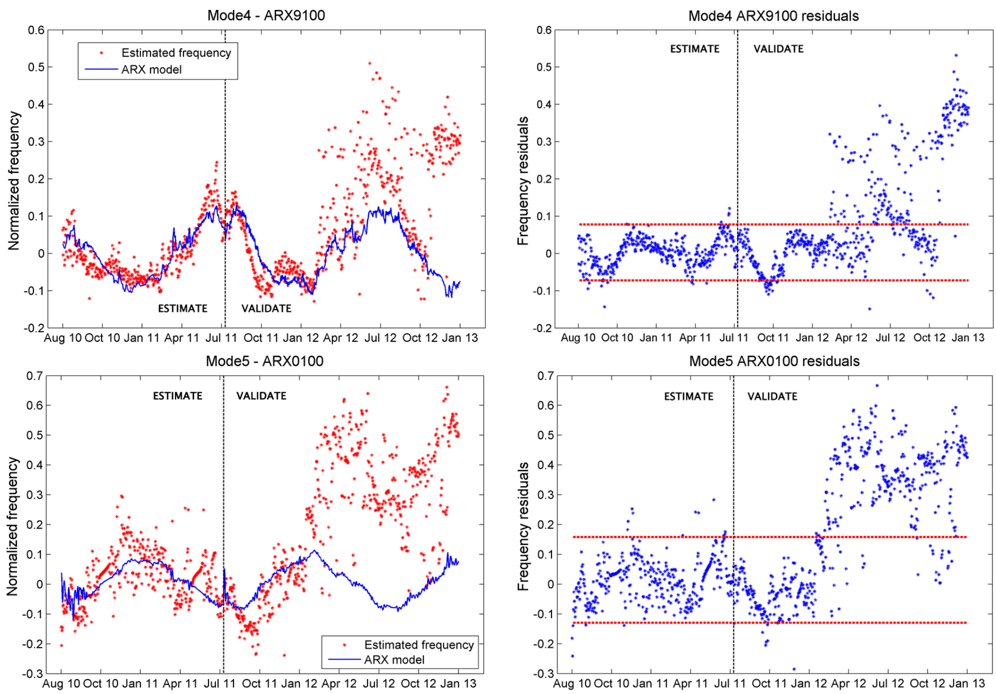


Fig. 8.8 - Left side: comparison between identified (red dots) and simulated (continuous blue line) frequencies for the first five vibration modes of the tower. Right side: analysis of residuals (blue dots) between measured and predicted values of the same modal parameters with 95% confidence intervals

Fig. 8.8 (left side) shows the comparison between the estimated frequencies, automatically identified by the subroutine, and the same modal parameters simulated by ARX models. The vertical line split data into two parts: an estimation period, where both input and output data (measurements) are used to estimate the statistical model and the validation period, where fresh input data related to environmental parameters are used to predict the response in terms of natural frequencies.

In Fig. 8.8 (right side) also the residuals, defined as the measured minus the predicted value, with the 95% confidence intervals, are given. If the measured natural frequencies stand constantly outside the confidence intervals, it means that other parameters (in addition to temperatures) are influencing their behavior or it is likely that the structure has been damaged.

The analysis of these plots demonstrates that until February 2012 the tower did not suffer significant damages, or better, that earthquake-induced damages were rather stable, as the frequency residuals are always included within the confidence intervals.

Starting from February 2012, as also demonstrated by the analysis of static parameters, the equilibrium conditions of the tower changed significantly due to a substantial displacement of the structure toward the palace in both horizontal directions (Fig. 8.5).

As described in 5.5.2, the 2009 seismic event caused major damages at the tower-palace connection, with the formation of big vertical lesions that partially disconnected the two structural bodies. Even if provisional strengthening measures were immediately applied, it seems that some relative movements between palace and tower are still permitted.

Changes in the structural and boundary conditions of the tower led to an evident variation also of the dynamic response. The slight rotation/displacement of the tower toward the palace caused in fact an increase of the degree of interconnection between the two structures and modifications of the restraint system offered by the palace. The new mechanical equilibrium conditions, characterized by stiffer constraints, are responsible of the increment of natural frequencies, clearly visible from the residuals of all analyzed vibration modes. This phenomenon is particularly evident for the first two closely spaced bending modes of the tower, characterized by the highest quality of the modal estimates.

Thanks to the adopted statistical-based procedures for damage identification it was possible to detect successfully significant changes in the structural conditions of the building, cross correlating static monitoring outputs with the outcomes of modal parameters estimation.

The two presented case studies demonstrate that, combining monitoring results and applying robust data-driven approaches, one can control the long-term effectiveness of structural interventions, even if provisional, in a post seismic scenario.

The procedure developed and validated within this paragraph proved to be very effective to filter out environmental effects and detect ongoing damaging processes. Based on this conclusion it is certainly possible to extend the methodology to other monitored structures, even if not characterized by severe damage patterns. In these cases it is not likely that a significant and sudden change of the monitored parameters occur in such short time. However, this procedure can be successfully applied when possible damage states lead to the modification of the dynamic structural response. Moreover similar statistical models can be constructed also to analyze the static response (e.g. opening/reclosing of cracks, inclinations, etc.), whose variations are strictly correlated with environmental factors.

8.3 SHM for model updating

Another main purpose of SHM and dynamic identification is found in the validation and, when necessary, improvement of Finite Element (FE) model through modal matching procedures.

Outcomes of monitoring (both in terms of static measurements and dynamic response) can be compared with numerical predictions resulting from a numerical model. In particular, the modal matching procedure is considered for this purpose. It involves upgrading the model until a satisfactory agreement in terms of both frequencies and modal shapes is obtained. The model improvements or updating may target to material properties, geometry and morphology of structural members, connections, influence of the soil and possible soil-structure interaction effects, influence of neighboring buildings or adjacent structures and damage distribution.

The calibration procedure starts from the construction of the FE model based on detailed geometrical and material surveys. In the second step elastic material properties are assigned (i.e. mass density, modulus of elasticity, Poisson's modulus), according to the results of on-site characterization of materials or to initial assumptions, based on material databases provided by codes and standards.

Then the numerical prediction in terms of natural frequencies and mode shapes are compared to the experimental outcomes. Previously defined material properties are iteratively changed until a satisfactory modal matching is reached. Model updating provides a progressive "improvement" of the FE model, through successive changes and corrections on the mass and stiffness matrices.

More in detail the calibration process involves:

- Comparison of frequencies (FE) of the modal analysis with the frequencies obtained from the experimental data; average errors for each numerical-experimental frequency pair is calculated.
- Comparison of mode shapes using the MAC index, which provides an indication that the modal vectors experimentally measured and numerically calculated are consistent or not. Values of MAC near unity indicate a quasi-perfect correlation between reciprocal mode shapes (numerical vs. experimental).

Model updating process can be implemented using an objective function to be minimized, composed by the residuals between numerical and experimental frequencies and mode shapes. In the context of the present work, since modal updating is not the main objective of the research, it was decided to apply a "manual" calibration process.

8.3.1 Applications to Verona case studies

In the framework of the present research two cases of model updating will be briefly analyzed and discussed. Modal parameters extracted from preliminary Ambient Vibration Tests and later continuously identified by the automated algorithm for OMA represent the output data of the calibration and validation process. In the next paragraphs the Finite element (FE) models of the Cansignorio stone tomb and of the wing of the Roman Arena in Verona are presented.

8.3.1.1 Cansignorio stone tomb

A detailed FE numerical model of the Canisgnorio stone tomb, based on a laser scanner geometrical survey of the monument previously carried out, was implemented in order to evaluate the static and dynamic behaviour of the monument. The cloud of points acquired by the laser scanner was imported in a CAD environment and prepared for the successive export in the FE software, Straus7, G+D Computing (2004).

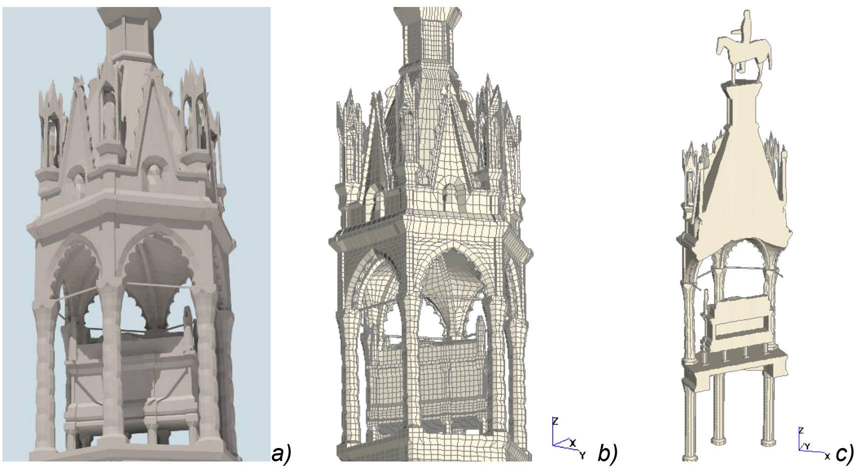


Fig. 8.9 - Construction of the FE model: from geometric survey (a) to the final mesh (b and c)

The evaluation of the initial results of the numerical model (linear static and natural frequency analyses) assisted the design phase of the strengthening intervention and indicated the most suitable places for the sensors' positioning (dynamic identification and monitoring). The first model was calibrated on the basis of the results of the experimental activities, in order to be subsequently used to simulate the response of the monument to different external actions.
As a first step, linear elastic constitutive laws were assigned to all materials. This is clearly a simplification of the real mechanical behavior of the structure. However, this schematization is very useful in the initial phase of study since it provides useful

information on the preliminary investigated phenomena (static analysis, dynamic identification of the model on the basis of the environmental vibrations and model updating process).

The model is composed by approximately 49'000 3D 8-noded solid elements and 53'600 nodes. Finite elements' sides are comprised between 0.10 - 0.15 m. The mesh is more refined in the slender elements (columns) and in the junctions, rougher elsewhere. The decorative elements and statues were modeled as the structural parts: only areas too small to be considered significant were neglected.

The proposed calibration procedure follows three main steps:

(i) identification of the morphology and of materials that characterized the structure (Fig. 8.10);

(ii) definition of a range of variation (i.e. upper and lower bounds) of the mechanical properties of identified materials (Tab. 8.7);

(iii) iterative variation of the mechanical properties of the selected materials within the defined range until reaching the final calibration and the optimal values.

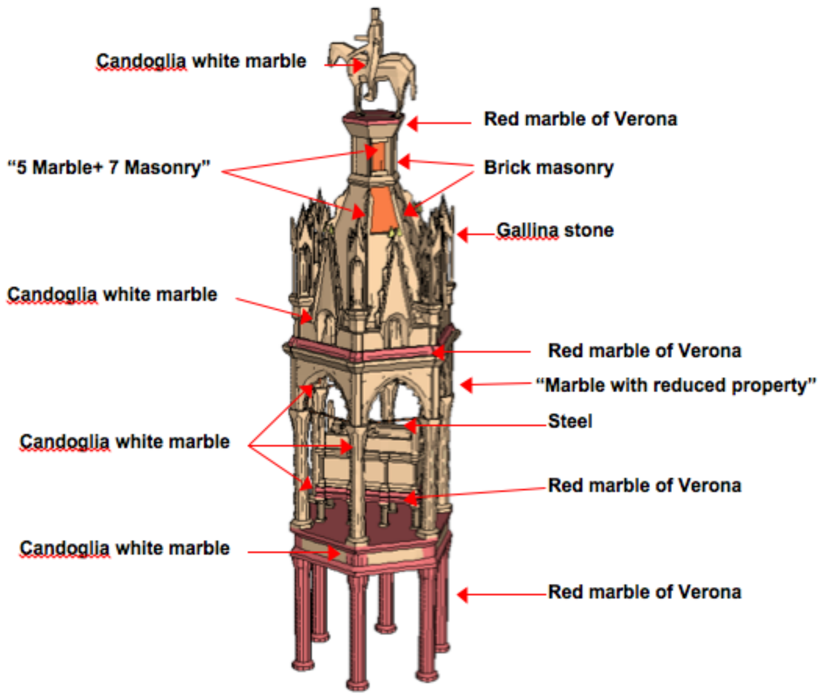


Fig. 8.10 - Identification of morphology and materials of the structure

The initial mechanical properties of materials have been iteratively changed until reaching a proper matching between modal parameters experimentally identified and numerically calculated. The values of material characteristics before and after the calibration are reported in Tab. 8.7

Tab. 8.7 - Initial (range) and final values of elastic properties assigned to the various materials that compose the monument

Material	INITIAL VALUES		FINAL VALUES	
	Mass density [kg/m^3]	Elastic modulus [GPa]	Mass density [kg/m^3]	Elastic modulus [GPa]
Red marble of Verona	2600÷2700	40÷70	2692	34
Candoglia white marble	2600÷2700	40÷70	2650	29
Brick Masonry	1800	1,8÷2,4	1800	3
Steel	7850	210	7850	210
Gallina stone	1400÷2000	10÷30	2000	15
5 marble+7 masonry	2000÷2200	10÷20	2155	13,83
Marble with reduced property	2600÷2700	15÷30	2350	23

The global updating results are presented in Tab. 8.8. For the comparison the average value of natural frequencies recorded during the entire monitoring period and automatically identified by the algorithm are considered. An average mode shape vector for each vibration mode is also calculated and used in the calculation of the MAC index.

Tab. 8.8 - Calibration results: comparison between experimental and numerical modal parameters

MODE	Type	f_{EXP}^{mean} [Hz]	f_{FEM} [Hz]	Average error ε [%]	MAC $(\{\psi^{EXP}\},\{\psi^{FEM}\})$
1	1st bending N-S	3,14	3,27	|4,13|	0,80
2	1st bending E-O	3,19	3,29	|3,16|	0,80
3	1st torsion	5,82	5,86	|0,61|	0,93
4	2nd bending N-S	12,55	12,5	|0,39|	0,50
5	2nd bending E-O	12,86	12,85	|0,09|	0,42
6	2nd torsion	19,42	19,11	|1,60|	0,30

The calibration results are satisfying and acceptable as the absolute frequency errors are always less than 5% and on average equal to 1,66%; MAC index, calculated between experimental and numerical mode shapes, is greater than 0,8 for the first 3 mode shapes and decreases to bad correlations for the higher modes. This problem is certainly correlated to the complexity of the structure and the calibration need to be improved in order to match better experimental and numerical mode shapes.

Fig. 8.11 presents the tuned numerical mode shapes of the structure after calibration. It is possible to observe the high level of symmetry of the bending mode shapes.

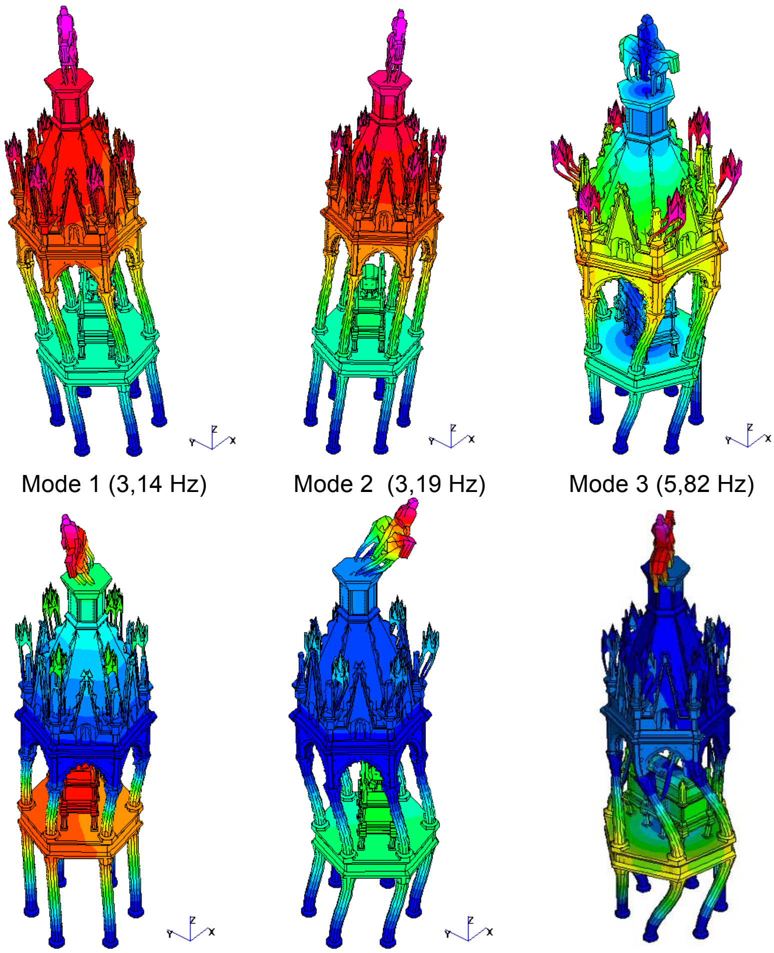


Fig. 8.11 - Numerical mode shape of Canisgnorio stone tomb after model updating

Other results regarding the correlation between experimental and numerical modal parameters can be observed in Fig. 8.12, through the so called calibration curve, where experimental and numerical frequencies pairs and MAC index are presented.

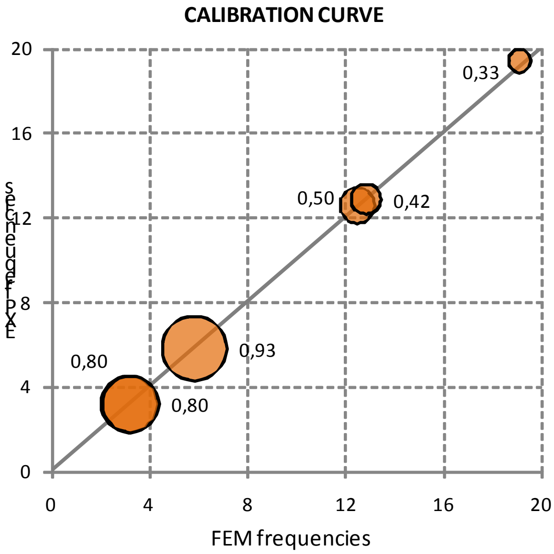


Fig. 8.12 - Calibration curve: comparison between numerical and experimental frequencies and MAC results

The tuned FE model is too complex to perform more sophisticated seismic analysis to assess the safety conditions of the monument. It was therefore necessary to implement a second model with a simplified geometry for this specific purpose. The FE model was built in DIANA FE package and tuned again on the basis of the experimental data available, starting from the updated parameters of the previous model.

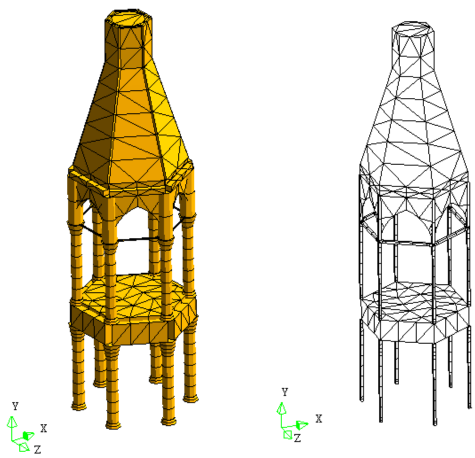


Fig. 8.13 - Simplified FE model of the Cansignorio stone tomb used for the implementation of advanced seismic analyses

The model is composed by 738 8-noded curved shell elements and 1235 nodes. It is possible to note that all the decorative parts have been removed and only the structural elements have been modeled, trying to find a good compromise between a mesh sufficiently refined and precise and computational costs. The same model updating procedure followed before was then applied, reaching also in this case a proper modal match between experimental and numerical parameters. For the sake of completeness the comparison table of results of the model calibration is reported in Tab. 8.9.

Tab. 8.9 - Calibration results of the simplified FE model: comparison between experimental and numerical modal parameters

MODE	Type	f_{EXP}^{mean} [Hz]	f_{FEM} [Hz]	Average error ε [%]	MAC $(\{\psi^{EXP}\},\{\psi^{FEM}\})$
1	1st bending N-S	3,14	3,23	|2,94|	0,72
2	1st bending E-O	3,19	3,231	|1,39|	0,71
3	1st torsion	5,82	5,67	|2,72|	0,96
4	2nd bending N-S	12,55	12,81	|2,04|	0,78
5	2nd bending E-O	12,86	12,82	|0,32|	0,69
6	2nd torsion	19,42	18,87	|2,89|	0,96

It can be noted a very good correspondence in terms of natural frequencies, with an average error always less than 3% and, on average, equal to 2,05%. The calculated MAC index demonstrate a rather good match of mode shapes for all the selected structural modes, with a significant improvement of this parameter from the previous model, especially for the higher frequencies. It is evident that a simplified model is easier to manage and the updating process is much easier to control. On the other hand it is necessary to avoid as much as possible any loss of information that can be fundamental for a proper description of the structural behavior.

The tuned simplified FE model was then used to perform advanced seismic analyses, implementing and adding non linear constitutive models of materials (WP9, NIKER Project 2012).

8.3.1.2 Arena of Verona

Given the huge dimensions and complexity of the Arena of Verona it was decided to concentrate the numerical modeling on the wing of the Arena, i.e. the freestanding wall survived from the collapse of the outer ring of the monument. This choice was made also considering that the wing is certainly the most vulnerable structural element from a seismic point of view and thus structural analyses to assess the safety conditions of the Arena will focus mainly on this part. Moreover the wing of the Arena was subjected during the 60’ to an important strengthening intervention, as described in §5.2.2, with the insertion of a system of post-tensioned steel cables along its massive stone pillars. The final aim of successive structural analyses (not reported within this research work) is the assessment of this intervention, evaluating its effectiveness after more than 50 years.

The FE model of the wing was built in *DIANA* finite element software (DIANA™, TNO, Delft, *release 9.4.4*, 2012), which provides several non linear constitutive models to describe the behavior of masonry structures. The idea is to perform advanced seismic analyses after the calibration and validation of the FE model on the basis of outcomes of dynamic investigations and monitoring.

The numerical model of the wing of the Arena is composed by 2D 8-noded curved shell elements that represent the masonry continuum, schematized as an isotropic and homogeneous material. Shell elements were selected, instead of fully 3D element to reduce computational costs, especially during the execution of complex nonlinear analyses.

Structural elements and loading conditions that are relevant in relation to the structural problems have been directly inserted into the numerical simulation. Portions of structures not included in the model but directly involved in the structural interaction (e.g. the connection with the main body of the amphitheater) are considered by the introduction of corresponding loading and boundary conditions.

The model is composed by approximately 2667 shell elements, 2451 nodes and 125 embedded beam elements to simulate the post-tensioned steel cables inside the stone pillars.

Once the numerical model was constructed the model updating process started, trying to reach a satisfying modal match between natural frequencies and mode shapes experimentally identified and numerically calculated. The target experimental modal parameters are those identified during the preliminary dynamic identification campaign, reported in §5.2.3.2.

The proposed calibration procedure, slightly different from the previous case study, follows three main steps:

(i) identification of the morphology and of materials that characterized the structure (Fig. 8.14);
(ii) definition of initial values of the elastic mechanical properties for each material that composes the structure (Tab. 8.10);
(iii) iterative variation of the mechanical properties of the selected materials within a predefined range until reaching the final calibration and the optimal values.

The first step started from the analysis of the available geometric and material surveys and lead to the definition of dimensions and thickness for each identified structural element. Fig. 8.14 presents the mesh of the numerical model with the identification of different materials, whose initial values of elastic mechanical properties are listed in Tab. 8.10.

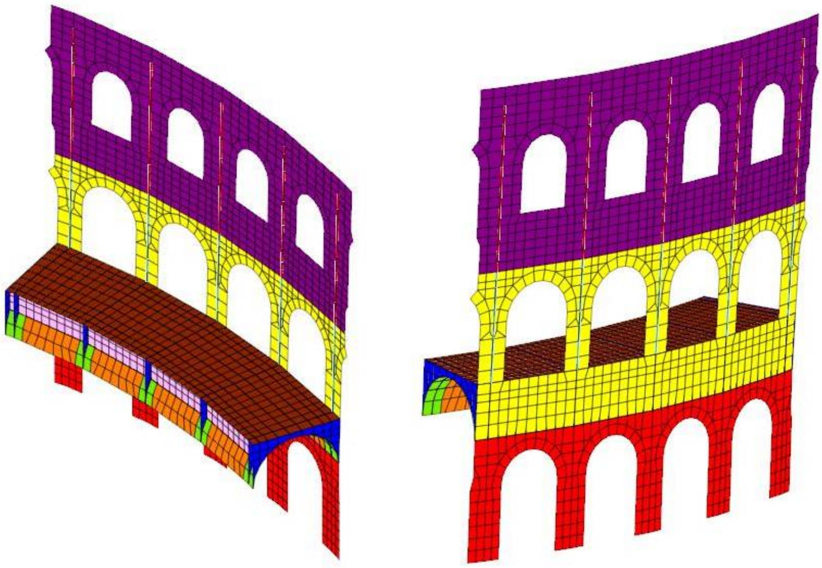


Fig. 8.14 - Mesh of the FE model with the identification of material and the updating parameters

Tab. 8.10 - Initial values of parameters to be updated

Structural element	Elastic Modulus [MPa]	Poisson ratio	Mass Density [kg/m^3]	Thickness [cm]	Element colour
Stone masonry I order	15000	0,2	2700	220	Red
Stone masonry II order	15000	0,2	2700	190	Yellow
Stone masonry III order	15000	0,2	2700	90	Magenta
Vault	2400	0,2	1800	60	Orange
Arches	15000	0,2	2700	60	Green
Frenelli	500	0,2	750	20	Blue
Infill of vaults	500	0,2	750	Variable	Pink
Stone floor	12000	0,2	2500	10	Brown

3D beam elements were used to model the post-tensioned steel cables inserted during the structural strengthening intervention performed during the ‘60s. Along the height of each pillar of the 2nd and 3rd order of arches holes were drilled, steel cables inserted and filled with reinforced concrete, before applying the prestressing load. For each element an equivalent area equal to the sum of the cross sections of the cables within each pillar is defined with a post-tension stress equal to 9500 kg/cm^2. This value has been selected according to the original calculations performed by Eng. Morandi (Morandi, 1956) during the design phase of the intervention. It is noted that the modulus of elasticity of steel cables was reduced of about 30% in order to take into account the not perfect bond between reinforcing bars and reinforced concrete.

Tab. 8.11 - Mechanical properties of the post-tensioned steel cables

Structural element interested by the strengthening intervention	Elastic Modulus [MPa]	Poisson ratio	Mass Density [kg/m^3]	Cross section [cm^2]	Post-tension load [kN]
Pillars III order	150000	0,3	7850	10,6 (3x18Φ5)	987,88
Pillars II order	150000	0,3	7850	21,2 (6x18Φ5)	1975,73

Model updating consisted in the iterative variation of the mechanical properties of the identified materials in terms of mass density and elastic modulus, directly correlated with the mass and stiffness matrices of the FE model. Experimental vs. numerical frequencies and mode shapes are compared during the calibration process, trying to minimize the average error in term of frequency and maximize the MAC index for each calculated/extracted mode shape pair.

The global updating results are presented in Tab. 8.12. For the comparison natural frequencies and mode shapes identified during the preliminary dynamic identification tests on the wing are considered.

Tab. 8.12 - Calibration results of the FE model: comparison between experimental and numerical modal parameters

MODE	Type	f_{EXP} [Hz]	f_{FEM} [Hz]	Average error ε [%]	MAC $(\{\psi^{EXP}\},\{\psi^{FEM}\})$
1	1st out-of-plane bend.	1,924	1,924	|0,01|	0,973
2	1st torsional	2,666	2,640	|1,00|	0,993
3	2nd torsional	5,103	5,122	|0,36|	0,984
4	2nd out-of-plane bend.	6,086	6,054	|0,53|	0,936
5	3rd torsional	7,308	7,323	|0,20|	0,886
6	4th torsional	9,434	9,464	|0,32|	0,821
7	5th torsional	10,970	10,944	|0,24|	0,973

The model updating leads to an excellent calibration and perfect matching between experimental and numerical modal parameters. The absolute frequency errors are always less than 1% and on average equal to 0,38%; MAC index, calculated between experimental and numerical mode shapes, is always greater than 0,8 with a quasi-perfect correlation for the first 3 mode shapes. In this case the mode shape correlation is very good also for the higher frequencies.

The high quality of updating results are also confirmed by the calibration curve, obtained from the comparison between numerical (x axis) and experimental (y axis) frequencies, as reported in Fig. 8.15.

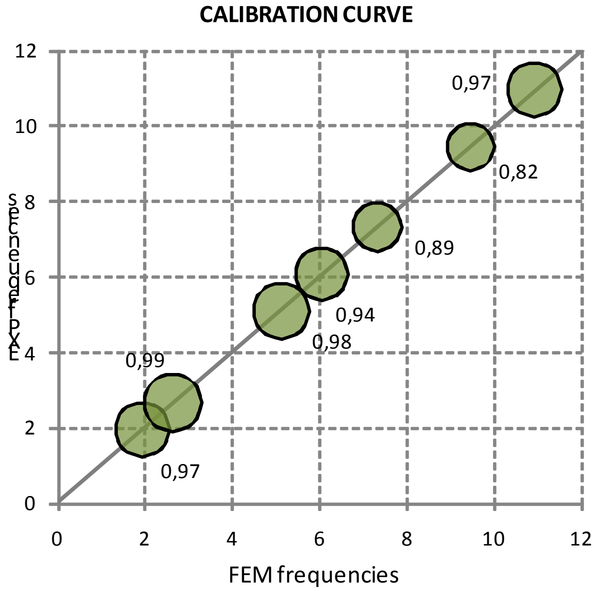


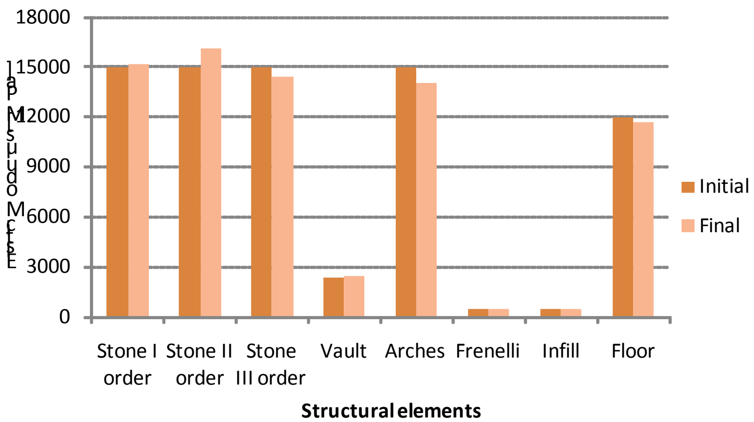
Fig. 8.15 - Calibration curve: comparison between numerical and experimental frequencies and MAC results

The calibration process caused the variation of the elastic properties of the structure in terms of elastic modulus and mass density. From the comparison between the updating parameters of materials, before and after the calibration (Tab. 8.13) it is possible to identified parameters that have been changed more to match better experimental and numerical frequencies. As expected the 1st order of arches at the base of the wing do not have a great influence on the dynamic response, whereas the 2nd and the 3rd orders are much more sensitive to changes both in terms of stiffness and mass.

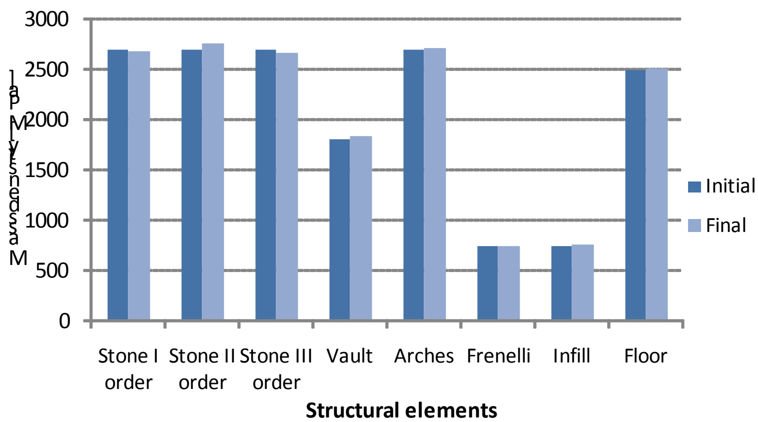
Also the vaulted structure that connect the wing at the first level with the amphitheater plays an important role in the dynamic response of the entire structure, acting as a spring element that connect the two bodies, whose stiffness variation is very sensitive to the overall dynamic behavior.

Tab. 8.13 - Variation of updating parameters related to the various structural elements

Structural element	ELASTIC MODULUS [MPa]			MASS DENSITY [kg/m^3]		
	Initial	Final	Diff. [%]	Initial	Final	Diff.[%]
Stone I order	15000	15223	1.49	2700	2687	-0.48
Stone II order	15000	16174	7.82	2700	2752	1.92
Stone III order	15000	14443	-3.71	2700	2658	-1.56
Vault	2400	2479	3.27	1800	1830	1.64
Arches	15000	14096	-6.03	2700	2703	0.12
Frenelli	500	477	-4.63	750	750	-0.04
Infill	500	483	-3.48	750	757	0.92
Stone floor	12000	11723	-2,31	2500	2509	0.36



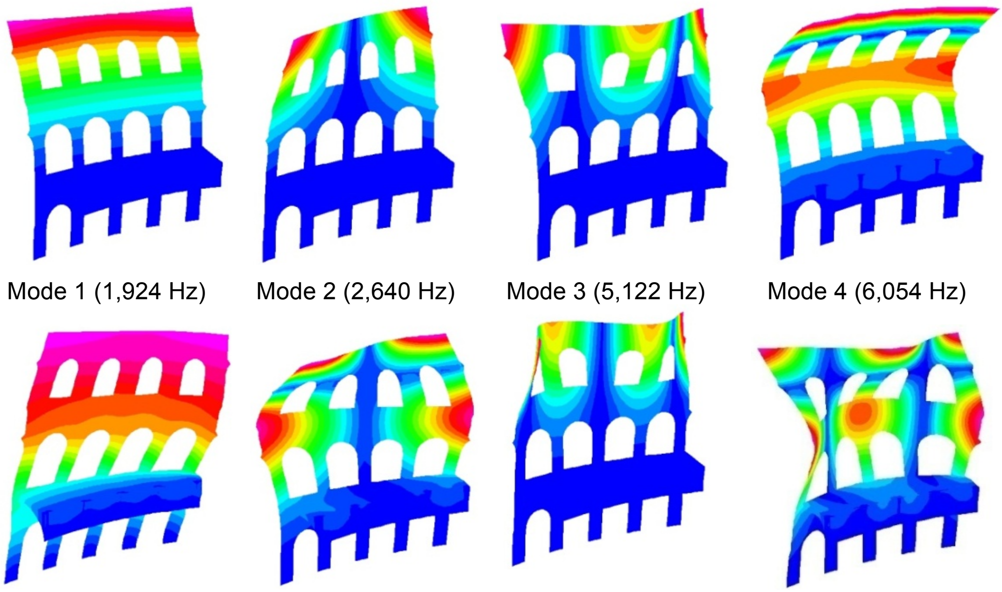
a)



b)

Fig. 8.16 - Variation of the elastic modulus (a) and mass density (b) for each identified structural element

To conclude Fig. 8.17 presents the tuned numerical mode shapes of the structure after model updating. It is worth noting that the numerical analyses allows identifying an additional mode, the 5th one, which is characterized by a global in-plane bending mode shape. This vibration mode was not included in the model updating process since it was not identified during Ambient Vibration Test, in which only out-of-plane sensors were implemented.



Mode 1 (1,924 Hz) Mode 2 (2,640 Hz) Mode 3 (5,122 Hz) Mode 4 (6,054 Hz)

Mode 5* (6,728 Hz) Mode 5 (7,323 Hz) Mode 6 (9,464 Hz) Mode 7 (10,944 Hz)

* in-plane bending mode not identified during AVT and dynamic monitoring

Fig. 8.17 - Numerical mode shape of the Arena's wing after model updating

The tuned simplified FE model was then used to perform advanced seismic analyses, reported in WP9, NIKER Project (2012).

8.4 SHM in case of exceptional events

SHM implemented on CH structures proved to be a fundamental tool, not only to control the operational conditions of buildings, but also to capture the response in case of exceptional events, especially in seismically prone areas. On the one hand the presence of a static system that controls crack patterns or specific collapse mechanisms that might be potentially activated by an earthquake, plays an important role in case of a seismic event, even of moderate intensity, as an early warning tool to control if the structure undergoes permanent and irreversible damages.

On the other hand the application of OMA techniques and dynamic monitoring (both continuous or trigger-based) permit to extract and identify the dynamic properties of the structure during the seismic event. The distinction and comparison of the dynamic response under operational conditions or in case of relevant events becomes crucial for the structural characterization, which can be also considerably different when the building is subjected to vibrations of large amplitude. Moreover in post-seismic scenarios, a continuous dynamic monitoring can be implemented to characterize the marginal response of damaged (or even severely damaged) structures under the effect of meaningful aftershocks.

Here, a representative example of this kind of application is reported.

8.4.1 Application to the Arena of Verona

8.4.1.1 Analysis of ground motion records

This paragraph reports how the SHM system installed on the Arena of Verona was able to capture a series of low/moderate seismic events and shows how the results have been exploited to assess the structural response and the safety conditions of the monument. Thanks to the monitoring system it was possible in fact to capture every single event and control the response of the structure during earthquakes, evaluating possible induced damages.

Starting from January 2012, northern Italy was hit by a large number of earthquakes, also of relevant magnitude, whose epicenters were located less than 200 km from the city of Verona. These intense seismic activities culminated with the strong earthquakes of the 20th and 29th May 2012 in Emilia-Romagna region.

In the context of the present research we will focus on the seismic events occurred from the 24th January to the 1st February 2012 in the surrounding areas of Verona and on the 29th of May, the second main shock of the Emilia-Romagna earthquake.

The effects of those earthquake produced in fact the strongest vibrations on the monitored structure.

The first step in the analysis of the seismic traces is trying to find possible correlations between earthquake time histories recorded by the official Italian Accelerometric Network (RAN) of the INGV research institute and data collected by the monitoring system. The presence of a dynamic monitoring system gives in fact crucial indications on site effects and allows characterizing the amplification factors of both soil and structure itself.

Five main earthquakes were recorded during the analyzed period, each of one followed by smaller aftershocks (Tab. 8.14). The first earthquake (1), occurred in the province of Verona (11km north from the Arena, 10 km deep, magnitude 4.2 on the Richter scale), induced the strongest response in terms of maximum amplitude of vibrations on the structures of the Arena. The other four seismic events with a higher epicenter distance (Reggio Emilia (2) - 75 km from the Verona, Parma (3) - 130 km from Verona, Finale Emilia (4) 80 km from Verona, Medolla (5) - 70 km from Verona), although characterized by a bigger magnitude, induced lower accelerations on the monument due to the greater distance from the epicenter, the deeper position of the fault and the different frequency content of the main shock.

Tab. 8.14 - Major seismic events occurred in the northern part of Italy in January and May 2012 (source: ISIDE - Italian Seismological Instrumental and Parametric Data Base - INGV)

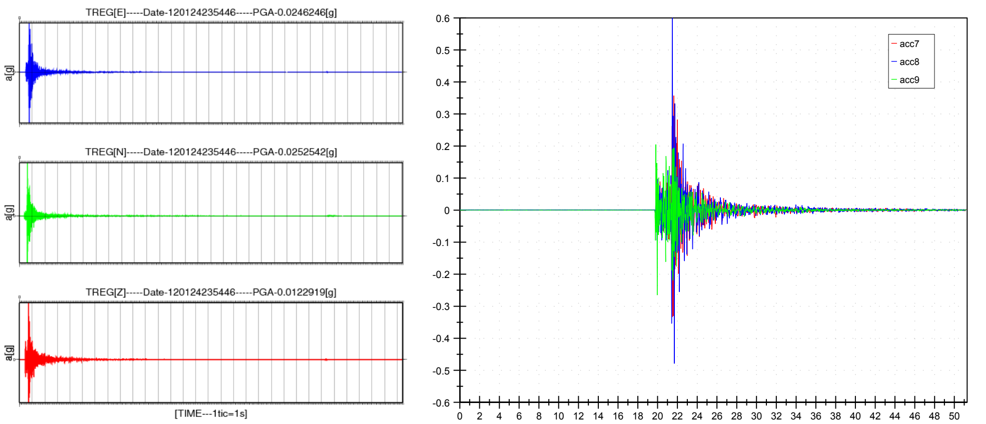
Seismic events	UTC	Magnitude	Depth	GPS Coordinates	
				Latitude	Longitude
1	2012-01-24 23:54:46	4.2	10.3	45.541	10.973
2	2012-01-25 08:06:36	4.9	33.2	44.854	10.538
3	2012-01-27 14:53:13	5.4	60.8	44.483	10.033
4	2012-05-20 02:03:53	5.9	6.3	44.890	11.230
5	2012-05-29 07:00:03	5.8	10.2	44.851	11.086

The first operation on the seismic trace recorded during the first earthquake on 24/01/2012 at 23:54:46 UTC (00:54 25/01/2012 of Italian time) was to compare the values of base acceleration recorded by the monitoring system with those recorded by the strong motion station of the Italian Accelerometric Network (RAN) of the INGV research institute in Tregnago, approximately 15 km from the epicenter.

Tab. 8.15 and Fig. 8.18 show a comparison between time histories of the earthquake recorded by the Tregnago INGV station and by the monitoring system.

Tab. 8.15 - Comparison between Peak Ground Accelerations (PGA) of the 25/01/2012 earthquake recorded by the INGV strong motion station and the Arena monitoring system

TREGNAGO INGV STATION			MONITORING SYSTEM		
PGA [cm/s^2]	Component	Distance [km]	PGA [cm/s^2]	Component	Distance [km]
24.1567	Horiz. E	15.4	61.9	Horiz. X	about 11
24.7744	Horiz. N	15.4	35.7	Horiz. Y	about 11
12.0584	Vert. Z	15.4	26.5	Vert. Z	about 11



a) b)

Fig. 8.18 - Comparison between time histories of the 25/01/2012 earthquake recorded by the INGV strong motion station (a) and by the monitoring system of the Arena (b)

It is possible to note that the maximum PGA recorded by the monitoring system is almost twice that recorded by the INGV station, both for the horizontal and the vertical components. This phenomenon is probably due to the fact that the Arena is closer to the epicenter of the earthquake and the site effects are significantly higher. Regarding the latter aspect it is necessary to point out that the INGV station is a free field station located in the countryside, whereas the soil underneath the Arena's foundations is characterized by several historic stratifications (with several archeological ruins) and by a system of underground tunnels used in the past as sewage plants. Those factors certainly contributed in the amplification of the seismic waves.

The maximum PGA recorded at the base of the monument was equal to 0,619 m/s^2; vibrations transmitted to the structure produced a maximum acceleration of 1,93 m/s^2 at the top of the Arena's wing (which is the most flexible structural element) with an amplification factor of about 3,11 and an acceleration of 1,251 m/s^2 at the top of the amphitheater with an amplification factor of 2,02.

The elaborated elastic response spectra of the recorded seismic event (Fig. 8.19), assuming a damping coefficient $\xi = 5\%$ indicates a local resonance with high peaks in the frequency range 5-10 Hz, where several structural eigenfrequencies of the wing are present.

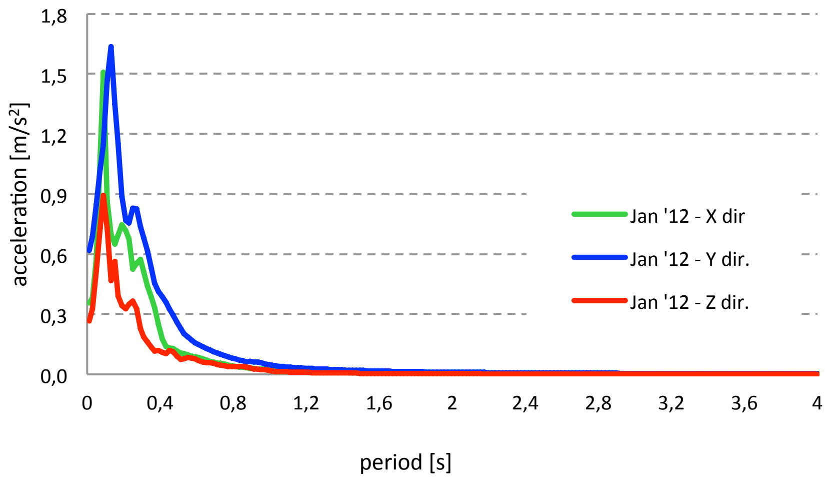
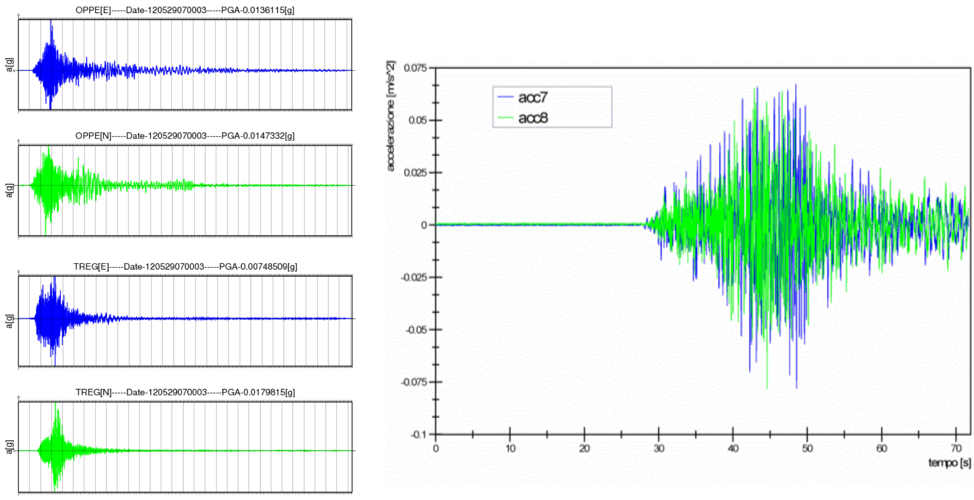


Fig. 8.19 - Elastic response spectra (ξ=5%) of the seismic event for the three directions

The second analyzed seismic event is the one occurred on the 29th of may 2012, magnitude 5.8 with an epicenter at about 75 km from the city of Verona. Also in this case it was possible to compare the maximum accelerations recorded by the dynamic monitoring system of the Arena and the ones recorded by three INGV strong motion stations near Verona (Fig. 8.20 and Tab. 8.16).

Tab. 8.16 - Comparison between Peak Ground Accelerations (PGA) of the 29/05/2012 earthquake recorded by three INGV strong motions station near Verona and the Arena monitoring system

Station	Component	Distance from epicenter [km]	Maximum PGA [cm/s^2]
Arena SHM	Horizontal	70	7,8
Oppeano INGV station	Horizontal	50	14
Tregnago INGV station	Horizontal	80	18
San Zeno INGV station	Horizontal	90	5,6



b)

a)

Fig. 8.20 - Comparison between time histories of the 29/05/2012 earthquake recorded by the INGV strong motion stations (a) and by the monitoring system of the Arena (b)

In this case it is possible to observe that the recorded PGA in the analyzed stations are rather homogeneous. The maximum acceleration recorded in the Tregnago strong motion station (the closer INGV station to the Arena of Verona) is more than twice the value recorded by the dynamic system of the Arena.

The main shock of the earthquake induced a maximum acceleration at the top of the Arena's wing of about 0,98 m/s^2 and at the top of the amphitheater of about 0,40 m/s^2, with amplification factors of 12,56 and 5,13 respectively.

Tab. 8.17 - Comparison between maximum accelerations and amplification factors recorded by the Arena monitoring system during the 25/01/12 and 29/05/2012 earthquakes

	BASE	TOP OF ARENA'S WING		TOP OF AMPHITHEATER	
Seismic event	PGA [m/s^2]	Max. Acceleration [m/s^2]	Amplification factor	Max Acceleration [m/s^2]	Amplification factor
25/01/2012 - Prealpi Venete district	0,619	1,93	3,11	1,251	2,02
29/05/2012 - Emilia-Romagna	0,078	0,98	12,56	0,40	5,13

Tab. 8.17 shows a comparison among maximum accelerations and amplification factors recorded by the SHM system of the Arena of Verona in case of the two main

seismic events analyzed within this section. It is evident that the 25/01/2012 earthquake (near-source event) induced higher absolute vibrations on the structures of the monument due to the closer distance of the epicenter. However in terms of amplification factor the 29/05/2012 earthquake (far-source event) produced a significant higher structural amplification both on the amphitheater itself and on the Arena's wing due to the different frequency contents of the inputs, as demonstrated by the comparison of the elaborated elastic response spectra of the two earthquakes (Fig. 8.21).

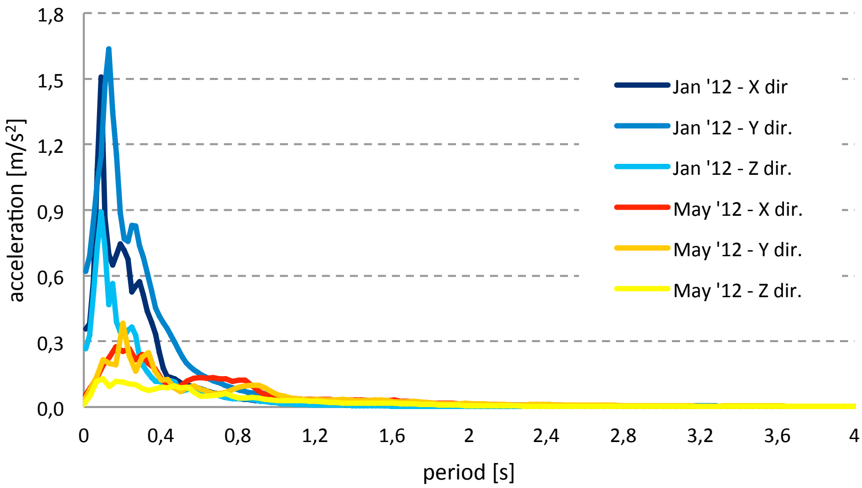


Fig. 8.21 - Comparison of the elastic response spectra (ξ=5%) of the two seismic events for the three directions

8.4.1.2 Modal parameters identification

In parallel to the study of the structural response of the building in terms of maximum accelerations recorded at the base and at the top of the monument, evaluating also the amplification factors with the height, it was decided to study also the variation of the modal parameters during the seismic event. This aspect becomes crucial to characterize the dynamic response of the structure when subjected to vibrations of large amplitudes compared with the ones induced by ambient excitation sources.

Data collected by the SHM system of the Arena during the 25/01/2012 earthquake (Ml 4,2) (Tab. 8.15 and Fig. 8.18) were used to carry out a dynamic identification of modal parameters before, during and after the seismic event. This event induced in fact the highest accelerations on the Arena. The final aim is to evaluate possible changes in modal properties of the structure during and after the earthquake, connected to permanent damages induced by the seismic event. A parallel aim is the assessment of the performance of dynamic identification techniques in case of transient input signals such as quakes.

In this framework the application of OMA techniques seems to be not reliable because most likely relevant nonlinear phenomena occurred on the structure due to the seismic input. Moreover such techniques are based on the assumption that the input is a white noise stochastic process. This hypothesis is certainly fulfilled in the presence of stationary or weakly nonstationary signals, but seem to be ineffective in the case of heavily nonstationary signals, as reported by Rainieri *et al.* 2010. This is because the frequency spectrum of the transient input shows very high amplitudes at particular frequency values which bias the modal parameter estimation in output-only conditions.

In order to overcome this problem it was decided to apply classical (input-output) experimental modal analysis (EMA), since both the input at the base and the output response are measured by the SHM system. Following this approach forced vibrations usually induced by mechanical devices or actuators in EMA are here substituted by the recorded earthquake, implementing a full scale dynamic test on the wing of the Arena (Fig. 8.22).

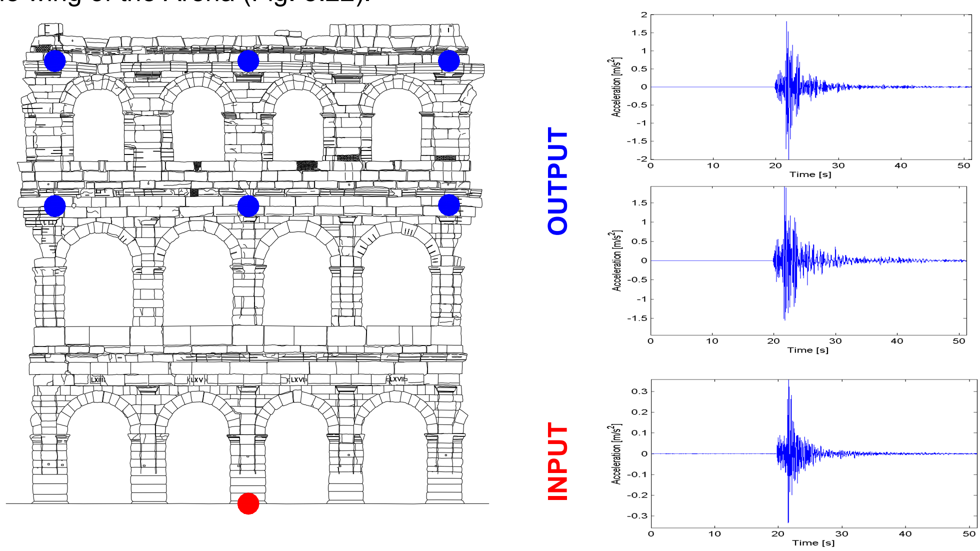


Fig. 8.22 - Input-output dynamic identification of modal parameters of the Arena's wing during the 25/01/2012 earthquake

Dynamic identification of modal parameters was performed in MACEC 3.2, where classical EMA is implemented with nonparametric frequency response function (FRF) estimation using the classical H_1 estimator (Heylen *et al.* 1997) and with the deterministic pLSCF method, which is a parametric method that starts from a nonparametric FRF description. In EMA, the influence of the unmeasured ambient forces is considered as disturbing noise; it is removed in the nonparametric FRF estimation.

Another possibility is to implement a combination of experimental and operational modal testing where the measured ambient loads are not considered as unwanted noise, but as a useful part of the excitation. Combined vibration testing has raised interest only recently, since it requires special system identification methods. One of these methods is the data-driven reference-based Combined deterministic-stochastic Subspace Identification (CSI/ref) method, which is also incorporated into the MACEC software. The interested reader can delve into the theoretical formulation of this method in Reynders *et al.* 2010 and Reynders & De Roeck 2008.

In the present application the combined experimental-operational approach was selected to identify the modal parameters of the Arena's wing during the earthquake.

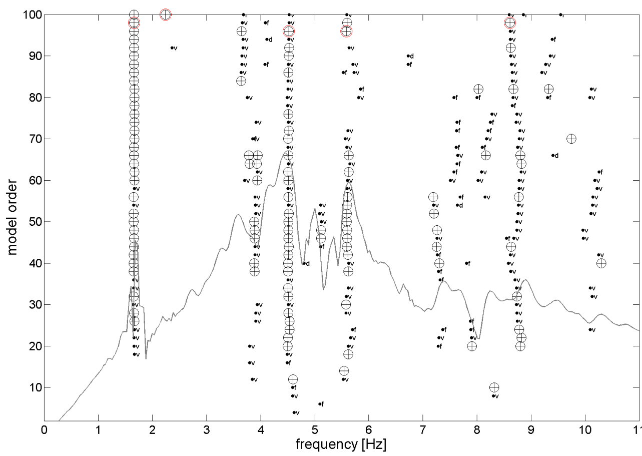


Fig. 8.23 - Stabilization diagram constructed from state-space systems identified with CSI/ref method. FRF estimated from input-output data is also presented.

This choice was made considering that during the main part of the recorded seismic input (characterized by higher ground accelerations) the transmitted forces are certainly greater than the amplitude of the ambient forces and the response of the structure is dominated by the amplitude and the frequency content of the earthquake. However also the beginning and the end of the seismic event are taken into account in the system identification process. In those phases ambient vibrations can influence the response and the implementation of combined vibration testing allows taking into account the ambient loads as part of the excitation, as previously described.

Fig. 8.23 shows the stabilization diagram constructed from state-space system identified with the CSI/ref method. The diagram consists of vertical lines of stable poles, that are present especially in correspondence to the most excited modes (e.g. mode #1, #4, #5, #7), as clearly demonstrated by the resonant peaks of the sum of Frequency Response Functions plotted on top of stabilization diagram.

The following cases have been considered for the analysis:

- Before earthquake (BE): data collected during 9 days before the seismic event
- Main shock (MS): dataset containing the transient signal due to the ground motion induced by the earthquake
- Post-peak (PP): data related to the final part of the earthquake's time history recorded by the system
- After earthquake (AE): data collected during the 9 days after the seismic event

Results obtained from the four cases are reported in terms of natural frequencies variation in Tab. 8.18, Fig. 8.24 and Fig. 8.25 and in terms of modal damping ratio variations in Tab. 8.19, Fig. 8.26 and Fig. 8.27. In cases BE and AE output-only identification techniques were used and the mean values of the extracted modal parameters during the 9 days are considered.

Tab. 8.18 - Natural frequency identification before (BE), during (MS and PP) and after (AE) the earthquake

FREQUENCY VARIATIONS							
MODE	BE [Hz]	MS [Hz]	PP [Hz]	AE [Hz]	f change (BE-MS)	f change (BE-AE)	MAC $(\{\psi^{BE}\},\{\psi^{MS}\})$
1	1,98	1,66	1,73	1,89	-16,28%	-4,44%	0,9998
2	2,75	2,24	2,35	2,62	-18,63%	-5,11%	0,9664
3	5,31	n.i*	4,50	4,97	/	-6,94%	/
4	6,44	4,52	5,29	6,07	-29,77%	-6,09%	0,9933
5	7,57	5,59	6,28	7,10	-26,15%	-6,55%	0,9372
6	10,00	n.i.*	n.i*	9,18	/	-8,89%	/
7	11,40	8,62	9,71	10,67	-24,34%	-6,78%	0,9581

*not identified

A very high correlation has been observed in terms of mode shapes estimated before ψ^{BE} and during ψ^{MS} the earthquake, as pointed out by very high values of the MAC index. The evaluation of the MAC values allows understanding if a peak in the estimated FRF is related to an actual vibration mode of the structure rather than to the frequency distribution of the input. Given the high correlation between mode shapes it is possible to state that the application of input-output identification methods leads to a correct identification of the structural modes during the earthquake, and, as consequence, of the frequency and damping shifts. It was not possible to identify two vibration modes, i.e. mode #3 and #6.

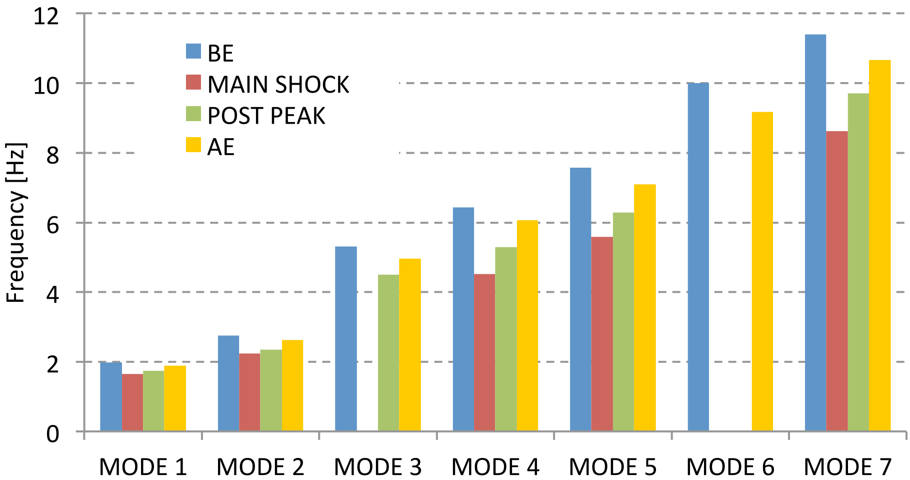


Fig. 8.24 - Natural frequencies variation before (BE), during (main shock and post-peak) and after (AE) the earthquake

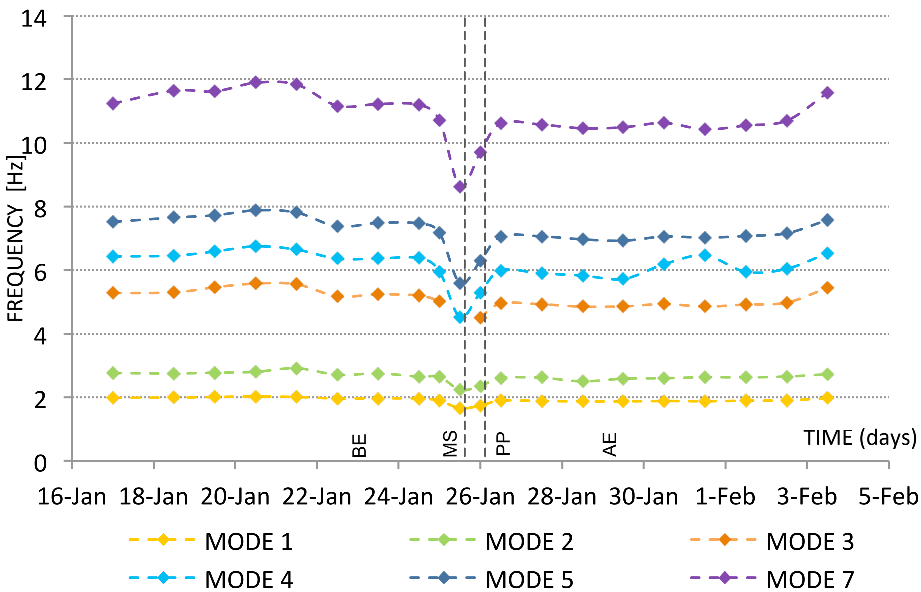


Fig. 8.25 - Evolution of natural frequencies 9 days before the earthquake, during the main shock ant the post peak phase and 9 days after the earthquake

The analysis of modal identification results show a significant decrease of all natural frequencies during the earthquake, with a percentage change ranging from 16% to 30%, as clearly showed also by Fig. 8.24 and Fig. 8.25.
This phenomenon is certainly related to the amplitude of vibrations transmitted to the structure during the earthquake, much higher than those induced by ambient forces. Moreover it is likely that a masonry structure like this presents a slight non linear behavior even if subjected to moderate seismic loads. Natural frequencies tend to recover to the original values during the post-peak phase (end of the earthquake), when the transmitted accelerations are subjected to a significant decrement. Comparing the situations before and after the seismic event it is possible to note a slight permanent decrement of frequencies, which is, however, not related to structural damages, as confirmed by careful inspections of the structure after the earthquake.
This frequency decrement can be related to the fact that the seismic waves lead to a change in the degree of interaction between stone blocks or to a temporary modification of the soil characteristics, in terms of compactness. In any case analyzing the results of automated identification of modal parameters in the successive period one cannot note a permanent modification of the dynamic properties, demonstrating that the earthquake did not induced significant damages to the structure.
Modal damping ratios underwent similar changes during the earthquake, as reported in Tab. 8.19, Fig. 8.26 and Fig. 8.27.

Tab. 8.19 - Modal damping ratios identification before (BE), during (MS and PP) and after (AE) the earthquake

DAMPING RATIO VARIATIONS						
MODE	BE [%]	MS [%]	PP [%]	AE [%]	ξ change (BE-MS)	ξ change (BE-AE)
1	1.17	2.71	2.18	0.96	+131.47%	-22.25%
2	1.11	5.11	2.67	0.82	+361.43%	-35.21%
3	1.03	n.i*	1.30	0.96	/	-7.10%
4	6.44	1.97	3.75	4.87	-69.45%	-32.23%
5	2.81	6.64	4.38	2.30	+136.19%	-22.44%
6	0.95	n.i*	n.i*	0.99	/	+3.71%
7	1.34	3.57	1.93	1.19	+166.63%	-12.51%

*not identified

It can be noted a spectacular increment of damping during the main shock for modes #1, #2, #5 and #7, meaning that the dissipation mechanisms of the structure have been fully activated, especially in correspondence to those modes, particularly excited by the seismic action. Mode #4, usually characterized by rather high values of damping under operational conditions, in contrast with the other modes, presented a decrement (-70%) during the earthquake, probably due to the fact that this mode was not excited so much during the earthquake.

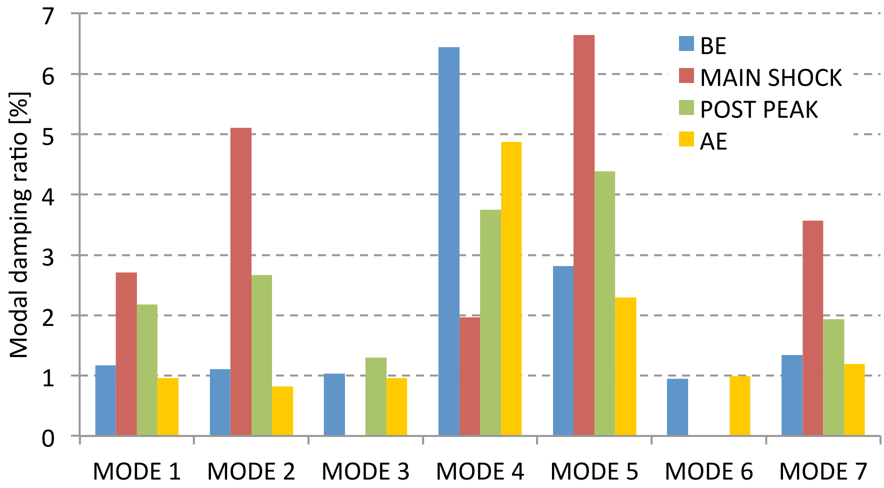


Fig. 8.26 - Damping ratios variation before (BE), during (main shock and post-peak) and after the earthquake

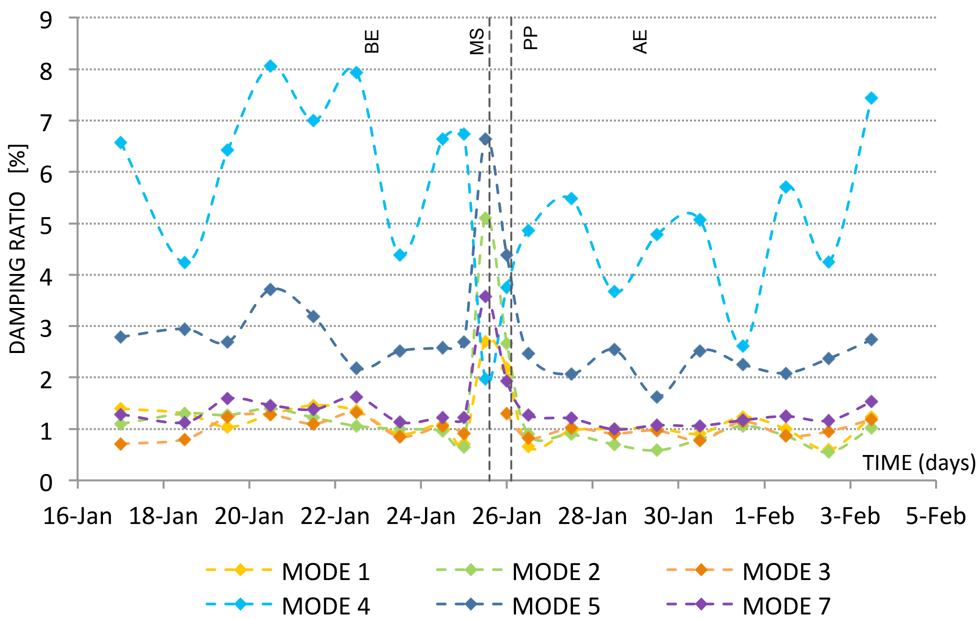


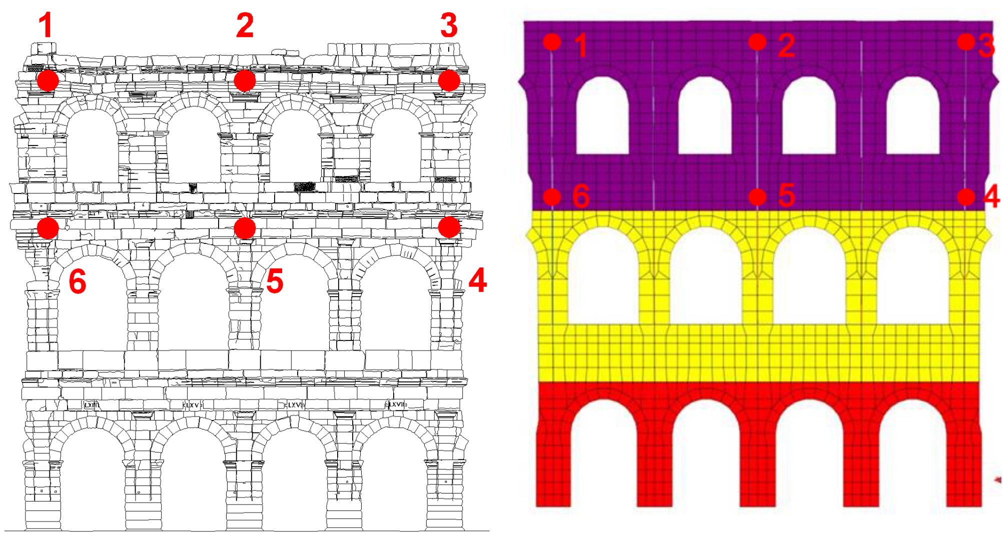
Fig. 8.27 - Evolution of damping ratios 9 days before the earthquake, during the main shock ant the post peak phase and 9 days after the earthquake

Generally speaking one can observe that damping coefficient reached during the earthquake the values that usually characterized masonry structures (3%-6%). This is often accompanied with a non-linear softening structural behavior appreciable even for rather low displacements. Damping ratios tend to recover to the initial values (BE) immediately after the earthquake, even if, as for frequencies, a slight permanent decrement was noted. It is necessary to point out that damping estimation present always several uncertainties, especially when it is identified from ambient vibration tests (Fig. 8.27).
In conclusion it is possible to state that the nonlinear response of the structure, especially next to resonance, and the larger amplitude of vibrations induced by the earthquake are the major responsible of the great variation of modal parameters during the seismic event.
Thanks to earthquake records, even of low/moderate intensity, provided by an active SHM system, the structural response under exceptional events can be evaluated in detail and compared with the operational conditions, understanding much better the dynamic properties and the seismic behavior of the investigated buildings. The recorded seismic traces can furthermore be exploited to simulate a real earthquake on a reference FE model, as reported in the following paragraph.

8.4.1.3 Numerical simulation

In order to refine the calibration of the reference FE model of the wing of the Arena, presented in §8.3.1.2, a dynamic analysis in the time domain was performed by analyzing the response of the numerical model subjected to accelerograms corresponding to the major earthquake of 25/01/2012 captured during the monitoring period. The dynamic response actually displayed by the building during the earthquake, regarding displacements, damage and possible collapse mechanisms, is then compared with the numerical prediction and again the model is improved until satisfactory agreement is obtained.
The procedure consisted in applying the base acceleration in the two orthogonal directions recorded by the system to the nodes at the FE model's base. Subsequently the accelerations extracted from the numerical calculations are compared with the experimental accelerograms of the monitoring system recorded in the same positions. This analysis allows refining the calibration of the FE model to a real earthquake and updating the elastic mechanical properties of the structure in order to obtain comparable values of acceleration.
Moreover it is possible to evaluate and accurately calibrate the damping coefficient, based on the estimation of this modal parameter performed during the captured

seismic event, in order to take into account the actual dissipation mechanisms showed by the structure.



a) b)

Fig. 8.28 - Positions of the accelerometers in the real structure (a) and nodes selected on the FE model (b) for the comparison of earthquake's recorded and calculated time histories

In general the FE method requires the definition of specific damping coefficient for each identified material that composes the model, adopting the formulation of the Reyleigh damping, proportional to the mass and stiffness matrices:

$$\boldsymbol{C} = a\boldsymbol{M} + b\boldsymbol{K} \tag{8.10}$$

where:

a is the mass proportional Reyleigh damping coefficient

b is the stiffness proportional Reyleigh damping coefficient

$\boldsymbol{C}$, $\boldsymbol{M}$ and $\boldsymbol{K}$ are damping, mass and stiffness matrices respectively

Usually the coefficients a and b are calculated imposing an equivalent damping ratio ξ equal to 5% in correspondence to the first and last frequencies of the structure, calculated at the achievement of 85% of participating mass.

In this case, since actual modal damping ratios estimation are available from input-output dynamic identification during a real earthquake (§8.4.1.2), those data are used to calculate the Reyleigh damping coefficients.

The next step is the implementation of time-history dynamic analyses applying the recorded earthquake at the calibrated FE model of the Arena's wing. A preliminary linear dynamic analysis was performed, under the hypothesis of linear elastic behavior of materials. Afterwards non-linear properties of constitutive materials

have been defined and applied and a series of non-linear transient analyses have been performed to compare the response of the structure with the linear model outputs and the actual experimental records, in terms of maximum acceleration and damping behavior.
FE analyses, both linear and non linear were performed using *DIANA* finite element software (DIANA™, TNO, Delft, *release 9.4.4*, 2012), where several non linear constitutive models, also for quasi-brittle material like masonry, are available. In the present case study a total strain rotating crack model (for details see the software user manual) is chosen. Masonry is assumed to have a constant behavior in tension with infinite value of fracture energy (G_f) and a linear hardening behavior in compression. The choice of a tensile elasto-plastic law was made due to convergence problems when implementing a linear-softening behavior with finite value of fracture energy.

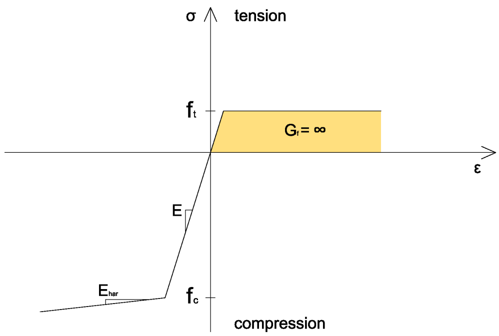


Fig. 8.29 - Non linear constitutive model of masonry implemented during non linear dynamic analyses

Masonry is simplified to be an isotropic material although it is known to possess anisotropic properties due to the arrangements of stone units and joints. Mechanical properties of the reference homogenized masonry materials are summarized below in Tab. 8.20.

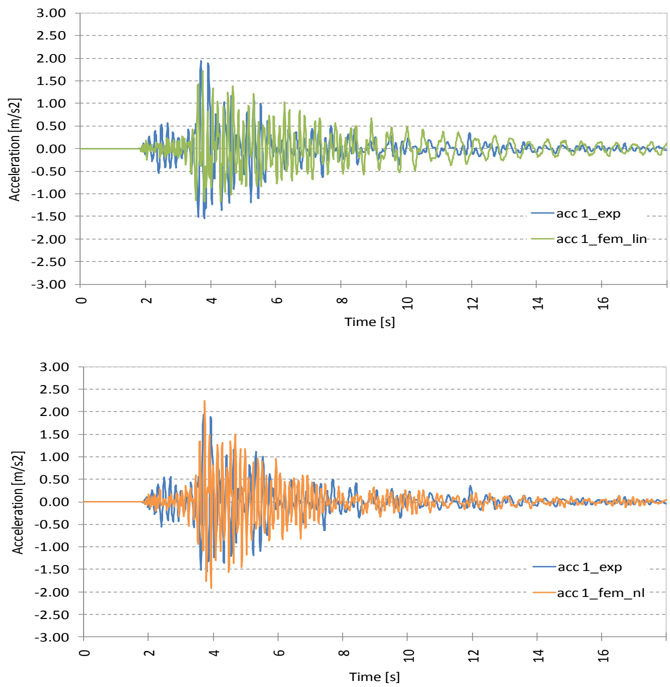
Tab. 8.20 - Non linear mechanical properties assigned to materials

Material	Tensile strength f_t [MPa]	Fracture energy G_f [N/mm]	Compressive strength f_c [MPa]	Elastic Hardening E_{har} [MPa]
Stone blocks masonry	0,13	∞	3,00	3,00
Opus coementicium (vaults and arches)	0,13	∞	3,00	3,00
Infill of vaults			linear elastic	
Stone floor			linear elastic	

Non linear properties are also assigned to steel post-tensioned cables along the massive pillars of the wing, inserted to simulate the strengthening intervention performed during the ‘60s. In this case an elastic-perfectly plastic behavior was assumed, with a reduction of the elastic modulus to 150000 MPa to account for the not perfect bond of steel cables with reinforced concrete.
The results and comparison between experimental, linear and non linear models will be presented in terms of acceleration time histories calculated on the FE model and recorded on the same positions by sensors on the real structure.

Tab. 8.21 - Comparison between maximum accelerations experimentally recorded and numerically calculated

Model	III order		II order	
	Max Acc. 1 [m/s^2]	Max Acc. 3 [m/s^2]	Max Acc. 5 [m/s^2]	Max Acc. 6 [m/s^2]
Experimental	1,934	1,840	1,670	1,817
FE linear	2,253	2,06	1,330	2,130
FE non linear	1,702	1,86	1,252	1,195



a)

b)

Fig. 8.30 - Accelerometer 1: time series recorded by the SHM system (blue) and numerically calculated for the linear (a) and non linear (b) model

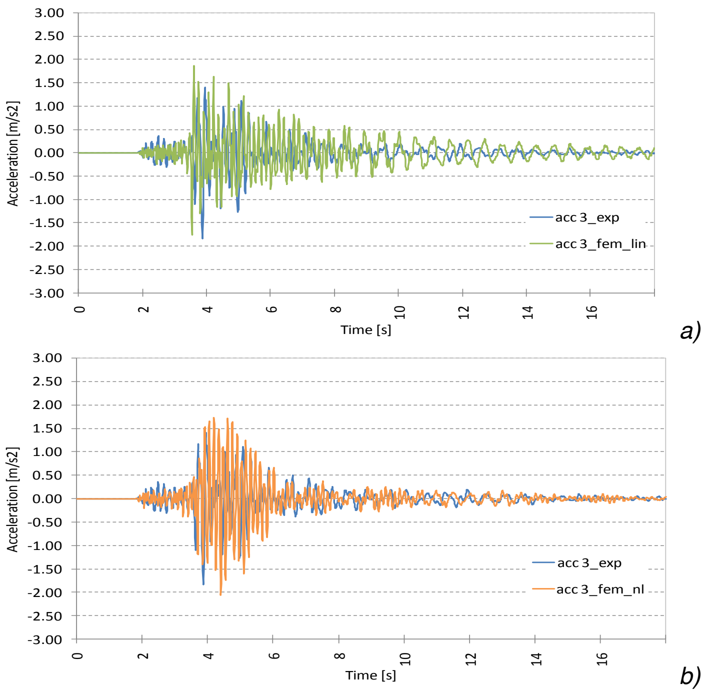


Fig. 8.31 - Accelerometer 3: time series recorded by the SHM system (blue) and numerically calculated for the linear (green) and non linear (orange) model

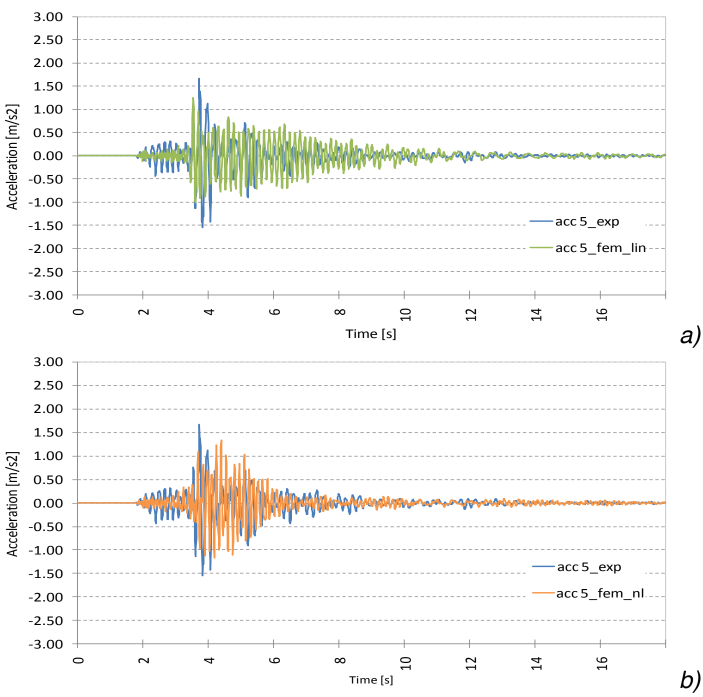
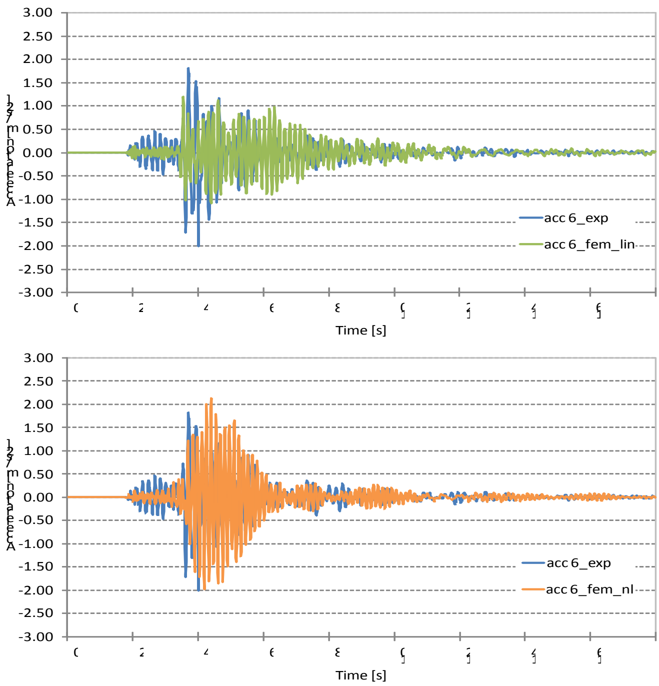


Fig. 8.32 - Accelerometer 5: time series recorded by the SHM system (blue) and numerically calculated for the linear (a) and non linear (b) model



a) b)

Fig. 8.33 - Accelerometer 6: time series recorded by the SHM system (blue) and numerically calculated for the linear (green) and nonlinear (orange) model

Analyzing maximum values of acceleration (Tab. 8.21) recorded on the structure and calculated on the FE model (both linear and nonlinear) it is possible to note a good correspondence of experimental results with numerical outcomes, meaning that the calibrated FE models represent sufficiently well the actual response of the structure.

From Fig. 8.30 to Fig. 8.33 the comparison between experimental and numerical accelerations during the main shock of 25/01/2012 earthquake are reported. In general the actual behavior of the Arena's wing is better described by the nonlinear model, as reported in Tab. 8.22, where an objective criterion of comparison is applied. The criterion, based on the least square method, allows computing the percentage of the experimental accelerogram that is explained by the prediction of the corresponding numerical output through the fit index given by:

$$fit = 100\frac{1 - norm(y_{fem} - y_{exp})}{norm(y_{exp} - \bar{y}_{exp})} \tag{8.11}$$

where:

y_{exp} is the experimentally measured output, $\bar{y}_{exp}$ is the mean

y_{fem} is the numerically calculated response

Tab. 8.22 - Fit index calculated between experimental and numerical accelerograms related to the response of the wing during the earthquake

Position	FE model type	Time history	2-10 sec	2-8 sec
Acc. 1	Linear	41,32	37,49	36,37
	Non linear	45,18	45,09	45,76
Acc. 3	Linear	67,60	62,77	60,70
	Non linear	76,53	77,17	78,02
Acc. 5	Linear	46,26	46,30	45,18
	Non linear	47,57	47,23	46,52
Acc. 6	Linear	34,11	33,76	32,67
	Non linear	58,19	58,33	58,52

From the analysis of results of fit indexes it is possible to note an evident improvement of the nonlinear model prediction over the linear one. The higher fit coefficients, the higher the percentage of the experimental output explained by the numerical prediction. This means that the implementation of a nonlinear dynamic analysis represent better the actual behavior of the structure during the recorded earthquake. Results are consistent with the assumption that a masonry structure like this presents a marked nonlinear behavior even if subjected to low/moderate seismic actions.

8.5 Conclusions

This chapter established and defined a clear procedure of analysis of monitoring results, exploiting the outcomes to assess the structural conditions of monitored buildings.

Different approaches have been applied. Firstly the so called data-driven approach is tested to verify the possibility to apply robust statistical methods and damage detection algorithms to filter out environmental factors and identify the presence of ongoing damaging processes. The analysis of structures under operational conditions was performed by the application of regression analysis and black-box models. Single Input Single Output (SISO) ARX models were constructed for the l'Aquila case studies, considering two emblematic structures heavily damaged by the 2009 earthquake, i.e. the Civic Tower and the Spanish Fortress. Natural frequencies of the buildings were used for the application of the proposed methodology, since they provide information on the global structural behavior.

Promising results have been obtained. In one case (i.e. Spanish fortress) it was demonstrated that the damage patter induced by the earthquake is rather stable also thanks to the provisional strengthening measures applied immediately after the seismic event. In the other case (i.e. Civic tower) a significant variation of the equilibrium and boundary conditions of the structure were revealed during the monitoring period, combining and cross correlating static and dynamic information.

The developed and validated statistical models proved to be very effective to filter out environmental effects and detect ongoing damaging processes. Based on this conclusion it is certainly possible to extend the procedure to other monitored structures, even if not characterized by severe damage patterns

A second possibility to use effectively the monitoring results is found in the so called model-driven approach. Outcomes of monitoring (in terms of dynamic response) have be compared with numerical predictions resulting from a FE model. In particular, the modal matching procedure was considered for this purpose. It involves the upgrading of the model until a satisfactory agreement in terms of both frequencies and mode shapes is obtained. Model updating procedures were applied to the Verona's case studies, i.e. the Cansignorio stone tomb and the wing of the Arena.

Finally SHM proved to be exceptionally useful for the analysis of the structural response in case of exceptional events, such as earthquakes. Several low to moderate seismic events were captured by the Arena's monitoring system at the beginning of 2012. The recorded seismic traces have been successfully used and implemented for different but connecting purposes:

- Analysis of the ground motion records in terms of amplitude and frequency content and verification of the structural response in terms of maximum accelerations and amplification factors.
- Identification of the modal parameters of the structure during the event, implementing input-output identification methods. Comparison of the extracted modal parameters with the results of ambient vibrations.
- Numerical simulation of the recorded earthquake on the available FE model, in order to verify and refine the calibration. Comparison of the model response with the experimental one implementing both linear and nonlinear dynamic analyses.

9 CONCLUSIONS AND FUTURE WORKS

9.1 Introduction

The final chapter reports some general remarks and final observations arising from the research carried out during the PhD program.
This work provides a contribution on the development of techniques and integrated methodologies, based on Structural Health Monitoring, for the assessment and protection of Cultural Heritage structures.
The thesis was divided into four main parts: (i) a literature review on SHM applied to the general field of civil engineering structures and a presentation of the basic principles of structural dynamics and OMA. Those concepts represent the baseline of the entire research.; (ii) definition and proposal of a methodology for the implementation of monitoring techniques to CH buildings and real application and validation on selected case studies; (iii) development of automated subroutines and dedicated software for static and dynamic monitoring; (iv) definition and application of specific procedures, based on both data-driven and model-driven approaches to exploit the monitoring results for the assessment and protection of the architectural heritage.

9.2 SHM of CH buildings: methodological and applicative aspects

After a detailed state of the art review on specific topics related to SHM of civil engineering structures and Operational Modal Analysis, the thesis faces some methodological and applicative aspects about the implementation of SHM to Cultural Heritage structures:

(i) **Methodology**. A specific methodology is proposed, defining the role of SHM as a fundamental knowledge-based tool for the assessment and protection of historic buildings and monuments. The main aim is to transfer well-established monitoring techniques and methodologies in the field of conservation and restoration, starting from the outcomes of the European research project

NIKER. In the general framework for the study and assessment of historical construction, SHM is able to interconnect diagnostic investigations, structural analysis and interventions. Before the design and installation phase of a monitoring system it is of fundamental importance to execute a detailed historical research and implement a series of preliminary inspections (both NDT and MDT) to increase the knowledge level on the building and define the optimum layout of the system.

(ii) **Role**. The role of monitoring is more specifically identified in the framework of the complex activities carried out on historic buildings, from investigations to the execution of interventions, from the evaluation of the effectiveness of the proposed strengthening solutions to the ordinary maintenance activities. In each phase monitoring plays a fundamental and active role that is clarified and defined in detail.

(iii) **Conservation principles**. During the design process of the architecture and layout of a monitoring system and the selection of the optimum monitoring strategies it is necessary to define a set of requirements oriented to ensure its adequate performance and ability to provide valuable information. One of the most important requirements considered in the selection of the monitoring techniques is their adequacy for the application on heritage structures. Such adequacy results from some general criteria on conservation and restoration of architectural heritage that can be successfully transferred to the implementation of SHM systems.

(iv) **Objectives**. The application of monitoring techniques to specific problems is presented through their real implementation on several case studies, characterized by different construction typologies, local seismicity and seismic vulnerabilities, damage conditions and strengthening solutions. A general strategy to face different problems is developed and discussed, focusing in particular on the use of SHM to:

- Increase the knowledge on the structural behavior using SHM to assess strengthening needs and avoid the execution of unnecessary interventions;
- Apply an incremental approach to the execution of strengthening interventions using SHM before, during and after the implementation, validating eventually their effectiveness;
- Control the evolution of the damage state of buildings in a post-seismic scenario. The emblematic case of l'Aquila after the devastating earthquake in 2009 is exploited to show how monitoring can be successfully used to verify the effectiveness of provisional strengthening measures. SHM can

play an important role also during the entire management process of the reconstruction.

In this framework some representative case studies are presented and analyzed in detail in order to validate the entire methodology.
Thanks to this complex, but necessary process, it is possible to increase the performance and thus benefits of SHM applied to CH structures.

9.3 Development of automated algorithms for SHM

The central core of each monitoring system is the capability to automatically extract information from controlled parameters. Only in this way it is possible to apply subsequently specific damage detection algorithms to identify structural damages and provide reliable information on the health state. Without a stable automation and computerization of the monitoring process, able to provide almost real-time information on the monitored parameters (both static and dynamic), SHM becomes useless. In general, the development of effective methods of monitoring depends on two key factors: the signal acquisition technologies and the signal processing interpretation algorithms.
Within the present research project an automatic computerized system aimed at the improvement of data treatment and feature extraction was developed. It was necessary to implement a continuous and regular analysis of the acquired data allowing a constant control of the structural conditions and reducing as much as possible any kind of user interaction.
Robust and effective routines running on an online basis were developed in MATLAB environment and implemented in the management process of several SHM systems, currently active on a series of CH buildings and installed by the Department of Civil, Architectural and Environmental Engineering of the University of Padova. Both static and dynamic data, recorded by the acquisition units, are continuously retrieved through an Internet connection to the central server of the university and automatically processed on arrival.
The automated algorithms for data processing fulfill the following requirements:

- Automatic data processing and analysis to constantly control the monitored parameters and provide quickly useful and exploitable information on the health state of the structure;

- Generation of early warning messages if one or more parameters chosen to represent the safety level of the structure, exceed some acceptable predefined thresholds;
- Easy-to-use and open to the integration of new functions and constant updating;
- Interoperability with other information systems developed to support planning and management stages of the conservation/maintenance of structures or infrastructures.

The processing software developed within this thesis includes the online automatic processing of data measured by static systems and the automatic identification of modal parameters.

The algorithm for static data treatment and analysis is composed by a fully automated subroutine that processes row data on arrival at the central server of university. The algorithm pre-processes data and sends early warning massages. It is then complemented by a graphical user interface (GUI), simple and universally applicable to every monitoring system. It allows to post-process the acquired data, analyzing and correcting possible errors and cross correlating the outputs with environmental parameters.

The algorithm for automated Operational Modal Analysis is split into two phases: a subroutine is running on the central server and is devoted to the processing of recorded time series and the automatic identification of modal parameters in terms of natural frequencies, mode shapes and damping ratios. Then a Graphical User Interface (GUI) was developed in order to plot the results, create graphs and again correlate the extracted modal parameters with ambient factors.

Both static and dynamic algorithms for SHM have been validated through a real application on the selected case studies and they proved to be very efficient and reliable.

9.4 SHM for the assessment of CH buildings

The final part of the thesis concentrates on the analysis and exploitation of monitoring results. The starting points are features and information automatically extracted from the algorithms applied to the selected real case studies. The idea is to interpret and post-process those data in the general framework of increasing the

knowledge level of a historical building, trying to assess its static/seismic capacities and in case intervene through some structural improvements.
This fundamental objective was successfully achieved through the analysis of different but interconnected problems:

- Control the structural behavior of the monitored buildings under operational conditions;
- Calibrate and validate reference behavioral numerical models;
- Study and characterize the structural response in case of exceptional events.

➢ **Operational conditions**

The study of a monitored structure under operational conditions was performed by the application of regression analysis and black-box models, able to filter out environmental effects, exploiting a large number of observations to establish relations between extracted or recorded parameters (e.g natural frequencies) and ambient factors that might influence the response. Only in this way one can detect and identify damages that, otherwise, could be possibly masked by other phenomena. The proposed procedure involves different phases:
(i) Single Input Single Output (SISO) ARX models are constructed for the l'Aquila case studies, considering two emblematic structures heavily damaged by the 2009 earthquake, i.e. the Civic Tower and the Spanish Fortress. The final aim is to assess if the strengthening measures, immediately applied after the seismic event, are still effective or not.
(ii) Natural frequencies of the buildings were used for the application of the proposed methodology, since they provide information on the global structural behavior. Other modal parameters (such as damping coefficients), even if related to global properties and damage, are not taken into account since their accurate estimation is difficult and most of the times presents large variability, which makes this parameter unreliable for damage detection.
(iii) Features extracted from the first year of monitoring (estimation period) are implemented to establish statistical models able to described the behavior of the considered modal parameters. Afterwards, in the second step, the constructed model is exploited to predict the outputs in the following years (validation period), comparing the predicted response with the actual measured parameters.
(iv) The final phase consisted in the application of a residual analysis, able to detect the outliers and provide the identification of possible damages.

The application of such procedure led to promising conclusions about the possibility to apply successfully damage detection algorithms on historic masonry structures.
Thanks to the adopted statistical-based procedures it was possible in fact in one case to exclude the presence of ongoing damaging processes and in the other case to detect significant changes in the structural conditions of the building, cross correlating static monitoring outputs with the outcomes of modal parameters estimation.
It has been demonstrated that, combining monitoring results and applying robust data-driven approaches, one can control the long-term effectiveness of structural interventions. Similar statistical models can be constructed also to analyze the static response (e.g. opening/reclosing of cracks, inclinations, etc.), whose variations are strictly correlated to environmental factors.
From the experience with the two case studies, the proposed methodology for damage identification seems to be reliable and in general applicable to historic masonry structures, considering natural frequencies a trustworthy parameter for damage detection.
However it is necessary to state that the application of damage detection algorithms based on vibration signatures on masonry structures is a very complex task and not always successful. The main problems are related to the fact that masonry buildings are usually massive and sometimes the frequency content and the amplitude of ambient vibrations (exploited by dynamic monitoring) is not always sufficiently appropriate to excite uniformly all the structural modes in the frequency band of interest. Moreover damage on masonry buildings is usually spread and diffused, not concentrated on specific points or nodes as in the case of reinforce concrete or steel structures. This intrinsic characteristic of masonry structure certainly affect the possibility to apply extensively damage identification algorithms based on the variation of global parameters (such as natural frequencies and mode shapes). This opportunity is further complicated by the general poor quality of the estimate and the large fluctuations induced by environmental factors (in some cases between 20% and 30%) . Moreover, if damage in some cases can be successfully detected, as demonstrated by the l’Aquila case studies, its localization and quantification seem to be rather difficult at the state of the art.

- **Model updating**

The dynamic characterization of those type of buildings can be further exploited to calibrate, validate and continuously update reference numerical models, as demonstrated within this thesis for the case studies of the Cansignorio stone tomb and the Roman Arena of Verona. Outcomes of monitoring (both in terms of static measurements and dynamic response) can be compared with numerical predictions resulting from a FE model. In particular, the modal matching procedure was considered for this purpose. It involves the upgrading of the model until a satisfactory agreement in terms of both frequencies and mode shapes is obtained. The model improvements or updating may target to material properties, geometry and morphology of structural members, connections, influence of the soil and possible soil-structure interaction effects, influence of neighboring buildings or adjacent structures and damage distribution. The calibrated and validate FE models can be then used to perform advance structural analyses, based on the assumptions that the model is reliable and match almost perfectly the real conditions.

- **Exceptional events**

SHM implemented on CH structures proved to be a fundamental tool also to capture the response in case of exceptional events, especially in seismically prone areas.

This crucial aspect was confirmed by the analysis of the wing of the Arena of Verona during a series of low/moderate earthquakes captured by the monitoring system. On the one hand the presence of a static system that controls crack patterns or specific collapse mechanisms that might be potentially activated by an earthquake, plays an important role in case of a seismic event, even of moderate intensity, as an early warning tool to control when the structure undergoes permanent and irreversible damages. On the other hand the application of input-output dynamic identification techniques permit to extract and identify the dynamic properties of the structure during the seismic event. It was demonstrated that the comparison between dynamic response evaluated under operational conditions and in case of relevant events becomes crucial for the structural characterization, which can be considerably different when the building is subjected to vibrations of large amplitude. The analyses performed on the recorded seismic events reveal that both natural frequencies and modal damping ratios are extremely sensitive to the level of vibrations, showing a significant variation during the earthquake. Moreover it is clear that a masonry structure like the wing of the Arena presents a slight non linear behavior even when subjected to moderate seismic loads.

The recorded seismic traces have been finally exploited to simulate the real recorded earthquake on a reference FE model. The procedure allows refining the FE calibration, taking into account the actual damping coefficients and thus the dissipation mechanisms really showed by the structure. The comparison between numerically calculated accelerations with the experimentally recorded ones proved that a nonlinear FE model described better the response both in terms of maximum amplitude and in terms of post-peak behavior.

One of the most important conclusion that can be traced from this research experience is that SHM represents a necessary tool for the study and assessment of CH buildings. In order to achieve successfully this ambitious objective it seems that the multi-model approach is the key solution. This means that the implementation of rigorous statistical models based on data-driven approaches on the one hand and the construction and constant updating of reliable numerical models on the other hand provide sufficient tools to exploit and combine the results of SHM in the general framework of the assessment and protection of historical constructions.

9.5 Future works

The research carried out and described within this thesis represented an attempt to apply monitoring techniques and methodologies in the field of historic buildings and monuments. A further and continuous research effort is still requested in order to solve the open issues, obviously still present. On the light of the presented outcomes and conclusions some possible developments are listed hereafter:

- Keep monitoring strategies, techniques and equipment constantly up to date to exploit always the technological advancement especially in the field of advanced sensors, such as fiber optic and wireless sensors networks (WSN);
- Improve the developed automated algorithms for SHM, implementing new functions and enhancing the performance;
- Explore the possibility to apply further and more robust statistical techniques on extracted or measured data, implementing more reliable damage detection algorithms;
- Implement advanced numerical modes, using both the FE method and the discrete element method, combining and cross correlating the outcomes with the monitoring results;
- Direct a strong research effort on the SHM-related topics of data interpretation, emergency management, risk analysis and decision-making to maximize the benefits coming from the fast development of these technologies for the preservation of the architectural heritage;

- Use SHM as an opportunity to manage the emergency phases after exceptional events, such as moderate or major earthquakes, integrating actively some civil protection functions;
- Install diffused monitoring systems in historic city centers. The case of the city of Verona will be exploited as a pilot project, creating a real SHM network with the integration of the currently active monitoring systems (Arena, Cansignorio stone tomb, Ponte Nuovo) with other diffused monitoring on the Roman theater, Castelvecchio and Lamberti tower. The idea is to monitor the principle and strategic structures of the city, using SHM as an early warning tool in case of exceptional events. The constant and continuous controls allows providing quick and reliable information of the health state of the monitored structures.

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