

System Identification of Heritage Structures Through AVT and OMA: A Review

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Abstract: In this review article, the past investigations carried out on heritage structures using Ambient Vibration Test (AVT) and Operational Modal Analysis (OMA) for system identification (determination of dynamic properties like frequency, mode shape and damping ratios) and associated applications are summarized. A total of 68 major research studies on heritage structures around the world that are available in literature are surveyed for this purpose. At first, field investigations carried out on heritage structures prior to conducting AVT are explained in detail. Next, specifications of accelerometers, location of accelerometers and optimization of accelerometer networks have been elaborated with respect to the geometry of the heritage structures. In addition to this, ambient vibration loads and data acquisition procedures are also discussed. Further, the state of art of performing OMA techniques for heritage structures is explained briefly. Furthermore, various applications of system identification for heritage structures are documented. Finally, conclusions are made towards errorless system identification of heritage structures through AVT and OMA.

Keywords: Ambient Vibration Test (AVT), heritage structures, Operational Modal Analysis (OMA), structural assessment, system identification.

1 Introduction

The heritage structures reflect social, religious and cultural significance of human evolution. They are symbolic representation of the ancient engineering and architectural heritage. These structures contribute towards culture, tourism and economy of a country. Due to natural hazards like earthquake, tsunami, cyclones, storms etc., these structures are continuously getting damages and are prone to collapse. Hence, monitoring and preservation of these structures become necessary.

Extensive and rapid developments in electronic hardware and computer software field made Ambient Vibration Test (AVT) and Operational Modal Analysis (OMA) as the first choice of the researchers for learning the system identification of the heritage structures [Brownjohn (2003)].

Further, system identification of heritage structures have become valuable in structural analysis, health monitoring, damage identification and retrofitting assessment [Garaygordobil (2004); Cunha and Caetano (2005); Colapeitro, Fiore, De Fino et al. (2015)].

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Over past decades, the research community has made several attempts and studies on the abovementioned subject. Motivated from the above reason, this paper is a comprehensive review of several important research investigations conducted by the researchers for the system identification and associated applications on the heritage structures around the world using AVT and OMA.

At first, the paper briefs about the primary field investigations conducted on heritage structures. Next, the state of art of performing AVT on heritage structures is elaborated in length including the specifications and locations of accelerometers, source of ambient vibration loads and data acquisition procedures. Further, it discusses different OMA techniques for system identification of heritage structures along with its key features. Lastly, the applications of system identification features of heritage structures for finite element model updation, seismic assessment, health monitoring, repair and retrofitting are presented.

2 Primary field investigation on heritage structures

It is learned from the literature that primary field investigations prior to AVT on heritage structures play an important role in system identification. During these investigations, researchers tried to gain maximum knowledge about parameters such as history, geometry, typology of the structure, construction materials used in the structure, cracks and flaws present in the structure. Upon understanding the above said parameters, selection of accelerometers, operational excitation techniques and arrangement of the accelerometer networks were accomplished for performing the AVT.

2.1 Historical investigations

In general, researchers had conducted the historical investigations which are not compulsory for performing AVT and OMA. However, historical investigations contributed towards successful system identification of heritage structures and related applications like numerical modelling. During historical investigations, understandings were made towards parameters such as the year of construction, architecture followed and operational period [Peña, Lourenço, Mendes et al. (2010)]. Information on past events including natural disasters, strengthening of structural elements, restoration programs of collapsed structures were also documented [Rinaldis, Celebi, Buffarini et al. (2004)]. The historical investigations reduced probable errors while determining system identification after AVT and OMA (as the construction and operational period causes of damages and restoration programs of the structure were clearly understood [Rainieri, Fabbrocino, Cosenza et al. (2007); Casarine and Modena (2008); Ramos, Aguilar, Lourenço et al. (2013); Diaferio, Foti and Giannoccaro (2015)]).

2.2 Geometric investigations

Usually, geometric investigations were carried out to measure the dimensions of the structure [Krstevska, Tashkov and Shendova (2012)]. These investigations included measurement of plan of the structure, dimension of the structural elements and non-structural elements [Benedettini and Gentile (2007)]. Geometric investigations also highlighted complex dimensions of the structural and non-structural elements and aided

the researchers in planning and placing the accelerometer network for AVT [Buffarini, Clemente, Paciello et al. (2008); Cimellaro, Piantà and De Stefino (2012)].

2.3 Typology investigations

AVT, OMA results of heritage structures have to be verified against the results obtained from mathematical models and modal analysis [De Matteis and Mazzolani (2010)]. In this concern typology investigations play an important role. Generally, typology investigations were carried out by visual inspection, *in-situ* and laboratory tests to understand the construction techniques, strength of construction materials, and material degradation [Schmidt (2009); Lacanna, Ripepe, Marchetti et al. (2016); Le and Argoul (2016)]. The understandings made during typological investigations were used towards accurate mathematical modelling, modal analysis and model updation process [Ramos, Casarin, Algeri et al. (2006); De Matteis and Mazzolani (2010); Serhatoğlu, Livaoglu and Bağbanci (2015)]. Typological survey showed that the dynamic behaviour of structure and its components are being affected due to material degradation over the time [Ivorra and Pallars (2005); Asteris, Chronopoulos, Chrysostomou et al. (2014)].

2.4 Damage investigations

Damage investigations were carried to study the present status of visible damages such as major and minor cracks, settlements of foundation, displacement and continuity of structural elements and faulty joints [Gallino, Gentile and Saisi (2009); Colapietro, Fiore, De Fino et al. (2015)]. The outcomes of these investigations revealed the present condition of the structures in term of cracks, voids, critical element and faulty joints [Gentile, Saisi and Cobboi (2015)]. Gentile et al. [Gentile and Saisi (2007)] noticed large vertical cracks on walls of Bell tower of Monza's Cathedral by visual survey. Few non-destructive tests were also performed for detecting voids and internal cracks in structure as a part of primary survey by Casciati and Al-Saleh [Casciati and Al-Saleh (2010)]. Colapietro et al. [Colapietro, Fiore, De Fino et al. (2015)] performed radar testing and sonic test for detecting internal voids and cavities in the walls of Bell tower of "Santa Maria di San Luca" in Valenzano. More details are available from Gallino et al. [Gallino, Gentile and Saisi (2009)], Asteris et al. [Asteris, Chronopoulos, Chrysostomou et al. (2014)], Gentile et al. [Gentile and Saisi (2014)] for damage identification in heritage structures.

3 Ambient vibration testing

Ambient vibration testing is a field procedure to record the vibrations of structures, excited by ambient activities like traffic, wind etc. Generally, sensors, data acquisition system, connecting cables are being used for conducting AVT. In AVT, the excitation signal will be the response signal measured on the reference point. The response signal on roving points will be taken as output signal. The easiness in transportation and installation of the sensors and other electronic instruments made AVT very robust for recording vibration response of heritage structures.

The sensing device plays major role in recording the response of the structures through AVT. Different sensors can be utilized for measurement of different parameters such as acceleration, displacement, strain, temperature, pressure and wind speed. For recording

the response of the heritage structures generally used sensors are seismometers [Pradhan, Singhal, Mondal et al. (2012); Lacanna, Ripepe, Marchetti et al. (2016); Krstevska, Tashkov and Shendova (2012);], Velocity meters [Rinaldis, Celebi, Buffarini et al. (2004); Cimellaro, Piantà, and De Stefano (2012)] Accelerometers [Taleb, Bouriche, Remas et al. (2012); Foti, Diaferio, Giannoccaro et al. (2012)]. Though, the aim of AVT is to get the dynamic parameters of the structure, and acceleration parameter is most suited representative form for dynamic response of the structure [Omenzetter (2013); Casarine and Modena (2008)]. The accelerometers proved to be better sensing devices for AVT in the case of heritage structures [Bergamasco, Bongiovanni, Carpani et al. (2014); Cunha and Caetano (2005); Brincker and Ventura (2015)]. Further, following are advantages of accelerometers over other sensors suggested by researchers Cunha et al. [Cunha and Caetano (2005); Brincker and Ventura (2015)]:

- The size of the accelerometer is small and can be installed easily.
- The accelerometers are low cost and highly sensitive.
- The accelerometer does not require any power source.
- The accelerometer can operate on wide frequency spectrum.

Furthermore, Brownjohn [Brownjohn (2003)] stated that the accelerometers can able to record acceleration of 10^{-5} m/sec² (1 μ g) and can sense displacement of 10^{-7} m at frequency range 1 Hz.

3.1 Accelerometers

Accelerometers are the sensors which converts acceleration into signal variable output like current, voltage etc. Specific characteristics of accelerometers such as range (difference between maximum measured response to minimum measured response), Resolution (Smallest value of increment recorded by response), type, sensing frequency (maximum value of frequency of response that is measured), working environment, and size (Physical dimensions of the accelerometers), play a vital role in acquiring accurate response of the heritage structure during AVT [Taleb, Bouriche, Remas et al. (2012)]. The significant characteristics of accelerometer have to be finalised before conducting an AVT [Prawin, Rao and Lakshmi (2015)]. Further, the selected accelerometer should be able to detect frequency as low as 1-2 Hz and sensitivity of 1v/g or higher is desirable for AVT on heritage structures [Atmaturkur, Pavic, Reynolds et al. (2009)]. There are many type of the accelerometers available in the market, from which piezoelectric type (for example Wilcoxon WR731A, BK Type 4506, PCB 393B12 etc.) and force balance type (for example EpiSensor FBA ES-T SA-107LNC etc.) accelerometers were commonly used for performing AVT on the heritage structures [Turek, Ventura and Placencia (2002); Bayraktar, Birinci, Altunişik et al. (2009); Foti, Diaferio, Giannoccaro et al. (2012); Min, Kim, Park et al. (2013); Gentile, Saisi and Cobboi (2015)].

3.2 Locations of accelerometers

Number of accelerometers and their locations are extremely significant as they have to record the response of the heritage structure wholly. While performing AVT, if the accelerometer networks are not covering the whole geometry, then the results will be unreliable and will not illustrate the exact response of the structure under ambient vibrations [Ataturkur and Sevim (2011)]. Effective accelerometer locations can be selected based on, (1) engineering judgement, (2) past experience, (3) initial numerical models and modal analysis, (4) optimization algorithms and (5) combination of above said all parameters. El Borgi et al. [El Borgi, Smaoui, Casciati et al. (2005)] used an initial finite element model for judicious placement of the accelerometers on a Palace structure and found that a total of 70 critical accelerometer locations were required for acquiring full response of the structure. Studies conducted by Ataturkur et al. [Ataturkur, Pavic, Reynolds et al. (2009)] on several Gothic Churches and concluded that the finite element modelling and modal analysis helps in the placement of accelerometer than engineering experiences and optimization algorithm. Pachón et al. [Pachón, Compán, Fdo et al. (2014)] and Compan et al. [Compan, Pachón, and Cámara (2017)] suggested that the response of the largest modal displacement determined by modal analysis on the initial finite element model must be taken into consideration for planning locations of reference accelerometers. Conde et al. [Conde, Ramos, Oliveira et al. (2017)] followed recommendations provided in previous research works on heritage bridges and used numerical models for placement of accelerometers. Prabhu and Ataturkur [Prabhu and Ataturkur (2012)] developed an algorithm for effective accelerometer locations on heritage structure. In their study, Effective Independence Method (EIM) [Kammer, (1991)] was modified to determine the optimized location of accelerometers on heritage structures and named as Modified Effective Independence Method (MEIM). A total of 781 candidate accelerometer locations were selected on Gothic Revival-style masonry cathedral initially and after applying MEIM, the optimum number of location reduced to 120 accelerometer locations at a loss of 10% initial dynamic information. More algorithms on optimization of location of accelerometers can be found in Rao et al. [Rao, Lakshmi and Krishnakumar (2014)] and Yang et al. [Yang and Lu (2017)].

Some of the studies showed that the accelerometers cannot be placed on the desired locations due to various constraints including complex geometry and inaccessibility. In this concern, Tab. 1 summarises few practical problems and solutions achieved by researchers regarding placement of accelerometers.

Table 1: Practical problems and solutions on placement of accelerometers

Sn.	Reference	Problems on placing accelerometers	Solution for placement of accelerometers
1.	Turek, Ventura and Placencia (2002)	Researchers were not allowed to enter in some part of the tower, because of on-going retrofitting work.	Researchers put the accelerometers in nearby locations of prohibited areas.
2.	Atamturktur, Pavic, Reynolds et al. (2009)	Authors found that curved surfaces were difficult for placing the accelerometers in desired horizontal direction.	Authors used steel screw plates for placing the accelerometers in exact direction.
3.	Foti, Diaferio, Giannoccaro et al. (2012)	The study dealt with an area occupied by staircase situated at left side of the tower.	The authors placed accelerometers only on the side where staircase was absent at the right side of the tower.
4.	Gentile and Saisi (2014)	The authors studied that the cross section of every floor was different in the tower.	The researchers placed the accelerometers at the point where cross sections were changing.
5.	Diaferio, Foti and Giannoccaro (2015)	Authors found that it was too difficult to place the accelerometers in exact orthogonal position in tower due to irregular geometry of the floors.	Authors designed proper rectangular blocks to place the accelerometers orthogonally.
6.	Gentile, Saisi and Cobboi (2015)	Authors faced difficulties in placing the accelerometers inside the Church structure due to summer season.	Authors placed accelerometers only on outer side of the walls and performed the test.
7.	Ceravolo, Pistone, Fragonara et al. (2016)	The study dealt with some areas with no proper accessibility in the building for placement of accelerometers.	The authors used optimization algorithms to choose other appropriate locations which can cover the whole structure.

Location of the accelerometers is crucial parameter in AVT, which has to be achieved incorporating practical solutions to the field problems. Tab. 2 highlights the geometry of test structure and the details of accelerometer locations used in past investigations.

Table 2: Details of number and location of accelerometers used in past investigations

S No.	Details collected from literature
<p>1. Diagram showing structural geometry and accelerometer location</p>	
<p>Description of accelerometer locations</p>	<p>Figure 1: Elevation of tower with accelerometer locations, [Gentile and Saisi (2007)] → 1st stage → 2nd stage</p> <p>The authors selected a total of 20 locations for placement of accelerometers. To cover the whole structure, they performed AVT in two deployment of accelerometers. In the first deployment, thirteen locations (shown in Fig. 1 with green colour) were used. In the second deployment, seven locations (shown in Fig. 1 with red colour) were used. The placement of accelerometers covered both horizontal orthogonal directions. The accelerometers which are placed on the right top corner (shown Fig. 1 as number 7, 12) were acted as reference acclerometers.</p>

2. Diagram showing structural geometry and accelerometer locations

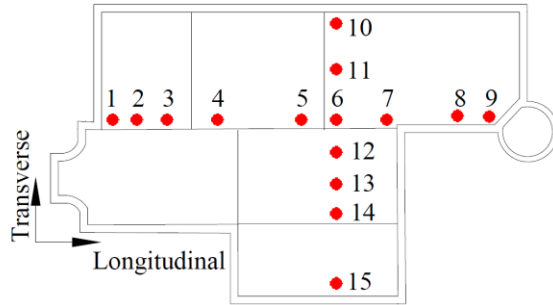


Figure 2: Third floor plan of building with accelerometer locations [Taleb, Bouriche, Remas et al. (2012)]

Description of accelerometer locations

For the objectives the authors have selected fifteen locations (shown in Fig. 2 with red colour) for the placement of seismometers on the third floor. They placed nine seismometers in longitudinal direction and six seismometers in lateral direction. The entire study was conducted in a single deployment.

3. Diagram showing structural geometry and accelerometer location

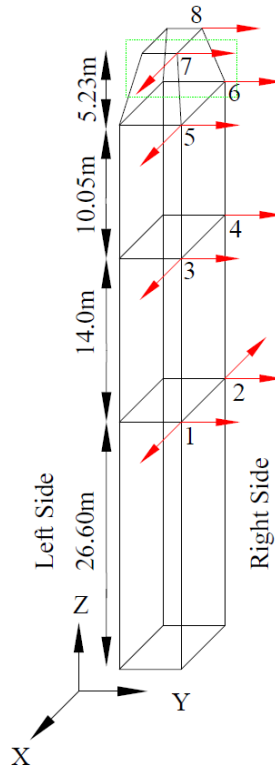


Figure 3: Elevation of tower with accelerometer locations, [Foti, Diaferio, Giannoccaro et al. (2012)]

Description of accelerometer locations

The authors selected a total of eight locations (shown in Fig. 3) for placement of accelerometers. Further, thirteen uniaxial accelerometers were employed in horizontal orthogonal directions. Among them, four accelerometers were placed at 5th floor. Further, 7th, 9th and 10th floors had three accelerometers each. The accelerometers at the 10th floor were used as reference accelerometers as shown in Fig. 3.

4. Diagram showing structural geometry and accelerometer location

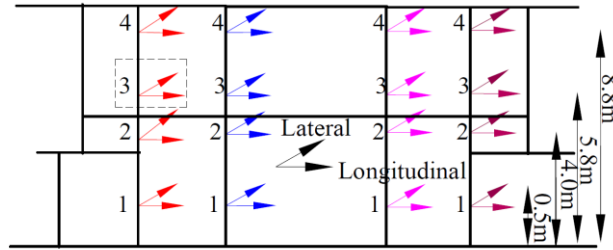


Figure 4: Elevation of Go-ju columns with location of accelerometers (1st to 4th deployment) [Min, Kim, Park et al. (2013)]

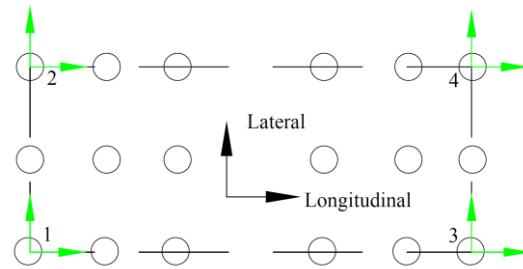


Figure 5: Plan of Goigo-ju column with location of accelerometers (5th and 6th deployment) [Min, Kim, Park et al. (2013)]

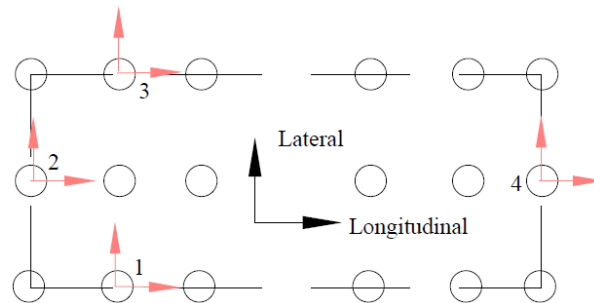


Figure 6: Plan of Pung-Ju column with location of accelerometers (7th deployment) [Min, Kim, Park et al. (2013)]

Description of accelerometer locations

The study was conducted choosing 48 different accelerometer locations. A total of ten accelerometers were used for AVT following redeployment of accelerometers for seven times to capture the vibration of whole structure. A pair of reference accelerometers were placed at 5.8 m height of left most Go-ju column (as highlighted inside the box, Fig. 4). At first, four pairs of accelerometers were deployed at the left most Go-Ju column at the elevations of 0.4 m, 4.0 m, 5.8 m, and 8.8 m and vibrations were recorded. Reinstallation and same test procedures were sequentially conducted for the remaining Go-Ju columns from the left to the right direction, (i.e. deployment 1st to 4th in Fig. 4). The same above said procedures were adopted for placing accelerometers at all Guigo-Ju columns twice at elevation of 5.8 m and 8.8 m respectively (i.e., 5th and 6th deployment in Fig. 5). Further the vibration measurement of Pung Ju columns were recorded placing the accelerometers at elevation of 8.8m (i.e., 7th deployment in Fig. 6).

5. Diagram showing structural geometry and accelerometer location

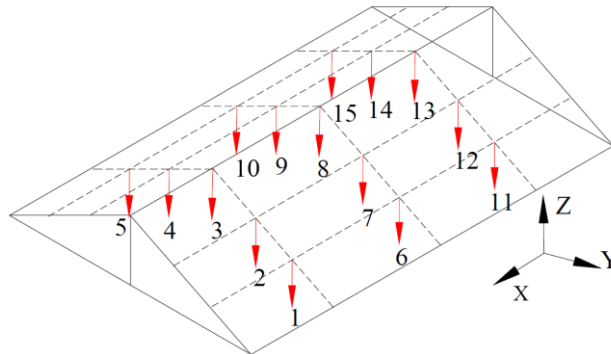


Figure 7: Elevation of Vault with accelerometer locations [Çalik, Bayraktar, Türker et al. (2014)]

Description of accelerometer locations

The authors chose fifteen locations to record the vibrations of the vault structure. They placed accelerometers in vertical direction on the vault structure in three rows having five accelerometers in each row (as shown in Fig. 7) in a single deployment.

6. **Diagram showing structural geometry and accelerometer location**

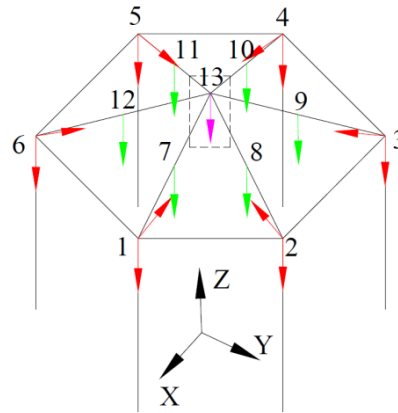


Figure 8: Elevation of Dome Structure with accelerometer locations [Uçak, Bayraktar, Türker et al. (2014)]

Description of accelerometer locations

Authors selected a total of nineteen accelerometers for conducting AVT. At first deployment, authors have placed eight accelerometers around the ring of the dome, in radial and in vertical direction (as shown in Fig. 8 with red colour). In the next deployment, they placed four accelerometers at mid level of the dome only in vertical direction. The reference accelerometer was deployed at the apex of dome structure (as highlighted in box).

7. **Diagram showing structural geometry and accelerometer location**

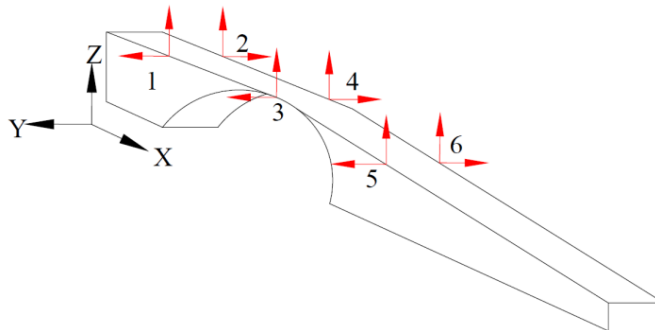


Figure 9: Location of accelerometers in Snayuva Bridge [Bayraktar, Birinci, Altunişik et al. (2009)]

Description of accelerometer locations

Authors selected a total of six locations for the placement of accelerometers. To identify the dynamic characteristic of whole structure they placed the accelerometers on the sides of the deck in vertical and horizontal directions(as shown in Fig. 9).

8. Diagram showing structural geometry and accelerometer location

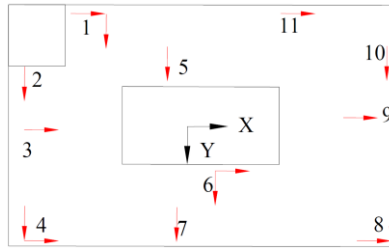


Figure 10: Location of Velocity sensors at second floor of Margherita Palace (1st deployment) [Cimellaro, Piantà and De Stefano (2012)]

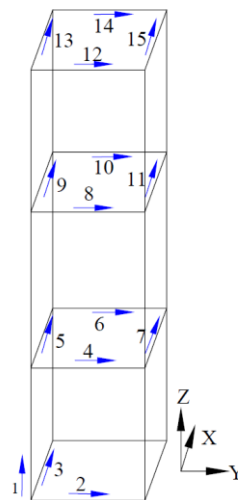


Figure 11: Location of Velocity sensors at the tower (2nd deployment) [Cimellaro, Piantà and De Stefano (2012)]

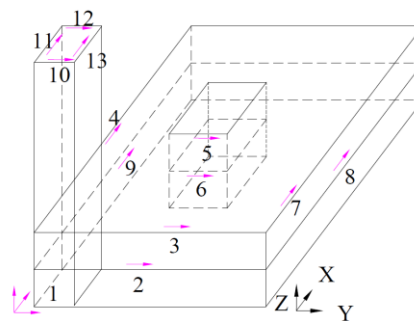


Figure 12: Location of Velocity sensors at the first and second floor of the palace and the tower (3rd deployment) [Cimellaro, Piantà and De Stefano (2012)]

Description of accelerometer locations

Authors selected a total of 38 locations for the placement of velocity sensors following three deployments to cover the whole structure. In first deployment, 15 velocity sensors were placed on the second floor of the palace (as shown in Fig. 10). In the second deployment, 15 velocity sensors were placed inside the civic tower structure. Among that three sensors were placed on the ground at the base of the tower, four inside the tower at the height of 10.5 m , four on the wooden slab located at height of 29.5 m, and four at the top of the tower i.e., 34.5 m. The last deployment includes partly Margherita Palace and partly the civic tower (Fig. 11). Three velocity sensors were located at the base of the tower and four at the top of the tower, while the remaining sensors were divided between the first floor and the second floor of the building (as shown in Fig. 12).

9. Diagram showing structural geometry and accelerometer location

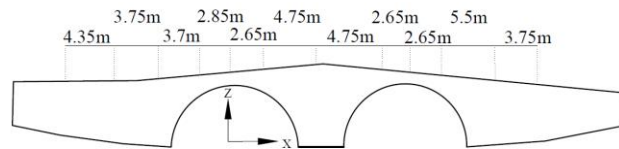


Figure 13: Elevation of the bridge with seismograph locations [Conde, Ramos, Oliveira et al. (2017)]

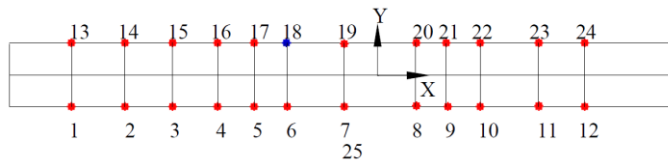


Figure 14: Top view of the bridge with 1st to 12th seismograph deployments [Conde, Ramos, Oliveira et al. (2017)]

Description of accelerometer locations

The authors selected a total of 24 locations for placement of the seismographs (as shown in Figs. 13-14) covering the whole structure. The study was performed using three seismographs following 12 redeployment. The deployment of the seismographs were as follows:

1st deployment-18, 7,3

2nd deployment-18, 8, 14

3rd deployment-18, 9, 15

4th deployment-18, 10, 16

5th deployment-18, 11, 17

6th deployment-18, 12, 6

7th deployment-18, 24, 5

8th deployment-18, 23, 4

9th deployment-18, 22, 3

10th deployment-18, 21, 2

11th deployment-18, 20,1

12th deployment-18, 19, 25

Seismograph at location 18 (as shown in Fig. 14 with blue colour) was the reference sensor.

10. Diagram showing structural geometry and accelerometer location

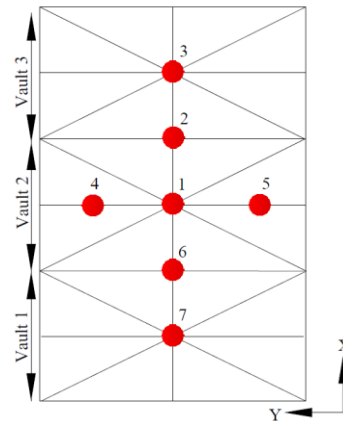


Figure 15: Accelerometer locations on the top of choir fan vault of Washington National Cathedral [Atamturkur, Fanning and Boothby et al. (2007)]

Description of accelerometer locations

Authors selected a total of seven locations for the placement of accelerometers. The accelerometers were placed in vertical direction on the three adjacent vaults (as shown in Fig. 15).

11. Diagram showing structural geometry and accelerometer location

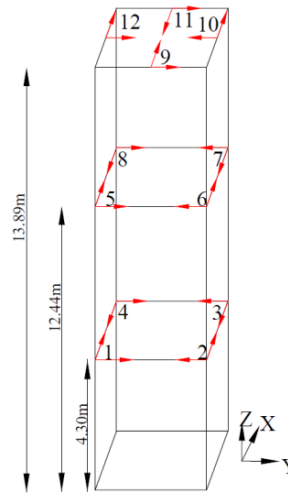


Figure 16: Elevation of tower of Annunziata with accelerometer locations [Diaferio, Foti and Giannoccaro (2015)]

Description of accelerometer locations

Authors selected a total of 12 locations for the placement of accelerometers. Eight accelerometers were placed in four corners of first floor (at height of 4.30 m), eight accelerometers were placed on second floor (at height of 12.44 m) and eight accelerometers were placed at top floor (at height of 13.89 m) in both orthogonal horizontal directions (as shown in Fig. 16).

3.3 Ambient vibration loads on the historical structures

In general, dynamic loads are the basis of vibration testing while assessing the constructed structures. In case of AVT, wind loads, traffic induced loads, wave induced loads, and other environmental loads are generally being used [Omenzetter, Beshkroun, Shabbir et al. (2013)]. The source of vibration and intensity of the vibration, have major effects for measurement of the dynamic properties of the structures [Wilson, Oyarzo-Vera, Omenzetter et al. (2008)]. Since AVT does not account the input of the external vibration, multiple input vibrations can be used for excitation of the structure at the time of AVT [Cantieni (2009)]. Tab. 3 discusses the excitation sources used by researchers on various heritage structures in their studies.

Table 3: Summary of dynamic excitation used in previous studies

Sn.	Reference	Details of excitation used
1.	Osmancikli, Uçat, Tarum et al. (2012); Ramos, Aguilar, Lourenço et al. (2013); Krstevska, Tashkov and Shendova (2012)	Many times it happened that the environmental loads were not enough to trigger all the target modes of the structure. The authors mentioned about the studies used other techniques like impact hammer, external micro-tremors devices to trigger the targeted modes of the structure. The studies also account that the excitation should be given at multiple point of the structure for testing.
2.	Gentile, Saisi and Cabboi (2012)	The authors studied the response of the heritage bell tower structure twice by conducting AVT and OMA. In first attempt they used micro tremors and ambient vibrations as excitation sources. In the second attempt they used swinging of bells as source of excitation. Upon analysis, they found that there is slight difference in higher modes. The investigation also concluded that there is slightly increase in frequency due to temperature variation.
3.	Votsis, Kyriades, Chrysostomou et al. (2012); Bayraktar, Altunişik, Sevim et al. (2011); Uçak, Bayraktar, Türker et al. (2014); Ubertini, Comanducci and Cavalagli (2016); Serhatoglu, Livaoğlu and Bağbanci (2015); Min, Kim, Park et al. (2013); Magalhães, Cunha, Caetano et al. (2010); Jaishi, Ren, Zong et al. (2003)	The mentioned studies considered the vibration from human activities, traffic, wind and operational vibrations for exciting the test structure. Authors had considered multiple input excitations because of the fact that the size of the structure was very huge, and single input excitation was not enough to trigger all the necessary modes of the structure.
4.	Ceroni, Sica, Pecce et al. (2014)	The authors used a ‘free-fall’ of concrete block from a truck to induce external vibration on the structure. This impulsive source of vibration should be used in controlled manner so that no damage happens to heritage structure.
5.	Russo (2016)	The author suggested ignoring the excitation of structure from extraordinary sources like fireworks, earthquake or natural calamity. Because of the reason that, the response of the structure will not represent its original service conditions.

3.4 Data acquisition

Data acquisition is another important aspect for conducting the AVT accurately. The data acquisition is a procedure of collecting data from the sensing device and stores it to computer system. For recording errorless response of the structure through AVT, following parameters should be selected properly:

3.4.1 Sampling rate

Generally, sampling rate is defined as upper limit of frequency band that can be utilized for analysis of recorded signals. Sampling rate should be selected for testing the structure for each measurement [Brinckner and Ventura (2015)]. In general, sampling rate is the number of samples recorded in unit time and measured in terms of Hertz (Hz) or sample per second (sps). Sampling rate directly affects the quality of data being recorded [Gallino, Gentile and Saisi (2009)]. Generally, there are two methods which are being used for selection for sampling rate [Brinckner and Ventura (2015)]: (a) Maximum significant frequency of the structure (b) Standard frequency for data acquisition system having Nyquist frequency of 50-100Hz for large structures.

3.4.2 Time duration

A proper time duration for recording the data for measurement is another parameter required during excitation of the structure [Brincker and Ventura (2015)]. The measured time duration should be long enough to ensure that all the targeted modes are sufficiently excited [Turek, Ventura and Placencia (2002)]. The time period should be chosen based on Brincker Criterion [Brincker, Ventura and Andersen (2003)] which is been followed in many AVTs conducted on heritage structures [Gentile and Saisi (2007); Ramos, Marques, Lourenço et al. (2010)]. According to Brincker criterion, if the fundamental frequency of any structure is f measured in Hertz, its modal damping factor is ζ , and the recording duration T should be at least $T = \frac{100}{\zeta f}$.

4 Operational Modal Analysis (OMA)

OMA is a common numerical procedure which is used for the processing of collected data acquired from the AVT (having characteristics of white noise) which covers the entire frequency range of modal characteristics of the whole structure [Ewin (1986)] to identify the dynamic characteristics of system. The OMA procedure can be performed in two platforms, i.e., time domain and frequency domain [Brincker, Zhang and Andersen (2000)]. OMA procedure has many techniques, which are used by researchers according to applications and their reliability. Some of the popular techniques in time domain are Random Decrement Technique (RD), Unweight Principal Component technique (UPC), Principal Component technique (PC) and Stochastic Subspace Identification technique (SSI). Further, some of the popular techniques in frequency domain are Peak Picking Method (PP), Frequency Domain Decomposition (FDD) and Enhance Frequency Domain Decomposition Method (EFDD).

The general outcomes of OMA are the dynamic parameters of the system such as frequencies, mode shapes and damping ratios. Over the period of decades, the OMA has

proven to be an effective method in dynamic system identification of many complex structures and mechanical systems. Hence, the investigations deal with heritage structures have also adopted OMA procedure and evaluated the complex dynamic parameters in their research studies using abovementioned OMA techniques. Tab. 4 presents the OMA techniques used by research investigations on heritage structures. The following section discusses more details about SSI and FDD techniques on heritage structures.

Table 4: OMA techniques used by research investigations on heritage structures

Sn.	Reference	Reference Structure	Modal Identification Method Used	
			Method-1	Method-2
1.	Jaishi, Ren, Zong et al. (2003)	Masonry temple, Nepal	PP	SSI
2.	Ramos, Casarin, Algeri et al. (2006)	Qutub Minar, India	UPC	PC
3.	Gentile and Saisi (2007)	The bell-tower of the Monza's Cathedral, Italy	PP	SSI
4.	Foti, Diaferio, Giannoccaro et al. (2012)	Provincial Administration Building, Italy	SSI	EFDD
5.	Bayraktar, Birinci, Altunişik et al. (2009)	Senyuva Historical Arch Bridge, Turkey	PP	SSI
6.	Gentile, Saisi and Cabboi (2012)	Historic masonry bell-tower of Milan, Italy	FDD	SSI
7.	Bayraktar, Altunişik, Sevim et al. (2011)	The Iskenderpas, Turkey	PP	SSI
8.	Osmancikli, Uçat, Tarum et al. (2012)	Hagia Sophia Bell Tower, Turkey	EFDD	SSI
9.	Cimellaro, Piantà, and De Stefano (2012)	The ancient masonry city hall in L'Aquila, Italy	FDD	RD
10.	Ramos, Aguilar, Lourenço et al. (2013)	The Saint Torcato church, Italy	FDD	EFDD
11.	Min, Kim, Park et al. (2013)	The Dongdaemun, Korea.	SSI	FDD
12.	Pachón, Compán, Fdo et al. (2015)	Chapel of the Würzburg Residence, Germany	FDD	SSI

4.1 Stochastic Subspace Identification technique (SSI)

The stochastic subspace identification technique [Van Overschee and Moor (1996)] is a time-domain method that directly works with time data without the need to convert them to correlations or spectra. The stochastic subspace identification algorithm identifies the state space matrices based on the measurements by using robust numerical techniques. The method may be introduced as follows, dynamic behaviour of any mechanical system of n_2 masses, which is assembled with springs and dampers can be formulated as.

$$M\ddot{U}(t) + D\dot{U}(t) + KU(t) = F(t) = B_2 u(t) \quad (1)$$

Where M-mass, D-damping, K-stiffness and F(t)-external dynamic force. F(t) is factorised into B_2 describing the inputs in spaces and vector $u(t)$ describing m input time. But for the above equation, system identification is not possible, because of the reason that the equation is in continues form but the measurement response is in discrete time samples and it is also not needed to determine all the Degree of Freedom (DOF). Van Overschee et al. [Van Overschee and Moor (1996)] linked several mathematical fields like control theory, statistics, optimization theory to form a stochastic subspace state model for Eq. (1) which can expresseed as follows,

$$\dot{x}(t) = A_c x(t) + B u(t) \quad (2)$$

Where, $\dot{x}(t)$ -continuous time state space, A_c is system matrix in continues time which depends on the mass, stiffness and damping properties of the structure, B is a input matrix and $x(t)$ is state vector. In practical application, all the DOF cannot be monitored. Let's assume, the measurement are evaluated at one accelerometer location, the observation equation will be,

$$y(t) = C x(t) \quad (3)$$

where, $y(t)$ -output vector and C is output matrix for the measurement.

Eqs. (2) and (3) represents the continuous time state space model. But the recorded measurements are in the form of discrete sampling of time instant $t_k = k \Delta t$. The solution of the above Eqs. (2) and (3) for output only vibration in discrete time is

$$x_{k+1} = Ax_k + w_k \quad (4)$$

$$y_k = Cx_k + v_k \quad (5)$$

Where, x_k -discrete time state vector which contains the displacement and velocities describing the state of system at time instant and y_k is a output vector of the state space. A-discrete state matrix and C-discrete output matrix which maps the state vector into the measured output. w_k -processed noise due to disturbance and inaccuracy in sensors, v_k -measured noise due to inaccuracy. It can be shown that the dynamic model parameters (frequency, mode shape, damping) of a structure under white noise excitation can be identified by measured output response y_k .

In case of AVT of the large structures to find the dynamic parameters, all the output can be measured at once, but they are divided into different setups. Assume for N elements of output, the measured response of the structure in form of acceleration are assembled in a data set Y

$$Y = [Y_1 \quad \dots \quad Y_N] \quad (6)$$

N – Discrete time sampling number of responses

At first, the data set Y is converted into Hankel matrix for better arrangement of measured response and expressed as

$$H = \frac{1}{j} \begin{bmatrix} y_0^{ref} & \cdots & y_{j-1}^{ref} \\ \vdots & \ddots & \vdots \\ y_{2i-1} & \cdots & y_{2i+j-2} \end{bmatrix} = \begin{bmatrix} Y_p^{ref} \\ Y_f \end{bmatrix} \quad (7)$$

Where, Y_p^{ref} and Y_f are the past and future Hankel Matrix, respectively of data set Y .

Next, data handle technique like orthogonal projection is applied to Hankel Matrix towards eliminating the unnecessary samples to avoid rigorous computation. Upon applying abovementioned technique the Projection matrix (O) for Hankel Matrix can be defined as the conditional mean of Y_f with respect to Y_p^{ref} .

$$O = E \left(\frac{Y_f}{Y_p^{ref}} \right) = Y_f Y_p^{ref} (Y_p^{ref} Y_p^{ref T})^{-1} Y_p^{ref} \quad (8)$$

Further, projection matrix (O) can be factorised into observability matrix (Γ) and Kalman filter state (X)

$$O = \Gamma X \quad (9)$$

In order to calculate observability matrix (Γ), the singular value decomposition (SVD) has been applied to projection matrix (O) as follows,

$$WOI = USV^T = [U_1 \quad U_2] \begin{bmatrix} S_1 & 0 \\ 0 & S_2 \end{bmatrix} \begin{bmatrix} V_1^T \\ V_2^T \end{bmatrix} \quad (10)$$

U -Left singular vector, S -Singular scalar value, V -Right singular vector

Where, $W = (Y_f Y_f^T)^{0.5}$ is a pre-multiplied weighing matrix, and I is Identity matrix. Now, the observability matrix (Γ) can be calculated as

$$\Gamma = W^{-1} U_1 S_1^{0.5} \quad (11)$$

Finally, from observability matrix (Γ), A and C matrices can be calculated as follows for i blocks of the Hankel Matrix (H) [Eq. (7)]:

$$\Gamma = \begin{bmatrix} C \\ CA \\ CA^2 \\ \vdots \\ CA^{i-1} \end{bmatrix} \quad (12)$$

At last, eigen value decomposition of A will give the eigen values and eigen vectors of the system as follows

$$A = \Psi \Lambda \Psi^{-1} \quad (13)$$

$$\Phi = C \Lambda \quad (14)$$

Here, Ψ -Eigen Matrix contains eigen vectors as column, Λ -Diagonal Eigen Matrix, Φ -mode shape vector. The mode shape of the previously mentioned accelerometer location, defined as the column vector Φ are observed parts of system eigenvectors Ψ and are thus obtained using Eq. (3). In general, the obtained dynamic parameters are compared against

model orders in form of stabilization diagram for best understanding. Stabilization diagram accommodates dynamic properties such as, frequency, damping ratio, mode shape in the horizontal axis denoted as $x(n,m)$. The vertical axis accommodates model order denoted as $y(n,m)$. Here, n is a value on vertical axis and m is a considered mode. A modal parameter is identified and considered as stable if it agrees the following stabilization criteria

$$|y(n-1, m) - y(n, m)| < \Delta x \quad (15)$$

Where, Δx is the threshold value provided for the operation. In stabilization diagram, the modes which are present on vertical lines will be a physical mode. Further, Tab. 5 summarises the key features of SSI technique highlighted in previous studies.

Table 5: Key features of SSI Techniques adopted in previous studies

Sn.	Reference	Remarks
1.	Ramos (2006)	Authors found that SSI technique is very efficient in finding closely spaced frequency of structural modes of the axisymmetric structure.
2.	Chiorino, Ceravolo, Spadafor et al. (2011)	Authors found that SSI technique has ability to distinguish between real modes of the structure and the modes that occurred temporarily. Further, the SSI technique was found to be more accurate for the linear dynamic characterisation of the tested monument.
3.	Foti, Diaferio, Giannoccaro et al. (2012)	The study concluded that SSI technique is very useful in finding out mode shapes and associated frequency of symmetrical structures.
4.	Diaferio, Foti, Giannoccaro et al. (2014)	The author concluded that SSI method did not highlight the peaks corresponding to frequency of structure due to very squat and fixed shape of the test structure.
5.	Kocaturk, Erdogan, Demir et al. (2017)	The study concluded that the SSI technique was very helpful in determining the higher modes of the cylindrical shaped building.

4.2 Frequency Domain Decomposition technique (FDD)

The frequency domain decomposition technique [Brincker, Zhang and Andersen (2000)] is a frequency domain method which is based on singular value decomposition (SVD) of the spectral density (SD) obtained from ambient vibration test. The technique is closely related to the complex modal indicator function (CMIF) [Shih, Tsuei, Allemang et al. (1988)] which was based on an SVD of the frequency response function (FRF) matrix and presentation of the singular values as function of frequency. It concentrates all information in one single graph with singular values of the SD matrix. Tab. 6 summarises the features of FDD technique highlighted in previous studies. The theoretical background of FDD method [Brincker, Zhang and Andersen (2000)] for OMA is as follows:

At first, the relationship between unknown input $x(t)$ and response $y(t)$ has to be expressed in form of power spectral density function (PSD)

$$G_{yy}(j\omega) = \bar{H}(j\omega) G_{xx}(j\omega) H^T(j\omega) \quad (16)$$

Where, $G_{yy}(j\omega)$ -response power spectral density function, $G_{xx}(j\omega)$ -input spectral density function, $H(j\omega)$ -Frequency Response Function (FRF) Matrix, T-Complex conjugate or transpose

Next, FRF matrix $H(j\omega)$ has to be reduced into pole/residue form as follows,

$$H(j\omega) = \sum_{k=1}^n \frac{R_k}{j\omega - \lambda_k} + \frac{\bar{R}_k}{j\omega - \bar{\lambda}_k} \quad (17)$$

Here, R_k -Residue, λ_k -Poles. The unknown input matrix has to be assumed as in form of white noise, then the input PSD matrix will become a constant matrix $G_{xx}(j\omega)=C$, further, the Eq. (16) can be written as

$$G_{yy}(j\omega) = \sum_{k=1}^n \sum_{s=1}^n \left[\frac{R_k}{j\omega - \lambda_k} + \frac{\bar{R}_k}{j\omega - \bar{\lambda}_k} \right] C \left[\frac{R_s}{j\omega - \lambda_s} + \frac{\bar{R}_s}{j\omega - \bar{\lambda}_s} \right]^H \quad (18)$$

Where H is complex conjugate transpose. In order to simplify Eq. (18) partial fraction theorem and mathematical manipulations has been performed as follows

$$G_{yy}(j\omega) = \sum_{k=1}^n \frac{A_k}{j\omega - \lambda_k} + \frac{\bar{A}_k}{j\omega - \bar{\lambda}_k} + \frac{B_k}{-j\omega - \lambda_k} + \frac{\bar{B}_k}{-j\omega - \bar{\lambda}_k} \quad (19)$$

Where A_k is the k^{th} residue matrix of PSD and contribution to residue from k^{th} mode can be given as

$$A_k = \frac{R_k C \bar{R}_k^t}{-\alpha_k} \quad (20)$$

Here, α_k -negative part of the real poles. In case of lightly damped structure or very low damping, the residue will become proportional to the mode shape vector which is shown below

$$A_k = d_k \phi_k \phi_k^T \propto R_k C \bar{R}_k^t \quad (21)$$

Further, at ω frequency, limited modes will be contributed significantly. Let the set of mode be $\text{Sub}(\omega)$, thus the response density function for PSD becomes

$$G_{yy}(j\omega) = \sum_{k \in \text{Sub}(\omega)} \frac{d_k \phi_k \phi_k^T}{j\omega - \lambda_k} + \frac{\bar{d}_k \bar{\phi}_k \bar{\phi}_k^T}{j\omega - \bar{\lambda}_k} \quad (22)$$

Furthermore, the estimated PSD $G_{yy}(j\omega)$ is then decomposed into singular value by singular value decomposition

$$G_{yy}(j\omega) = U_i S_i U_i^T \quad (23)$$

Where, $U_i = [u_{i,1} \ u_{i,2} \ \dots \ u_{i,m}]$ is unitary matrix and S_i is diagonal matrix contains scalar values of k^{th} mode. If k^{th} mode is dominating then there will be only one term in Eq. (22). Thus the estimated mode shape will be:

$$\hat{\phi} = u_{i,1} \quad (24)$$

Finally, the mode shapes are estimated from the singular vectors of the SVD of the spectral density matrix. However, we have an SVD for each frequency where the Singular Diagonal matrix is known because the SVD can be carried out for all known

frequencies. This also means that if we have the same number of modes as we have sensors, then in principle all mode shapes can be found at one single frequency line. In the spectral density plot of PSD vs frequency, PSD function is identified around the peak is estimate of the mode shape $\hat{\phi}$ with the singular vectors for the frequency lines around the same peak. From the Single Degree of Freedom (SDOF) density function obtained around the peaks of PSD function, natural frequency and damping can be estimated for respective mode shapes.

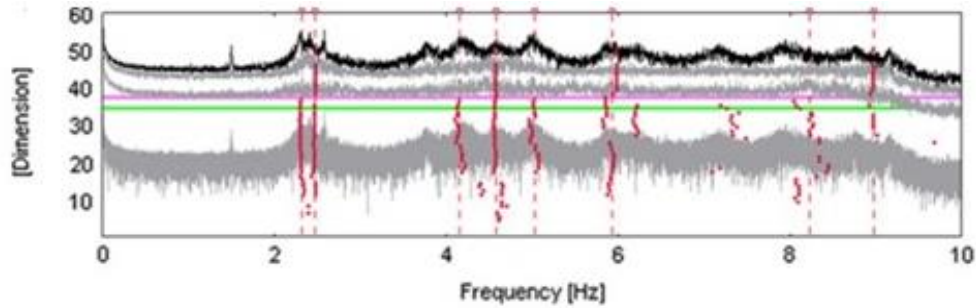
Table 6: Key features of FDD Techniques adopted in previous studies

Sn.	Reference	Remarks
1.	Gentile and Saisi (2007)	Authors found that the estimated frequency for the AVT is matching the result of field test performed in year 1992. FDD techniques were found efficient in estimating bending mode shapes of the tower structure.
2.	Casarin and Modena (2008)	The investigation showed that FDD technique was able to find the natural frequency of targeted modes. However, only few higher structural modes were detected.
3.	Cimellaro, Piantà, and De Stefano (2012)	Authors found that FDD techniques were very exact in extracting modes when input data is in the form of white noise. But FDD technique was unable to extract closely spaced frequency of structural modes.
4.	Lacanna, Ripepe, Marchetti et al. (2016)	Authors found first six structural modes by using FDD technique within frequency range of 0 to 6 Hz. The authors also conclude that FDD technique allowed to measured detailed mode shapes of the Baptistery.
5.	Conde, Ramos, Oliveira et al. (2017)	The investigation showed that FDD technique was able to find first six vibrational modes and associated frequencies of arch bridge structure. The estimated frequencies were well spaced and varied in range of 4.67 Hz to 12.45 Hz.

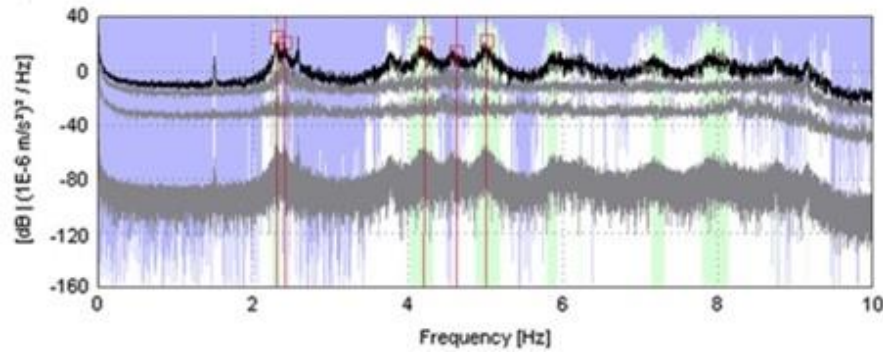
4.3 Demonstration of OMA methods

4.3.1 The tower of the Provincial Administration Building in Bari, Italy [Foti, Diaferio, Giannoccaro et al. (2012)]

The researchers carried out modal extraction from recorded AVT data by FDD and SSI techniques. Stabilization diagram by SSI technique of the output results [as shown in Fig. 17(a)] which shows that well aligned poles are formed between the frequency ranges 2-6 Hz. The same results are obtained from the spectral density matrix plot obtained from FDD technique [Fig. 17(b)] shows five mode shapes were in same frequency ranges 2-6 Hz. The results from both the methods are presented in Tab. 7 for clarity.



(a) Stabilization diagram by SSI technique



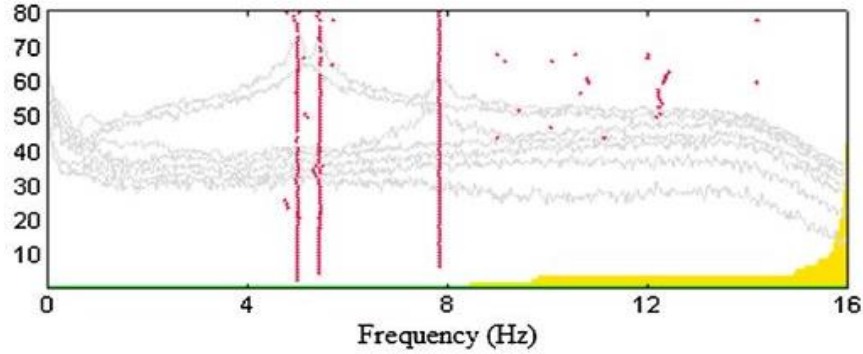
(b) Spectral density matrix by FDD technique

Figure 17: Experimental results obtained from AVT and OMA [Foti, Diaferio, Giannoccaro et al. (2012)]**Table 7:** Frequencies determined by AVT and OMA [Foti, Diaferio, Giannoccaro et al. (2012)]

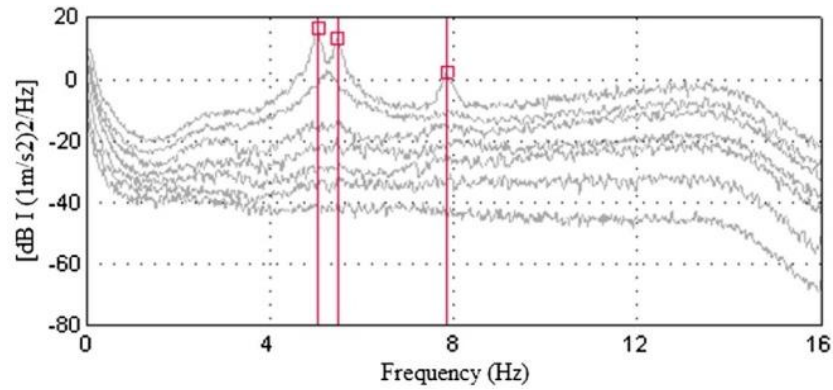
	FDD Technique (Hz)	SSI Technique (Hz)
Mode-1	2.303	2.303
Mode-2	2.398	2.439
Mode-3	4.178	4.172
Mode-4	4.594	4.594
Mode-5	5.011	5.021

4.3.2 Hamza Paşa Mausoleum, Turkey [Uçak, Bayraktar, Türker et al. (2014)]

The authors recorded and processed the data of AVT. SSI and FDD techniques were used for determining the mode shapes and respective frequencies. First three frequencies were determined by both the techniques as presented in Stabilization diagram from SSI technique [Fig. 18(a)] and Power Spectral matrix of the FDD technique is given in Fig. 18(b). Further, modes and frequencies estimated by both the methods are presented in Tab. 8 for clarity.



(a) Stabilization diagram from SSI technique



(b) Power spectral density function from FDD technique

Figure 18: Experimental results obtained from AVT and OMA [Uçak, Bayraktar, Türker et al. (2014)]

Table 8: Experimental Modes and Frequency [Uçak, Bayraktar, Türker et al. (2014)]

	FDD Technique	SSI Technique
	Frequency (Hz)	Frequency (Hz)
Mode-1	5.035	5.026
Mode-2	5.469	5.470
Mode-3	7.861	7.864

4.4 Comparison of SSI and FDD techniques

For the dynamic parameter estimation of heritage structures two major representative techniques i.e. SSI technique and FDD technique were elaborated in previous sections. These two techniques are highly complex and mathematical in nature. Though the methods are equally compatible, many studies showed that the results from both the techniques for same response dataset are not same, this may due to unwanted noise, geometrical configuration of the structure, mathematical algorithm followed by both the methods. Some of the studies are elaborated in Tab. 9 which compares the differences

found in dynamic parameter estimation by both techniques for same response dataset with advantages and disadvantages.

Table 9: Comparison of SSI and FDD techniques

Sn.	Reference	SSI Technique	FDD Technique
1.	El Borgi, Smaoui, Casciati et al. (2005)	The investigation showed SSI technique can able to estimate all the modal frequencies and damping ratio of the palace building.	Authors found that FDD technique was able to estimate only natural frequencies, mode shapes and damping ratio could not be estimated.
2.	Magalhães, Cunha, Caetano et al. (2010)	Authors found that the time taken for estimating the dynamic parameters from dataset by SSI technique was 1/4 th that was taken by FDD technique.	Authors found that the time taken for processing the dataset for estimating dynamic parameters was longer compared to SSI technique.
3.	Gentile and Saisi (2011)	Authors found that SSI technique was unable to estimate 1 st torsion mode at frequency 2.014 Hz for tower structure.	Authors concluded that Singular Value plot from FDD technique identified a torsion mode at frequency 2.014 Hz for tower structure.
4.	Aguilar, Torrealva, Ramos et al. (2012)	Investigation showed that an improved identification of nine natural frequencies from SSI techniques for 19 th century hotel building.	Authors found that only two natural frequencies were estimated by FDD techniques for same set of response data of 19 th century hotel building.
5.	Diaferio, Foti and Giannoccaro (2015)	The investigation concluded that the SSI technique is able to evaluate all the higher structural modes of the test tower structure.	FDD technique was unable to estimate higher structural modes of the test tower structure.
6.	Gattulli, Lepidi and Potenza (2016)	Investigation showed that SSI techniques estimates the structural modes clearly.	Authors found that the estimation of structural mode was difficult due to unclear peaks in spectral density plot.
7.	Altunişik, Genç, Günaydin et al. (2018)	The authors found that the first three natural frequency of tower was in range of 4.452-7.011 Hz in SSI techniques for Bastion structure. The second mode frequency observed for bastion was higher in SSI technique.	The authors found that the first three natural frequency of tower was in range of 4.452-6.967 Hz in FDD techniques for Bastion structure. The second mode frequency was 0.77% lower in FDD technique.

8.	Roselli, Malena, Mongelli et al. (2018)	Authors have found first four natural frequencies clearly for the bridge structure.	The investigation showed that FDD technique was unable to capture the second and third mode frequency of the bridge structure.
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5 Applications of system identification through AVT and OMA

AVT and OMA techniques are not just restricted only for determining system identification. There are numerous applications in the field of civil infrastructure design and assessment where the results of the AVT and OMA can be used. As far as heritage structures are concerned, there are many practical applications such as (1) finite element model updation, (2) seismic vulnerability assessment, (3) damage identification, (4) assessment of retrofitting of structural elements and (5) structural health monitoring for heritage structures, are investigated by researchers. The following section discusses the abovementioned applications briefly.

5.1 Finite Element model updation

Finite Element (FE) model updation is a simple procedure for calibrating the existing numerical model incorporating the results obtained from AVT and OMA. An existing FE model can account only the initial geometric and material properties. But in the case of heritage structure it becomes at most necessary to update the numerical model regularly for present day condition towards successful conservation. In general, FE model updation can be achieved by adjusting the physical properties, mechanical properties, boundary conditions, masses, restrained properties, joint properties and geometry of the elements in the numerical model to match against the results obtained from AVT and OMA [Pau, De Sortis, Marzellotta et al. (2005); Gentile, Saisi and Cobboi (2015); Lacanna, Ripepe, Marchetti et al. (2016)]. A common procedure followed in the literature studies for FE model updation for heritage structures is presented in Fig. 19.

Ramos et al. [Ramos, Aguilar, Lourenço et al. (2013)] used four mechanical parameters such as Young's modulus of masonry, stiffness of the soil near to the structure, normal and shear stiffness of the contacting elements to update the model of Mogadouro Clock Tower, Portugal using Douglas-Reid method [Douglas and Reid (1982)]. Votsis et al. [Votsis, Kyriades, Chrysostomou et al. (2012)] used sensitivity analysis [Mottershead, Link and Friswell (2011)] for performing model updation of St. Mamas Church, Cyprus where Young's Modulus was chosen as variable to calibrate the numerical model against the results obtained from AVT and OMA. Foti et al. [Foti, Diaferio, Giannoccaro et al. (2012)] used 96 parameter for calibrating the FE model of Provincial Administration Building, Italy by sensitivity analysis. These 96 parameters included 11 Young's Modulus, 11 densities, 30 springs (to incorporate stiffness of the building) and 44 masses (to account the masses of non-structural elements). Tab. 10 presents the parameters used for FE model updation of heritage structures which are available in literature.

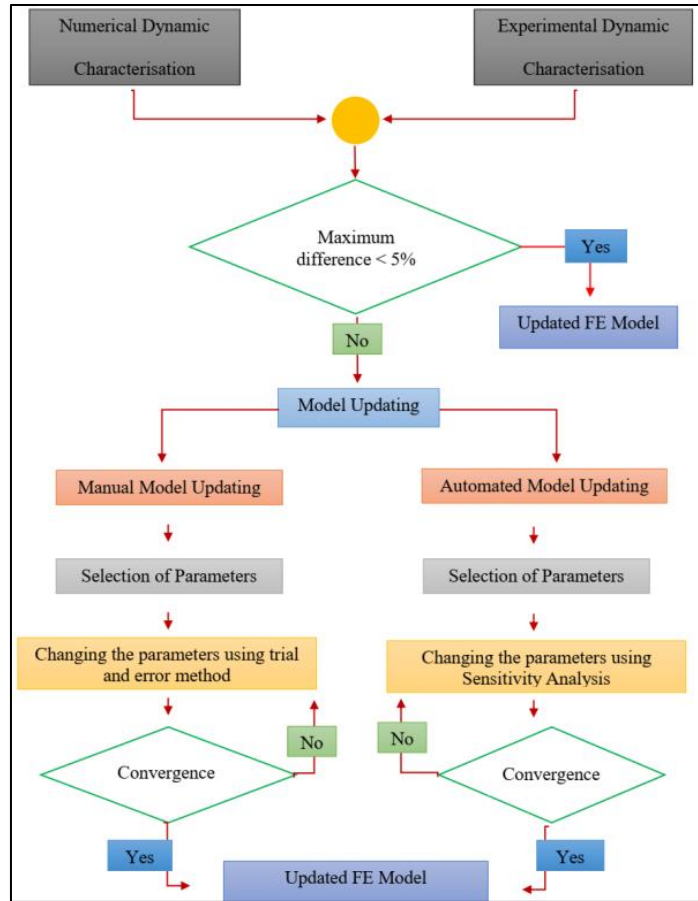


Figure 19: FE model update through AVT and OMA

Table 10: Parameters used for updating FE model in previous studies

Sn.	Reference	Structure	Optimization technique	Assumed Parameters	Modified Parameters
1.	Foti, Diaferio, Giannoccaro et al. (2012)	Provincial Administration Building, Italy	Douglas-Reid method	$E=2.2 \times 10^{11} \text{ N/m}^2$ $\rho=2.2 \times 10^3 \text{ kg/m}^3$	$E=3.3 \times 10^{11} \text{ N/m}^2$ $\rho=2.197 \times 10^3 \text{ kg/m}^3$
2.	Votsis, Kyriades, Chrysostomou et al. (2012)	St. Nicholas Cathedral	Sensitivity analysis	$E=603 \text{ MPa}$	$E=702 \text{ MPa}$
3.	D'Ambrisi, Mariani and Mezzi (2012)	The medieval civic tower of Soncino	Sensitivity analysis	$E=805 \text{ MPa}$	$E=1000 \text{ MPa}$

4.	Ramos, Aguilar, Lourenço et al. (2013)	Saint Torcato church	Sensitivity analysis	$E_{\text{masonry}}=10 \text{ GPa}$ $E_{\text{soil}}=3.9 \text{ GPa}$	$E_{\text{masonry}}=5.64 \text{ GPa}$ $E_{\text{soil}}=0.63 \text{ GPa}$
5.	Cimellaro, Piantà, and De Stefano (2012)	L'Aquila city hall;	Sensitivity analysis	$E_{\text{masonry}}=1500 \text{ N/mm}^2$ $E_{\text{stones}}=4200 \text{ N/mm}^2$ $E_{\text{column}}=6300 \text{ N/mm}^2$	$E_{\text{masonry}}=1100 \text{ N/mm}^2$ $E_{\text{stones}}=3200 \text{ N/mm}^2$ $E_{\text{column}}=4900 \text{ N/mm}^2$
6.	Uçak, Bayraktar, Türker et al. (2014)	Hamza Pasa Mausoleum	Sensitivity analysis	$E_{\text{stone}}=2 \times 10^9 \text{ N/m}^2$ $E_{\text{strecher}}=2 \times 10^{11} \text{ N/m}^2$ $\rho_{\text{stone}}=2169 \text{ kg/m}^3$ $\rho_{\text{strecher}}=7850 \text{ kg/m}^3$ $\mu_{\text{stone}}=0.2$ $\mu_{\text{strecher}}=0.3$	$E_{\text{stone}}=1.8 \times 10^9 \text{ N/m}^2$ $E_{\text{strecher}}=2 \times 10^{11} \text{ N/m}^2$ $\rho_{\text{stone}}=1850 \text{ kg/m}^3$ $\rho_{\text{strecher}}=7850 \text{ kg/m}^3$ $\mu_{\text{stone}}=0.2$ $\mu_{\text{strecher}}=0.3$
7.	Conde, Ramos, Oliveira et al. (2017)	Vilanova bridge	Sensitivity analysis	$E_{\text{arches}}=1 \text{ GPa}$ $E_{\text{spandrel}}=1 \text{ GPa}$ $E_{\text{peirs}}=1 \text{ GPa}$ $E_{\text{backing}}=0.1 \text{ GPa}$ $E_{\text{infills}}=0.1 \text{ GPa}$	$E_{\text{arches}}=2.66 \text{ GPa}$ $E_{\text{spandrel}}=1 \text{ GPa}$ $E_{\text{peirs}}=3 \text{ GPa}$ $E_{\text{backing}}=0.18 \text{ GPa}$ $E_{\text{infills}}=0.10 \text{ GPa}$

5.2 Seismic assessment

Seismic assessment is a generalized procedure for determining the capacity of the structures while resisting seismic loads. Generally, seismic assessment is performed upon understanding the non-linear seismic response of the updated FE model of the test structure against various seismic loads. An updated numerical FE model can enhance the reliability of the results of seismic analysis and assessment. Seismic analysis of an updated FE model of heritage structures allows identification of weakest structural component and also quantifies the seismic vulnerability of structure as whole [De Matteis and Mazzolani (2010); D'Ambrisi, Mariani and Mezzi (2012)]. Peña et al. [Peña, Lourenço, Mendes et al. (2010)] performed non-linear seismic analysis on Brick minaret named Qutub Minar situated in India using a FE updated model along with synthetic acelerogram readings compatible with Indian Seismic Code. D'Ambrisi et al. [D'Ambrisi, Mariani and Mezzi (2012)] performed three types of seismic analysis (i.e. linear static analysis, non-linear static analysis and non-linear dynamic analysis) on numerical model calibrated with AVT and OMA results to understand seismic behaviour of Civil Tower of Soncino, situated in Italy. De Matteis and Mazzolani [De Matteis and Mazzolani (2010)] performed limit analysis on The Church of Fassanova Abbey situated in Italy for identification of vulnerable structural elements and determining the current seismic vulnerability. Likewise, there are more studies available in literature for seismic assessment of heritage structures performed after AVT and OMA [Jaishi, Ren, Zong et al. (2003); Pradhan, Singhal, Mondal et al. (2012); Ferraioli, Miccoli, Abruzzese et al. (2017)].

5.3 Dynamic health monitoring

Dynamic monitoring systems are valuable for predicting long-term structural evaluation. These systems (consist of accelerometers, data acquisition system) are installed on the heritage structures for tracking changes in the global frequency with respect to time, as well as for acquiring the data required for the subsequent implementation of damage notification tools [Gentile, Saisi and Cabboi (2012)]. The main objective of dynamic monitoring system is to correlate the variation of natural frequencies with damage and environmental effects [Gentile and Saisi (2012)]. The different phases involved in dynamic monitoring are, (1) model updating, (2) results interpretation, (3) highlighting the peculiarities when dealing with architectural heritage structures [Ceravolo, Pistone, Fragonara et al. (2016)].

Ramos et al. [Ramos, Marques, Lourenço et al. (2010)] conducted dynamic monitoring for Mogadouro Tower, Portugal. The dynamic monitoring system was installed in April 2006 to evaluate environmental effects and to detect non stabilized phenomenon in structure. The task was performed for different time period during year. Ten series of recording were completed from April 2006 to December 2017. The study concluded that the water absorption during raining season can increase frequency upto 4%.

Gattulli et al. [Gattulli, Lepidi and Potenza (2016)] conducted monitoring of Basilica of S. Maria Di Collemaggio in June 2011. A total of 16 wireless sensors were installed all over the structure. The monitoring was aimed for investigating causes of collapse, performance of scaffoldings and long term dynamic response of the structure and updation of FE model. The study concluded that dynamic monitoring was effective tool for damage identification and evaluation of performance of structure during operational condition.

5.4 Repair and retrofitting

The AVT and OMA can be performed before and after the repair and retrofitting of the structure. Many studies states that AVT and OMA can be safely used to observe the restoration effects by determining dynamic response of heritage structures. Fig. 20 shows the common procedure adopted by literature studies for the assessment of repair and restoration of heritage structures. Few noticeable studies on the assessment of repair and rehabilitations of heritage structures using AVT and OMA can be seen from Tab. 11.

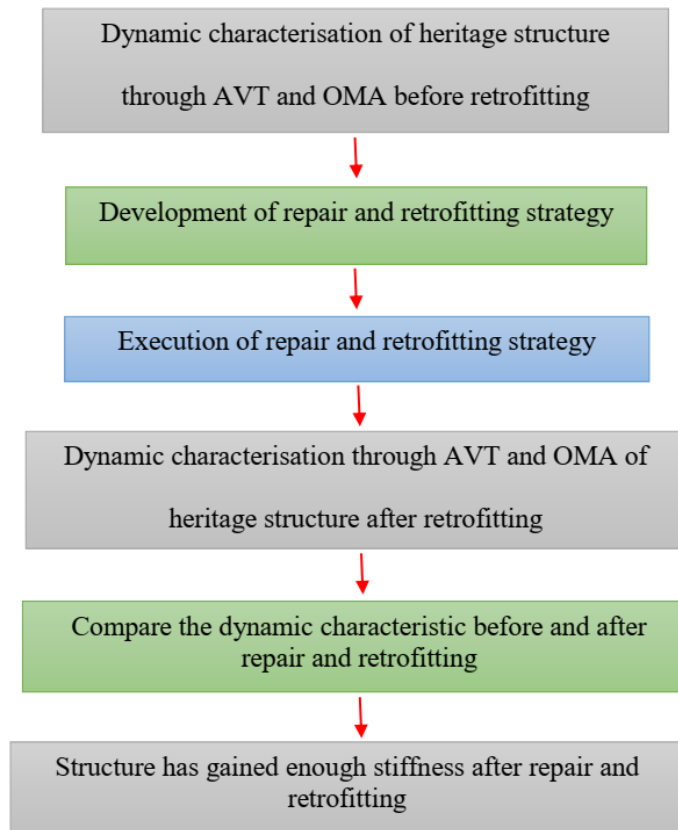


Figure 20: Procedure for repair and rehabilitation through AVT and OMA

Table 9: Summary of repair and retrofitting case studies

Sn.	Reference	Reference Structure	Description
1.	Çalik, Bayraktar, Türker et al. (2014)	The Vault of Bahçecik Küçük Fatih Mosque, Turkey	The investigation showed the results from AVT and OMA before and after the restoration work was found to be very close. The first five natural frequencies of the repaired case of the vault were identified in the frequency range of 9.371-27.850 Hz, which was more than pre-restored case of the structure. It was concluded that the increased frequencies were due to increase in the rigidity of the structure due to restoration work.

2.	Osmancikli, Uçat, Tarum et al. (2012)	The bell-tower of Hagia Sophia Church, Turkey	The authors performed AVT and OMA for Hagia Sophia Bell Tower in pre-restored and restored condition. The authors found that the first and second bending modes of the restored tower had close frequency values as pre-restored case. The mode shapes of the tower for restored and pre-restored cases were approximately the same. The average modal damping ratios of the tower were also decreased from 2.69% to 0.6%. Further it was concluded that the restoration was effective.
3.	Ercan (2018)	The “Grand House”, Turkey	The author conducted AVT and OMA masonry structure before and after retrofitting. The study showed that the 1 st modal frequency had increased three times as compared to initial condition. The author also concluded that the AVT and OMA are most reliable method to evaluate the impact of retrofitting of the heritage structures.

6 Conclusion

This review paper summarised detailed procedures, methodologies, instrumentation, and application of AVT and OMA carried out by research community for past 20 years on heritage structures for successful system identification. Further, the paper has appraised various outcomes of these investigations and highlighted the applications of system identification. Following are some of the major conclusions drawn from this paper:

- AVT and OMA are found to be most reliable procedure for system identification (identifying mode shapes, frequency, damping etc.) of heritage structures. Further, the results determined from AVT are quite dependable with minimum errors.
- It is learnt that primary survey is an essential step prior to performing AVT. The primary investigation includes understanding the historical data, whole geometry, and material characteristics. An apt primary survey will be a leap step in understanding the response of the structure under operational conditions as well as sources of the errors.
- Investigations showed that knowledge of excitation sources along with geometry of the structure will help in selecting suitable instrumentation. Adding to this, the accelerometers should be sufficiently sensitive in order to capture the smallest response of the structure. Further, the accelerometer locations should be chosen such that whole structure to be covered in order to get full response. If there are not

- enough number of accelerometers available, multiple deployment technique should be used carefully. Some time it is not possible to put accelerometers on prerequisite positions, in that case one can choose the location nearer to actual position.
- The investigations showed that the natural sources like wind, traffic, operational excitation sources are useful for exciting the structure for vibration measurement. Sometime, these excitations are not enough for exciting the whole structure for targeted modes. In such cases, the better way to excite the structure for essential mode is to use impact hammer in different directions on different locations. Another artificial method is to use micro-tremor which generates the vibration of low level, which can be useful for excitation of the structure.
 - The literature shows that two major OMA techniques (time domain as well as frequency domain) should be performed simultaneously and the results of the methods should be compared and validated.

7 Future scope

- Generally, the results of AVT and OMA are used to update the FE model such that the dynamic characteristics of FE model should match with the response estimated by AVT and OMA. However, during FE model construction, the material characteristics are generally assumed which perhaps can be harvested from some non-destructive tests (NDTs) such as ultrasonic pulse velocity, rebound hammer, flat jack test etc. so that the accuracy of the response of the structure is improved alongside of AVT and OMA and updated FE model. At present very few studies have practised NDTs along with AVT and OMA which may be considered as mandatory.
- Many heritage structures are being controlled and maintained within permissible vibrations by active/passive vibration control systems. The implementation of these control systems will be effective if they are designed using the results obtained from AVT and OMA. Till now, no applications are found in this avenue.
- The complex geometry of heritage structure always demands optimized placement of sensor network. Only few studies were found and more studies will pave clear direction to achieve best sensor network to capture the response of whole structure.
- The advent of wireless sensors and their effectiveness to be explored for AVT and OMA in case of heritage structures.
- Present OMA techniques are highly complex mathematical in nature. This put practising engineers to undergo rigorous computations. Simplified algorithms for OMA will be effectively useful for practising engineers towards system identification.
- No studies are found in AVT and OMA incorporating Soil structure interaction with heritage structures. Conducting such studies can be very effective in predicting seismic vulnerability of heritage structures.
- There are strong requirements of understanding environmental effects like temperature variation, humidity, rain etc. while evaluating the life and deterioration of heritage structures. Pursuing such studies with AVT and OMA will provide a new scope for the maintenance schemes and plans for heritage structures.

- There is strong need of a common platform where all the AVT and OMA data on different heritage structures shall be stored and maintained, which will help the research community for progressing in field of the study.

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