4. Analysis and monitoring of oval domes

This chapter is about the dynamic and seismic behaviour of domes. After an introduction to domes and their geometric and structural peculiarities, the coverage focuses on three ideal benchmarks on reconciling geometric survey with dynamic monitoring. The analyses concerned structures with oval shape domes, such as the Sanctuary of Vicoforte, S. Caterina in Casale Monferrato and S. Agostino in L'Aquila. The final products are virtual model which were enabled to predict the linear dynamic response under earthquake excitation.

4.1 Geometry and structural peculiarities of oval domes

4.1.1 Domes in historical architecture

The first examples of domes date back to 4000 BC in Mesopotamia, realised by cantilevering layers of masonry from a circular or oval plant and achieving the top of the building. These domes were of the type that nowadays we refer to as "false domes". Few examples of domes can be found in Egyptian and Greek architecture [1]. In fact it is only during the Roman age that the concept of arch and dome were explored and used in any type of application. The arch has probably been the most important innovation in the history of architecture, transforming, thanks to the curvature associated with the thrust at the springs, bending moments in compression forces, even if a certain bending strength is indispensable to maintain a stable shape. Spatial vaults (usually with circular, square or polygonal bases) are a development of the arch concept. Their particularly satisfactory behaviour is due to the double curvature and mainly to the hoop effect of horizontal rings. As previously stated, the first real domes arise and develop with the Romans and the dome of the Octagonal Room in the Domus Aurea (I century) is the first important example (Figure 4.1).

The Pantheon in Rome (Figure 4.1) is the first notorious example of dome: it is a very simple structure made of a cylinder (the Rotunda) and a hemispheric dome of the same diameter (around 43 m), though at the same time it is full of intuition and innovation [2]. From inside the dome is clearly a hemisphere where the meridians spring vertically from the cylinder itself. From the outside, the cylinder appears higher than from inside and the dome emerges from the cornice with a flatter shape. It is interesting to note that the "steps" visible on the extrados of the dome are not an architectural choice but the consequence of the technique of pouring the concrete in subsequent rings. From outside, the cylinder appears as a big brick wall, containing within its thickness a series of arches which inside correspond to niches and empty spaces.


Figure 4.1 - Dome of the Octagonal room (left), Pantheon (right).

Hagia Sophia (built in the present shape under the Emperor Justinian I in the 6th century) chronologically is the second biggest dome in the history of architecture and got its inspiration from the Pantheon. However, Hagia Sophia features some important differences related to the fact that the dome is supported by four huge pillars placed on the corners of an ideal square base (32 m). Two problems arise:

- how to resist the circumferential forces at the border of the dome;
- how to transfer the vertical forces from the meridians to the pillars.

The solution of the first problem was the introduction of hemi-domes and abutments to balance the thrusts while "pendentives" on the four corners, associated with arches, have solved the second issue, allowing the forces to flow from the top to the ground. These innovations turned out to be very important; most of the following domes are inspired on these principles [2].

The dome of the Church of S. Maria del Fiore (Brunelleschi, 15th century - fig. 16) is the first example of a big dome with a double shell on an octagonal plan. The dome, having a diameter of 43 m similarly to the Pantheon, received its inspiration from the Gothic vaults. In order to reduce the thrust, the shape is ogival and to reduce the weight, the main bearing structure is made of 8 principal ribs (or spurs) in the corners and 16 supplementary ribs in the middle of the webs (or segments of the shells). The circumferential connection is ensured by 4 stone ribs (a kind of "chains of stone" reinforced with steel clamps) and a wooden chain; in addition small horizontal arches improve the connection between the corner ribs and the adjacent ribs on the webs.

Figure 4.2 - Cross section (top) and interior view (bottom) of the dome of Hagia Sofia.

One of the main issues that Brunelleschi had to solve was how to build the dome without scaffolding, which would have been too big and too heavy (see figure 4.3). The octagonal shape of the dome, proper also of drum and plan of the church underneath, lets reasonably imagine that seismic behaviour with, as already said, the "stone chains" or supplementary steel chains are able to provide tensile strength. Some small cracks visible on shells, in the zones over the windows, appear to be related more to the phase of construction than to seismic effects (which, on the other hand, are very low in Florence) [3].

The octagonal base of the dome is made of four huge pillars on four sides and four arches in between so that the supporting structure is stiff, and even more stiffened by the connection with small hemi-domes, probably inspired by Hagia Sophia.

On the top of the dome there is a lantern; this implies that differently from the Pantheon the meridians arrive at the edge-ring of the oculus with a smaller inclination with respect to the vertical line, necessary to support the weight of the lantern.

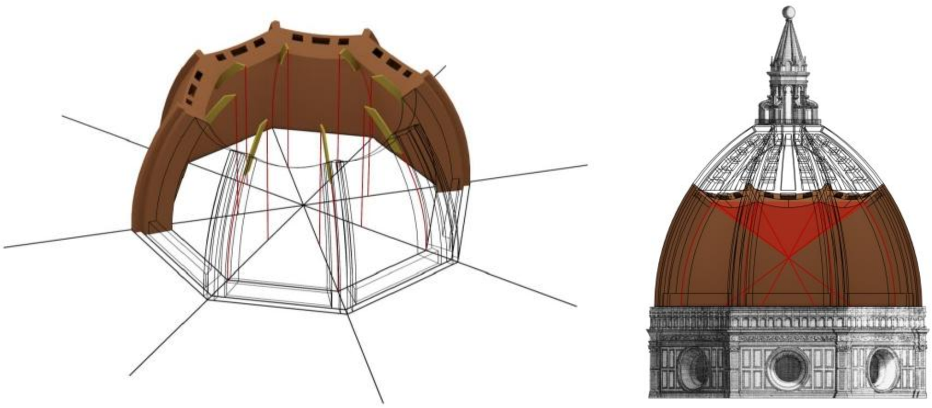


Figure 4.3 - The rampant scaffoldings system referred to a unique central point for the eight edges tracking probably used by Brunelleschi [4] (left) and Brunelleschi' invention for the dome stability: bricks laying on curved "beds" (corda branda) constitute conical and continuous surfaces, combined to the adoption of angular staggered bricks in correspondence to the corners Brunelleschi [4].

San Pietro's dome (16th century) represents the last exceptional dome ever built, having a diameter of around 42 m and it is similar to the Pantheon, Hagia Sophia and Santa Maria del Fiore. The history of San Pietro's dome is particularly troubled: several architects followed one another together with several projects [2,5]. The final project is due to Michelangelo while the construction was headed by Giacomo della Porta since Michelangelo died previously. A significant difference between the domes of Brunelleschi and Michelangelo is in the shape of the drum, that here is circular, even if the dome is made with 16 ribs. The circumferential stone chains of S. Maria del Fiore, in St. Peter are replaced with steel chains. From seismic point of view there are certain analogies between the structural scheme of Hagia Sophia and S. Peter. As in both cases the forces are obliged to flow from the circular plane of the dome to the four columns placed on the corners of an ideal square, through four arches and relative pendentives. However, the substantial difference is that in St. Peter there is a strong drum whilst in Hagia Sophia, as we have seen, not only the drum is absent but the base of the dome is weakened by a series of windows.

4.1.2 Oval geometry in historical architecture

Few examples of oval vaults and domes are present in the Egyptian architecture while, the masters of arches and domes of the ancient world, the Romans, used the oval shape in few cases, such as the plant for amphitheatres or in few examples of arch bridges [1].

Most medieval domes have a centralised form, probably due to the Roman influence. However, there are also some exceptions, and the church of Santo Tomás de Olla (Léon, Spain), dated to the tenth century, presents an octagonal dome on an oval plant, 6 x 5.5 m, surrounded by horseshoe arches [1].

In late Gothic, however, in England, Germany and Spain, the Gothic masters began to employ oval forms, generated by the tangency of circles of different radii, particularly in vaults and arches.

The idea of using the oval in different aspects of the arts was "in the air" at the beginning of the Cinquecento. Michelangelo's first project for the Tomb of Julius II already contained an interior oval space. Correggio was the first painter to introduce an oval in a composition (Madonna of St. Francis, 1514, Dresden Gemäldegalerie), and Gian Maria Falconetto the first sculptor to employ one. It seems clear that the oval form exerted a new attraction to the artists at the beginning of the Cinquecento. The main architects in promoting the oval as a new form of defining the architectural space were Baldassare Peruzzi, Sebastiano Serlio and Giacomo Vignola. It was Peruzzi who first thought in taking advantage from the peculiarities of an oval space in church design, a compromise between the central space of the Quattrocento and the more linear character of traditional churches. However, his death left the diffusion of his ideas in the hands of his disciple Serlio. Indeed, Serlio's treatise [6], one of the most popular architectural treatises ever published, was responsible of the spread of the oval form in the late Renaissance and Baroque in Europe. In his Book I on geometry [6], published in 1545, he includes a discussion on ovals. He says explicitly that it is possible to draw many different ovals and proposed four oval constructions (Figure 4.4). These were copied again and again in later architectural manuals and were used many times in actual designs. However, architects and masons knew that for any two axes it is possible to construct many (in fact, infinite) different ovals and they departed from Serlio's models when desired [1].

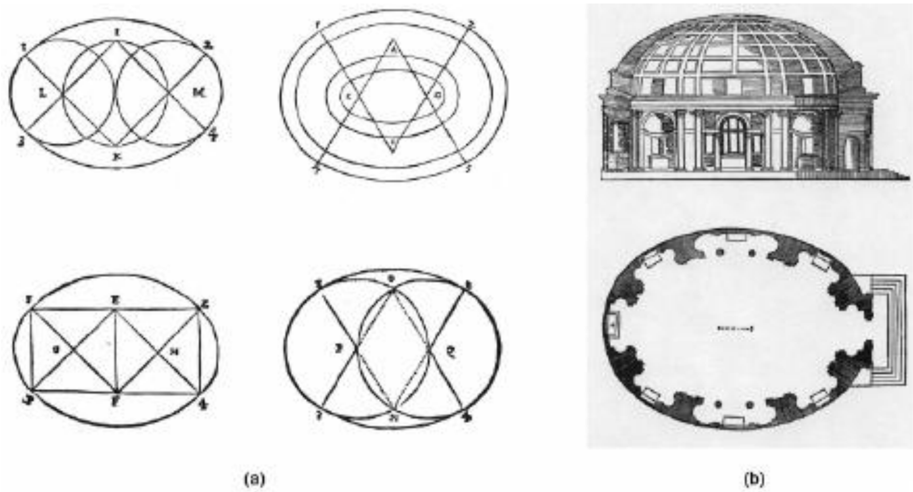


Figure 4.4 - Serlio's models for ovals in his Book I of 1545; b) Serlio's design of an oval temple in his Book V of 1547.

The best architects of the Baroque exercised their ingenuity by solving the problems created by a non-central space. For example, Smyth-Pinney [7] has studied in detail the design process followed by Bernini for the plan of S. Andrea al Quirinale and the subtle position of the axes of the perimeter chapels. Bernini's oval does not correspond to any of Serlio's models, and shows a delicate adjustment in the interior space (Figure 4.5).

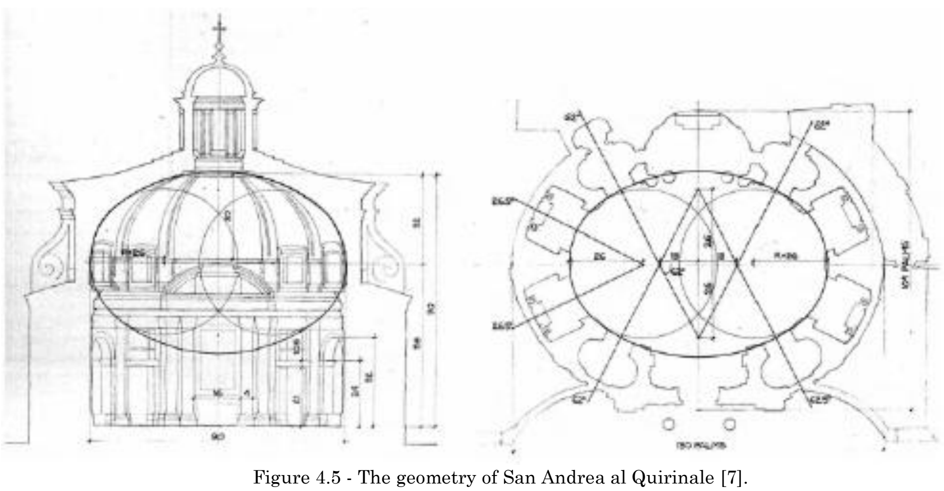


Figure 4.5 - The geometry of San Andrea al Quirinale [7].

The project of Francesco Borromini for San Carlo alle Quattro Fontane (1663) presents a more complicated geometry, as the oval which generates the plan changes at the base of the dome (Figure 4.6). This last oval deviates very much from the usual form of ovals so far. No doubt, Borromini chose this form to provide “tension” in the space. Neither of the ovals corresponds with Serlio’s models [1].

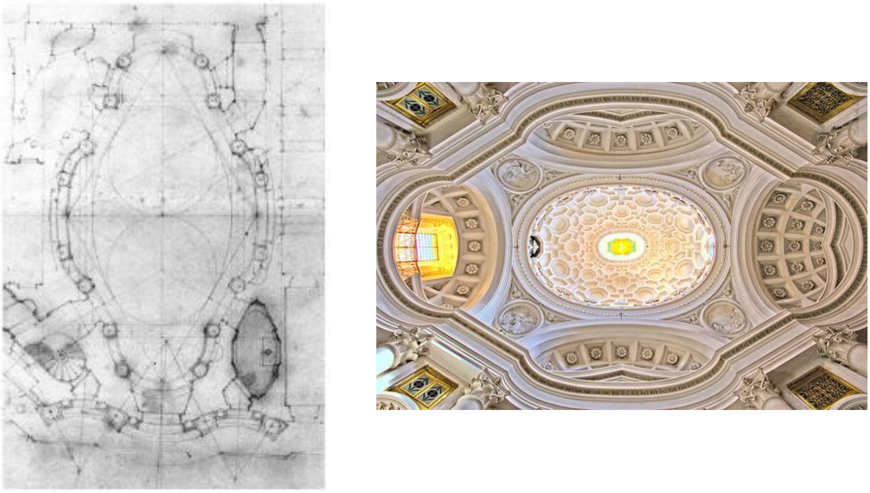


Figure 4.6 - San Carlo alle Quattro Fontane by Borromini.

4.1.3 Structural monitoring of domes

In this brief review about domes, it is interesting to analyse few examples monitoring systems existing on some important domes. Two examples of permanent monitoring will be analysed: the traditional monitoring system installed in Santa Maria del Fiore and the more recent monitoring system installed in the church of Anime Sante in L’Aquila after the 2009 earthquake. Finally, a review of the experimental modal analysis performed on the Hagia Sophia dome after the 1999 earthquake will be presented.

4.1.3.1 A traditional monitoring system: Santa Maria del Fiore

Blasi and Ottoni have recently presented a review of the monitoring system of Santa Maria del Fiore’s dome [4]. This dome is affected by widespread crack pattern, substantially symmetric, which seems to confirm the collapse mechanism typical of the domes, with a drop of the top under its own weight. The cracks symmetry evidences some notable variations with a concentration on the “peer slices”, due to the different underlying bearing structures (the huge pillars instead of the wide arches).

Actually, the main passing cracks of Santa Maria del Fiore are on webs 4 and 6 (clockwise direction, web 1 facing the nave). Two minor cracks, quite symmetric, are visible in webs 2 and 8 and numerous inclined cracks stay near the eyes, in the uneven webs, above the keystones of the underlying arches, constituting a minor cracks system, while not passing cracks stand in the 8 edges of the dome.

During centuries numerous monitoring systems (spie of marble, stone, alloys, iron wedges, up to modern digital deformometers) have been positioned on these cracks, in order to control their evolution in time. The most impressive are certainly the last ones, installed in the last century and nowadays still working on Santa Maria del Fiore dome: the mechanical system installed by Opera del Duomo in 1955 and the digital one placed by ISMES in 1987.

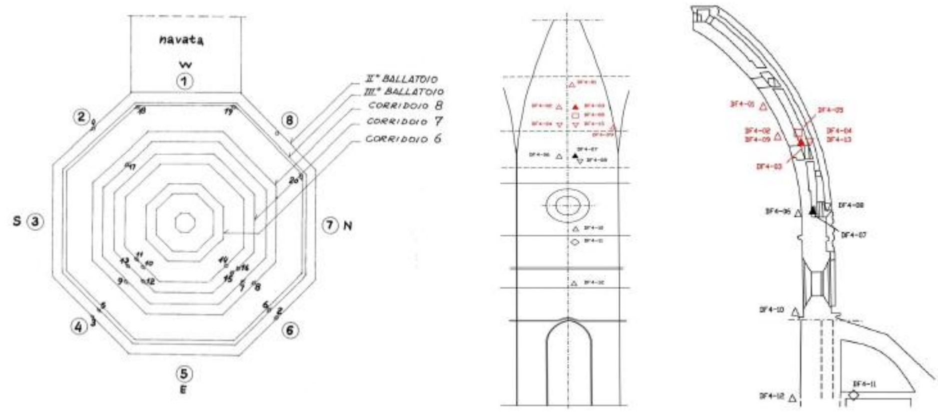
The first system (from 1955 to 2009) has been installed by Opera del Duomo on the major cracks of the inner dome, following the indications of the coeval structural Ministerial Committee (Pier Luigi Nervi and Padre Alfani). It is constitutes by 22 mechanical deformometers which record cracks width variations four times a year. 214 data for each instrument (conserved into the archives of Opera del Duomo) are available at present, having recorded the last 55 years cracks width evolution, together with dome internal and external temperatures. During this long period - maybe the longest continuously monitored for a monument - different events have occurred (earthquakes, groundwater level variations, windstorms and the 1966 flood).

The second system, more articulated, was placed by ISMES in 1987 on the two domes, following the indications of the last Ministerial Committee (composed by Salvatore Di Pasquale and Andrea Chiarugi). It is composed by 166 instruments which register not only the cracks width but even the structural movements most important in the description of the dome conditions (temperature, vertical displacements, inclination and underground water levels). In this case, 72 displacements transistors (Dfn-mm) inductive types, with a precision of +/- 0.02 mm, are placed on the main cracks of the inner and outer domes, at five different levels. The system is then completed by 8 plumb-lines at the centre of each web measuring relative displacements between pillars and tambour, 8 livellometers and two piezometers (located near the web n. 4, and under the nave).

In his report of 1936 P.L. Nervi hypothesized the temperature to be the main cause of the crack width variations. The first Commission monitoring data elaboration (made by Padre Alfani after one year long observation), even finding significant movements for major cracks in webs 4 and 6 (deformometers 6-7 on web 4 and deformometers 9-10 on web 6 registered 0.5 mm maximum width variation), evidenced a strict connection between cracks and temperature behaviour, moreover concluding that the cracks would be substantially steady, returning (in correspondence to temperature variation) at the starting width. In the same Commission's report a deep investigation of the hypothesized relations is recommended in order to fully understand the problem with a most significant numbers of data.

In step with this, during the last 20 years both air and masonry temperatures have been monitored and the ISMES system includes 60 thermometers, measuring masonry and air temperature in the two domes (TMn-mm and Tan-mm) on each web, at the second corridor level (+/- 0.05°C precision). The acquisition system registers data every six hours, starting at 6.00 AM every day and 20 years data, recorded from 8th January 1987 to 31st July 2007 (about 31373 measures for each instrument, five million in total).

In 1986, 48 deformometers have been added by Soprintendenza ai Beni Architettonici. These further instruments have been installed above the toll collector holes (6 for each slide) in order to register width variations of the lintels cracks. These cracks in fact are merely secondary considering the global crack pattern of the dome, but this monitoring system, if related to the previous ones, can give important information about the dome behaviour.



(a) (b)

Figure 4.7 – (a) First monitoring system on Santa Maria del Fiore installed in 1955; (b) Second monitoring system installed in 1987 [4].

The systems above described are, considered together, the most significant and complex monitoring system nowadays present on an historical monument, not only for the huge number of instruments installed but also for the exceptional duration of measurements.

At the end of Eighties, after the first year of digital monitoring of the monument, the Civil Engineering Department of the University of Florence carried out a long report [8] considering and analysing the first collected data. These partial results contribute to the furious polemic on dome's stability and on the supposed damages caused by the encircling scaffolding installed on the dome for restoration of the frescoes. In the same years, following the studies by Di Pasquale, Chiarugi, Fanelli and others, the "secrets" of the dome were unravelled and some hypothesis on cracks causes and evolution in time were formulated.

Previous studies have already examined the data recorded by these monitoring systems until 1996 (Blasi [9], Chiarugi *et al* [10], Gabbanini and Vannucci [11]) finding the global trend of the deformometers and suggesting a relation with temperature. Moreover, ISMES had produced, once a year, a report on instruments regular functionality. However, by a comparative process of historical reports, a linear progression of the cracks widths of about 5 or 6 mm for century can be traced.

The huge number of available data has not been properly used until now, never being object of a systematic elaboration which would have compared cracks evolution to the traumatic events occurred to the monument during the last decades (earthquakes, wind and high thermal variations).

Nevertheless, dome mechanical behaviour and its response to seismic events and environmental conditions are hidden in these monitoring data and, for the more numerous data available at this time (thanks to the strict support of Opera del Duomo), some new considerations on the dome structural interpretation and possible consolidation opportunities can be advanced.

4.1.3.2 The monitoring system of the church delle Anime Sante in L'Aquila

Santa Maria del Suffragio (commonly called the church of Anime Sante) is an 18th century church in L'Aquila, Italy. It started being built on 1713 and in 1770 was added a baroque façade. Later on, in 1805, the church was completed with a neoclassical dome by Giuseppe Valadier [12].

Figure 4.8 - Church of Anime Sante in L'Aquila after the 2009 earthquake (left) and the temporary scaffold (right).

The church of Santa Maria del Suffragio in L'Aquila has suffered severe damage during 6th April 2009 earthquake. The monitoring program is started at November 2009 - born by relationship between the Research Unit "Monitoring of Structural Heritage" of IUAV University of Venice and Historical Heritage Management of Abruzzo leaded by Prof. Russo [13] - is defined by static local control to check the wide crack variation and by global dynamic identification. The static and dynamic monitoring systems are constituted by 8 displacement transducers and 20 accelerometers (16 monodirectional and 4 tridirectional) respectively for 28 accelerometer directions (Figure 4.9).

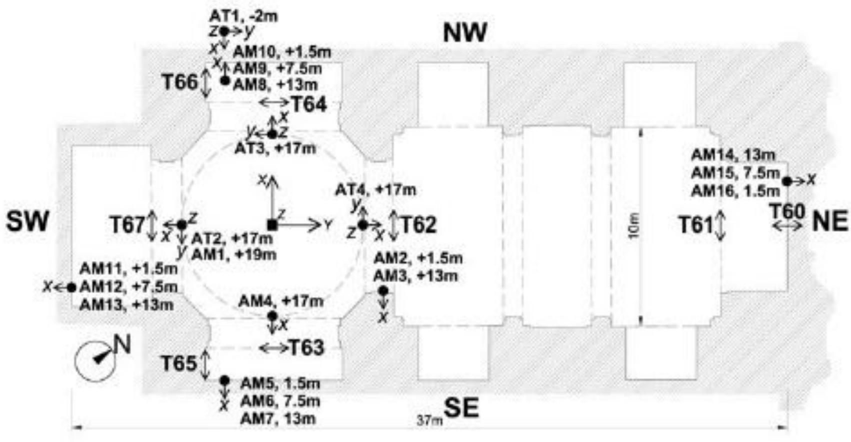


Figure 4.9 - Measurement equipment installed in the Church of Anime Sante.

The monitoring equipment consists in linear displacement transducers (identified in figure 4.9 as T60,…, Tth) and accelerometric sensors, mono-directional (identified as AM1, AM2, …, AMth) in x local direction and tri-directional (AT1, AT2, …, ATth) for x, y, e z local directions. The double arrow of each displacement transducers shows the axial direction of crack opening, while the accelerometric directions are defined by local coordinate system x, y and z. The structure's geometry is defined by global coordinate system X, Y e Z, see figure 4.9.

For the dynamic identification the time history of the earthquake-induced ground motion - recorded march 16, 2010 in Aquilano's station by the INGV (Istituto Nazionale di Geofisica e Vulcanologia) network - has been used. The seismic event was decomposed in X, Y and Z direction considering the signal recorded by AT1 accelerometer, see figure 4.9.

The analysis in frequency domain has been carried out in [13] using Frequency Domain Decomposition technique (FDD), allowing to determine the first three modes of vibration. Damping for the mode has been determined using the Half Power Bandwidth method.

For what concerns the static monitoring, The linear displacement transducers, of all static equipment (depicted in figure 4.9), that monitor the structural system of tambour/dome macroelement are T62, T63, T64 and T67. In particular the monitored cracks are in keystone of transept (T63 and T64), abs (T67) and nave-transept (T62) arches.

The analysed period, December 2009-August 2011, allows considering two cycles of temperature (°C), winter and summer; the variation of crack opening which are not directly related to temperature but dependent on structural problems can be determined.

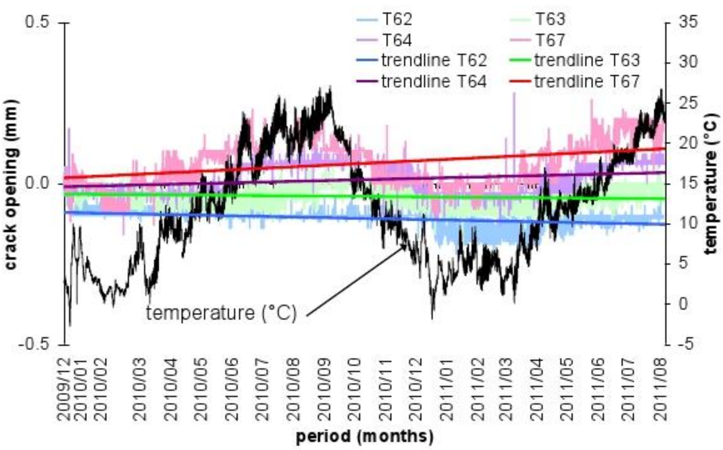


Figure 4.10 - Tendency of cracks opening in the period 12/2009-08/2011.

Figure 4.10 shows the crack opening variation, the trendline of each linear displacement transducers and the ambient temperature. By varying the temperature the sensor T67 recorded a crack opening by 0.23 mm; the other linear displacement transducers record the smallest crack opening (below 0.1 mm). The residue variation of opening crack has been recorded during January 2010-January 2011 period by T62 transducer. The crack width reduction is equal to 0.12 mm, while for the other sensors are less than 0.03 mm.

Finally, by using the data collected by these monitoring systems, a FE model of the whole church has been realised and updated using dynamic measurements, in order to perform a seismic retrofit of the building [13].

4.1.3.3 Dynamic identification of Hagia Sophia after the 1999 earthquake

The last example regards Hagia Sophia and it is an example of periodic monitoring of a dome. Hagia Sophia received slight damage during the Kocaeli earthquake of 1999, which with a moment magnitude (Mw) of 7.4 is one of the largest earthquakes in recent times on the Anatolian plate in Turkey. Even if the damage reported by visual inspection was reported to be light, due to the great importance of the building a periodic dynamic monitoring of the building was scheduled [14].

In order to record the ambient vibrations of the structure, a total of 10 single-component seismometers were mounted at critical points as shown in figure 4.11. Locations of the sensors are key points believed to reflect the structural characteristics. L22 type low frequency seismometers with peak-to-peak 0.15 in amplitude precision were utilized for the tests. Digitized data was acquired at a rate of 122 Hz through an A/D converter. In order to quantify and minimise the effects of changing environmental conditions, four sets of ambient vibration measurements were carried out to identify the dynamic characteristics of the structure along with the free-field measurements in the courtyard. Ambient vibrations were recorded for each structural point for duration of 5 min in each setup.

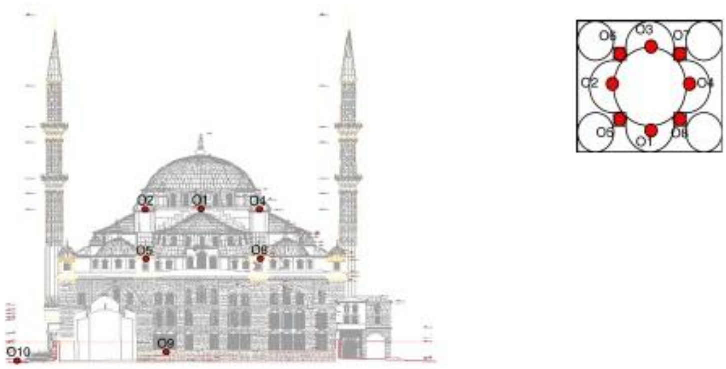


Figure 4.11 - Seismometer locations from the back courtyard viewed from South-East towards North-West.

After the structural measurements were completed, for calibration purpose, another test was performed at the base level of the structure for each direction corresponding to symmetry axes of the rectangular structural geometry in plan.

To eliminate the effects of white noise, hardware problems and effects of the different cable lengths, baseline correction (linear and, if necessary nonlinear), decimation for eliminating high frequency spikes and filtering between frequencies of 0.2 Hz and the Nyquist frequency were performed. In order to compensate the differences in instrumental transfer functions, other calibration tests were performed for logarithmically increasing amplitudes of the swept-sine excitation up to frequency at which the seismometers yield almost constant amplitude under the lab conditions.

The measurements allowed identifying average peak frequencies, respectively at around 2.5, 3.5, 4.3, 5.3 Hz and so on for NW–SE direction. Similarly for the SW–NE direction, peaks are seen at around 2.6, 3.2, 4.5–5 Hz with a broadband characteristic with small differences in peak amplitudes.

The author of the identification in [14] proposed to fit the data with two autoregressive models (ARX) considering the system as a SISO or as a MIMO system. The SDOF model captures the important peak frequencies with sufficient accuracy, but magnitudes are arguably small. In MDOF model, responses at the crown stations 1, 2, 3, and 4 display almost the same peak magnitudes at the same frequency, i.e., 2.51 Hz for NE–SW direction, but station 3 (which possesses smaller peak amplitude at a smaller frequency, 2.2 Hz) is exceptional and indicative of an anomaly.

For both main directions, deformed shapes of the structure under both the earthquake vibrations and ambient vibrations also have been estimated by using the MIMO parametric model equivalent to the MDOF system. Mode shapes confirmed indicative deformed shape at station 3 for the first mode for the NE–SW direction.

The first mode of the structure, 2.4 and 2.5 Hz for NS and EW directions respectively, falls into the range of the dominant broadband period of the seismic loading (Kocaeli earthquake). This highlights the possibility of resonance which might have occurred during the earthquake. The duration of the earthquake and following shocks of many small earthquakes in addition to the resonance might be the key contributors to the damage.

4.2 The oval dome of the Sanctuary of Vicoforte

The Sanctuary of Vicoforte, a bold, highly prestigious structure, with its centuries-old history and the damages suffered in the past, is a classic example of a cultural asset exposed to earthquake hazard, being located in the proximity of a seismic area [15]. Research and monitoring activities have recently progressed on this building [16,17], including the realization of a non-linear model for the whole structure-foundation-soil system. Because of the technical complexity of the masonry structure of the building and the peculiarities of the soil on which it rises (presence of deep layers of soft soil, which caused appreciable settlements over the years and in the event of an earthquake would probably have significant effects on seismic input), the acquisition of knowledge and the modelling process pose difficulties to be addressed through an all-encompassing approach [16,17,18,19].

4.2.1 History of the building

The Sanctuary of Vicoforte, near Mondovi (Cuneo, Italy), is a building of great historical, architectural, and structural significance, owing its fame primarily to the great masonry elliptical dome (Figure 4.12), which is the biggest in the world of this shape in terms of overall dimensions (internal axes 37.23 by 24.89 m).

Figure 4.12 - The Sanctuary of Vicoforte (left). The interior of the dome (right).

Originally conceived by Duke Charles Emmanuel I of Savoy to serve as the mausoleum of the dynasty, the erection of the Sanctuary began in 1596 on a project of the famous architect Ascanio Vitozzi (1583–1615), but only in the eighteenth century the problem of the dome's construction was solved. In fact, in 1615 when Vittozzi died, the construction had reached only the level of the impost of the big arches at the base of the drum. During the seventeenth century, construction works were resumed and dragged on for several years, reaching about mid-height of the drum structure. The architect Francesco Gallo (1672–1750) designed a new drum-dome system different from the original one, and the dome was realized in 1731-1732 [20,21,22]. The original design drawings of the dome are stored in the archives of the Sanctuary, and are fundamental for understanding the construction of the dome because they contain annotations regarding the geometrical conception of the dome and of the temporary devices, scaffolding and centrings [23,24]. The architect decided to demolish the previous part of the drum, levelled its base because of excessive deformations, and erected a new slender drum with large window openings. The construction of the shallow baroque ribbed dome was started in 1731, when the monument had lost its original role as a royal family mausoleum, and was completed in less than one year. The Sanctuary was inaugurated in 1735 on the completion of the lantern top [20,25].

4.2.2 Geometry survey of the dome oval shape

In the archives of the Sanctuary are stored 25 drawings of the project of the scaffolding and of the dome. They were traditionally credited to Francesco Gallo, but according to recent studies [24] some of them should be credited to the great baroque architect Filippo Juvarra, who helped Francesco Gallo in 1728. Such original drawings provide clear information about the geometrical configuration of the dome only in the intersection of the dome itself with the three main orthogonal section plans (horizontal plan at the basis of the vault, longitudinal cross section and transversal cross section), but there is a lack of knowledge regarding the tridimensional shape of the entire dome. The modern 3D modelling techniques and software gave the opportunity to create a 3D surface on the basis of the information contained in the historical documents, and, for the parts of the surface that are not described in the documents, on the basis of geometrical hypotheses; on the other hand the modern 3D survey techniques provide information about the geometrical coordinates of points of the surface of the dome, and in particular of the points of the surface that are not described by the three main sections of the dome contained in the historical drawings. The combination of these two modern approaches gives the opportunity to make and verify hypotheses on the 3D shape of the dome by measuring the similarity to the real surface.

As stated in previous studies [23] the original drawings are not all in the same scale of representation, and even those that are apparently in the same scale show some imperfection, so each digital image was scaled in order to be dimensionally comparable to the other images. The dimensions of the two axis of the drum designed by Francesco Gallo should vary from 36.58 and 24.38 m to 36.99 and 24.66 m.

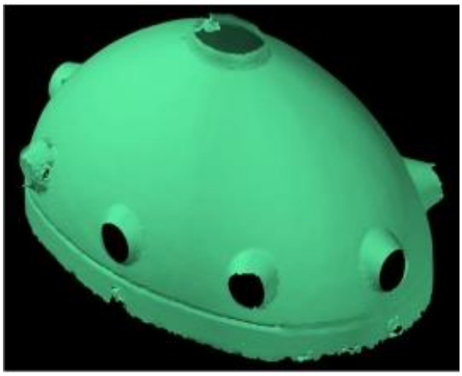
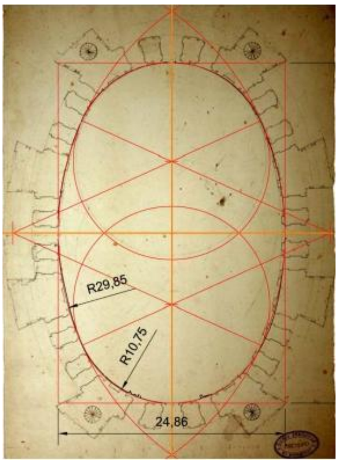


Figure 4.13 - Render view of the 3D mesh surface of the intrados of the dome.

Zander [23] gave a precise description of the main 8 drawings. Concisely, the dome internal surface should have an oval horizontal section both at its base and at its top. According to those documents, the surface between the base and the top should be divided in 80 meridians, and each of them should be an arc (a portion of a circle) with the centre on the springing plane and the radius decreasing from the transversal meridian to the longitudinal one.

The comparison of the original drawings with the mesh surface generated by laser scanner survey has been made by Novello and Piumatti [26], in order to find the intersection of the mesh surface with both horizontal planes (at the basis and at the top of the dome) and vertical planes (in correspondence of the longitudinal and the transversal axis and of the meridians). Before intersecting the mesh surface with the vertical planes of the meridians, it was necessary to identify such vertical planes. In fact the dome is not a revolution surface but has a complex shape, so the term “meridian” in quite inadequate. For the selection of significant vertical planes this study considers the constructive method typical of masonry domes. The traditional constructive method is based on the preliminary construction of temporary centrings, timber structures that has the profile of a slice of the intrados surface; this wooden temporary profile is used for the positioning of the voussoirs. The drawing number 67 of the Archive, credited to Francesco Gallo with the intervention of Filippo Juvarra [24] contains information about the construction of the scaffolding and centrings (see figure 4.14). The architects designed 20 centrings for each quarter of the dome, for a total of 80 centrings for the entire dome. The centrings are draft using a line that represents the plane that divides one cantering from the nearest one. As noticed from Zander [23], the lines do not converge in a single point, but more or less in an area around the intersection of the main axis. Novello and Piumatti [26] showed that the lines representing the centrings converge in different points, that correspond approximately to the centres of the circles that constitute the polycentric oval line at the top of the dome; in particular the lines from 1 to 12 converge rather precisely in the centre of the smaller circle, while the lines from 13 to 20 converge in the centre of the bigger circle. There is no evidence that the dome was built using exactly the 20 centrings designed by Francesco Gallo, but the drawings suggest the geometrical idea that supported the architect constructive project.



(a)

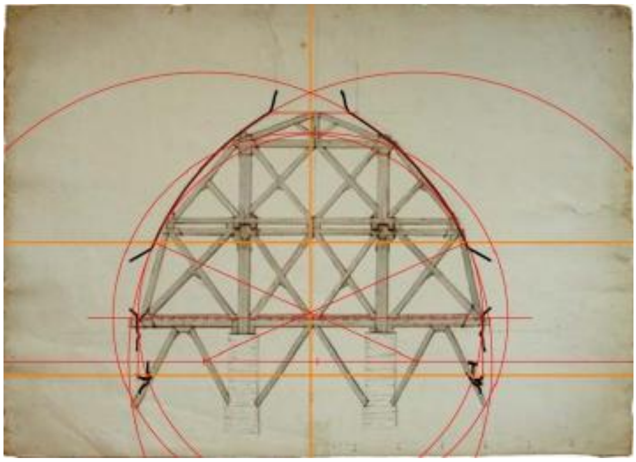

(b)

Figure 4.14 - (a): Comparison between an archival project drawing and the real polycentric oval; (b): Comparison between an archival project drawing of the transversal section and the polycentric curves obtained with a graphical interpolation of the laser scanner survey: it is clear that the original project was greatly modified during the construction, in fact the profile of the centring in the drawing is a single arch, while the built profile of the intrados is a polycentric curve. The drawing of Francesco Gallo with Filippo Juvarra shows the design of the scaffolding: a timber structure built on the basis of 6 columns and called "ponte reale".

Even if the need for a study specifically dedicated to the shape of the centrings has been recognized (Zander [23]), the following studies concentrated on the shape of the main sections, even if the geometrical data usable for such work were available. In 1976 in fact a photogrammetric survey took place. But it is symptomatic that it represented the intrados surface through isohypse lines. Isohypse are a good and common way to represent complex surfaces, but in this case they are not relevant because they don't follow the constructive method. In fact the horizontal sections, except the lower and the upper one, were not designed, and their shape is a consequence of the centrings shape. Novello and Piumatti concluded their study [26] stating that:

- the profile of the centrings used in the construction of the dome is very different from those described in the drawings conserved in the Sanctuary of Vicoforte; this signify that they are preliminary draft, and that the design of the dome was modified in the constructive phase;
- the curvature of the intrados doesn't start at the same springing plane for all the vertical sections, but is different for the various centrings;
- the centrings were built drawing two arches of different radius;
- the point of transition from the lower arch to the upper one is approximately similar in all the centrings, and is approximately at the same height of the second level of the scaffolding designed by Francesco Gallo and Filippo Juvarra. This may say (but further research is needed to confirm such hypothesis) that the profile of the intrados was built using two sets of centrings: the first set placed between the first and the second level of the scaffolding (the scaffolding was called "ponte reale" in the archive documents), and the second set of centrings placed on the second level of the scaffolding.

4.2.3 Structural monitoring and dynamic characterisation

The dome-drum system has suffered over the years from significant structural problems, partly due to further settlements of the building induced progressively by newly built masses, and, to a large extent, arising from the bold structural configuration of the dome-drum system itself.

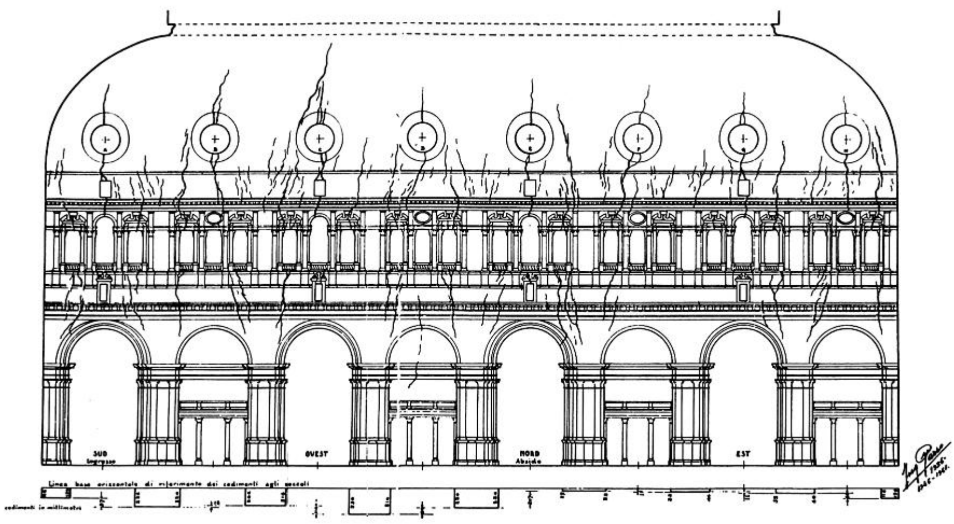


Figure 4.15 - Crack distribution and total settlements along the perimeter; maximum differential settlements (west versus northeast) approximately 300 mm [15].

In 1983, concerns over the severe settlement and cracking phenomena affecting the structure prompted the decision to undertake inspection, monitoring, and strengthening interventions (Figure 4.15). After a survey and investigation campaign designed to acquire detailed data on the conditions of the foundations, the geotechnical aspects of the site, the geometry of the dome and the monument as a whole, and the mechanical parameters of the masonry, a strengthening system was put in place (1985–1987). It consisted of 56 active tie-bars placed within holes drilled in the masonry at the top of the drum along 14 tangents around the perimeter, slightly tensioned by jacks (Figure 4.16).

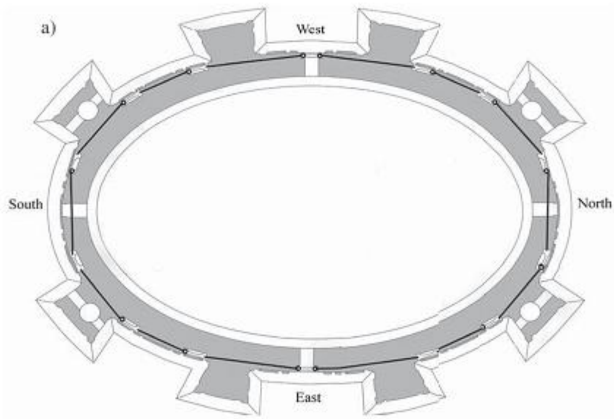


Figure 4.16 - Modern strengthening system.

A monitoring system was set up to measure strains and stresses in the structure and crack propagation, as well as stresses in the reinforcing tie-bars (re-tensioned in 1997).

In recent years, a thorough renovation of the monitoring system was carried out and a new research program was initiated in order to provide new advanced bases for the general plans for the preservation and the protection of the monument. The research program, still underway, aims, in particular, to improve the knowledge of the construction, both by completing an extensive series of diagnostic tests and by analysing the structure through models able to provide reliable interpretations of its behaviour and damage state. The final goal of this research is to define a model of the structure which, once integrated with monitoring results, might be able to describe the actual behaviour of the construction, predict its response to expected future loads (such as seismic actions), and optimize future strengthening interventions.

The dynamic studies conducted on the Sanctuary were designed first of all to assess seismic risk, while they also provided an opportunity to test out the application of the new regulations. In fact, the Sanctuary of Vicoforte has been recently chosen as a case study for the evaluation and application of the Directive PCM 2008 [27], in the frame of an agreement protocol with the Italian Ministry for Cultural Heritage. Although Vicoforte is characterized by low seismicity, the seismic vulnerability of the cathedral deserves to be investigated owing to its historical, architectural and structural significance. This attention is also justified by the typical weakness of the dome-drum systems against earthquakes. On the basis of a seismo-tectonic study, three main faults are interested:

Monferrato, Western Alps and Western Liguria faults. They are characterized by the seismological parameters listed in Table 4.1. Deterministic ground shaking scenarios on bedrock have been recently calculated, and a parametric study has been performed to identify the most critical rupture mechanisms for the ground response at Vicoforte [28].

Seismogenic Source	M_w	**Latitude, Longitude**	**Depth [Km]**	**Strike, Dip**	**Type**	**L [Km]**	**W [Km]**
Western Liguria	6.3 ↔ 6.7	43.74° N 8.13° E	6 ↔ 12	60°,60°	Reverse	10 ↔ 20	8 ↔ 12
Western Alps	5.7 ↔ 6.0	44.84° N 7.26° E	3 ↔ 8	60°,45°	Normal	8	4
Monferrato	5.1 ↔ 5.6	44.82° N 8.42° E	8 ↔ 15	50°,80°	Strike - slip	6	3

Table 4.1 - Parameters of the seismic source. L and W indicate length and width of faults (Lai *et al*[19]).

The Western Liguria event generates the largest PGA and PGV values of 0.06 g and 0.065 m/s respectively. Similar values of maximum PGV of ~0.02 m/s and PGA of ~0.02 g have been calculated for both the Western Alps and Monferrato events (Lai *et al*, 2009).

The Directive describes the procedures to be followed in assessing the safety of an existing building. In this connection, it is essential to acquire a thorough knowledge of the structural conditions of a building. In particular, when dealing with a historic building, this must be done in several steps, including: (a) Identification of the structure; (b) Geometric data gathering; (c) Historical analysis; (d) Survey of the materials and their state of preservation; (e) Mechanical characterization of the materials; (f) Soil and foundation analysis; (g) Monitoring. Needless to say, the acquisition of knowledge meets with the difficulties normally associated with the assessment of existing structures that have to be preserved. Earlier studies conducted on the Sanctuary of Vicoforte included investigations aimed to characterise the masonry and the foundation soil, and to determine the geometric data and crack patterns (for further information on the construction of the Sanctuary and the investigations conducted previously, see Chiorino *et al*, [16]. The next step along these lines was the execution of non-destructive dynamic tests, designed to characterise the dynamic behaviour of the structure and the mechanical properties of the materials, and to identify the overall and local response of the structure.

Also due to the dimensions of the building, the experimental campaign focused on the dome, while in a seismic assessment the main interest is for global modes, especially those falling in a low frequency range.

4.2.3.1 Dynamic characterisation

The characterization of dynamic behaviour is an essential part of structural monitoring and damage control, especially in the case of monumental masonry buildings. Most structures of artistic-historic interest are the fruit of construction processes that took decades to complete and used heterogeneous materials and different building techniques. Furthermore, these structures having undergone extensive changes over the centuries, each monument has unique peculiarities of its own. The impossibility of resorting to generalised criteria makes the study of such structures extremely complex and an even more critical aspect is that the reliability of the results cannot always be ascertained. It is no chance that the new seismic standards introduce and encourage explicitly the attainment of an appropriate "level of knowledge" which can be obtained solely through extensive investigations. Though they cannot be directly correlated with a specific safety level, dynamic investigations contribute to the calibration of valid models for use at the reliability assessment stage. Moreover, no other study can shed light on the global behaviour of a building. Hence, they are indispensable to the characterization of complex structures, and especially to the formulation of accurate forecasts as to their dynamic and seismic behaviour. Due to the uncertainties and difficulties encountered in defining a modelling method for generalised application to masonry structures, the results of numerical analyses – performed as a rule with finite element methods (FEM) – must be supported by experimental confirmation.

4.2.3.2 Testing campaign

The data acquisition campaign at the Sanctuary of Vicoforte was performed by Eucentre, Pavia, in June 2008 [29]. The tests were executed by means of 4 Lennartz 3D/5s triaxial geophones with a sensitivity of 400 V/m/s and 5 PCB 393B31 ICP piezoelectric accelerometers with a sensitivity of 10 V/g. The signals of the transducers have been digitalized through a multiplexer SCXI1140 and an A/D PCI National Instruments converter with a resolution of 16 bit. Different sampling frequencies were used in the acquisitions.

Figure 4.17 - Examples of accelerometers and geophones placement: (a) on the top of the dome and (b) on a column.

The instruments were positioned according to different acquisition setups to arrive at a global identification of the structure (Figure 4.18). In the various setups, a few instruments were placed at fixed nodes that remained the same throughout the series of acquisitions, so that at a later stage it would be possible to assemble the results and work out an overall description of the modal shapes. In particular, the elliptical dome was tested with different setups, by arranging the instruments both along the axes and along the directions diagonal to the axes. The linking points used for assembling the experimental modes of the dome were channels 0, 1 and 2 (as indicated in Figure 4.18a-b). Wind velocity was measured by means of an anemometer fitted to the structure and wind direction was determined by means of a transducer.

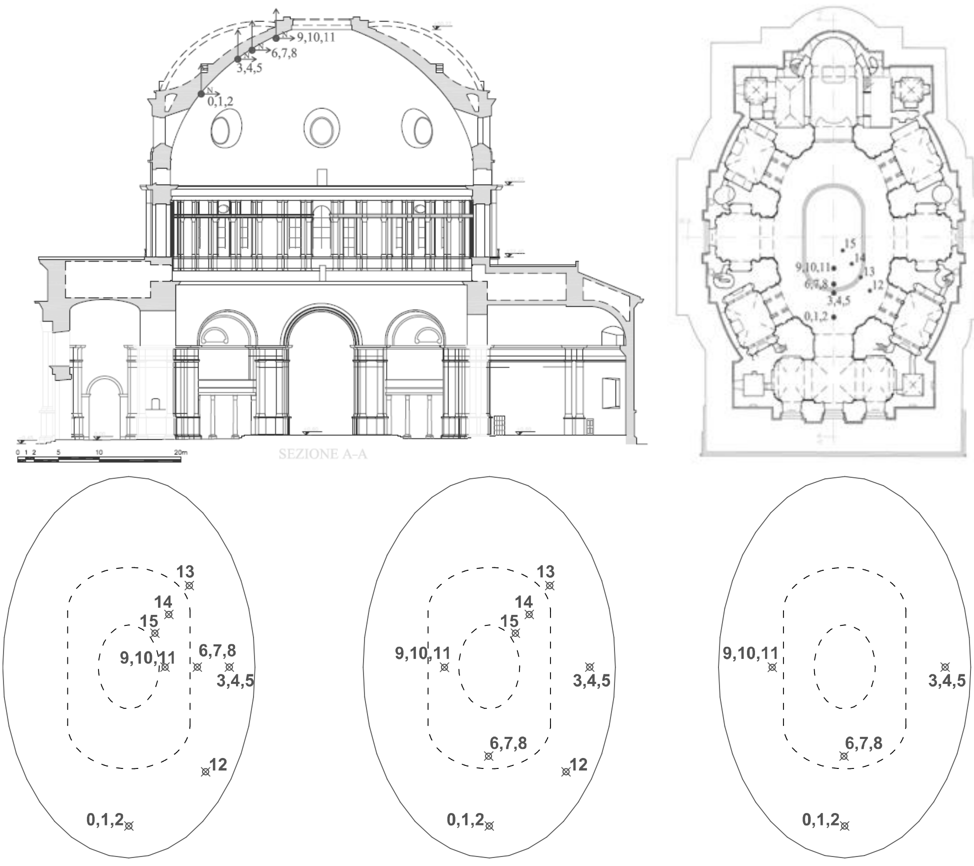


Figure 4.18 - Arrangement of sensors in the various setups: (a) setup 1, (b) setups from 2 to 4, (c) setups 5 and 6. Fixed nodes for linking shapes were in 0,1,2 (a triaxial geophone).

Eucentre conducted a series of tests with wind and vehicle traffic serving as sources of excitation, and for column identification bells were also used. Hereby are shown the setups used for the identification procedure: setups 1 to 4 were focused on the dome, whilst setups 5 and 6 were centred on the columns (Figure 4.18). Relevant acquisitions are listed in table 4.2, together with type of excitation and associated setups. Part of the tests were sampled at 512 Hz, then, after a pre-analysis, the sampling frequency for ambient vibration acquisitions was reduced to 128 Hz. Ambient vibration signals, which had different time lengths, were truncated to allow signal segmentation and statistical analysis.

Acquisition	**Setup**	**Excitation**	**Notes**
1A	1	Ambient vibration	Dome
1B	2	Ambient vibration	Dome
1C	3	Ambient vibration	Dome
1G	3	Ambient vibration	Dome
1H	3	Ambient vibration	Dome
1I	3	Ambient vibration	Dome
1L	3	Ambient vibration	Dome
1M	3	Ambient vibration	Dome
1N	4	Ambient vibration	Dome
2A	5	Ambient vibration	Column A
2B	5	Ambient vibration	Column A
2C	5	Ambient vibration	Column A
2F	6	Ambient vibration	Column B
2G	6	Ambient vibration	Column B
2H	6	Bells	Column B

Table 4.2 - List of the acquisitions used in the application.

4.2.3.3 Output-only identification of the Sanctuary of Vicoforte

The first step in the structural identification of the Sanctuary was a preliminary analysis of the signals acquired by Eucentre, conducted by means of SSI (output-only) methods, in consideration of the use of unknown ambient excitation tests. The code used to perform the identification is the SDIT code developed at the Politecnico di Torino [30], that runs in the MatLAB environment.

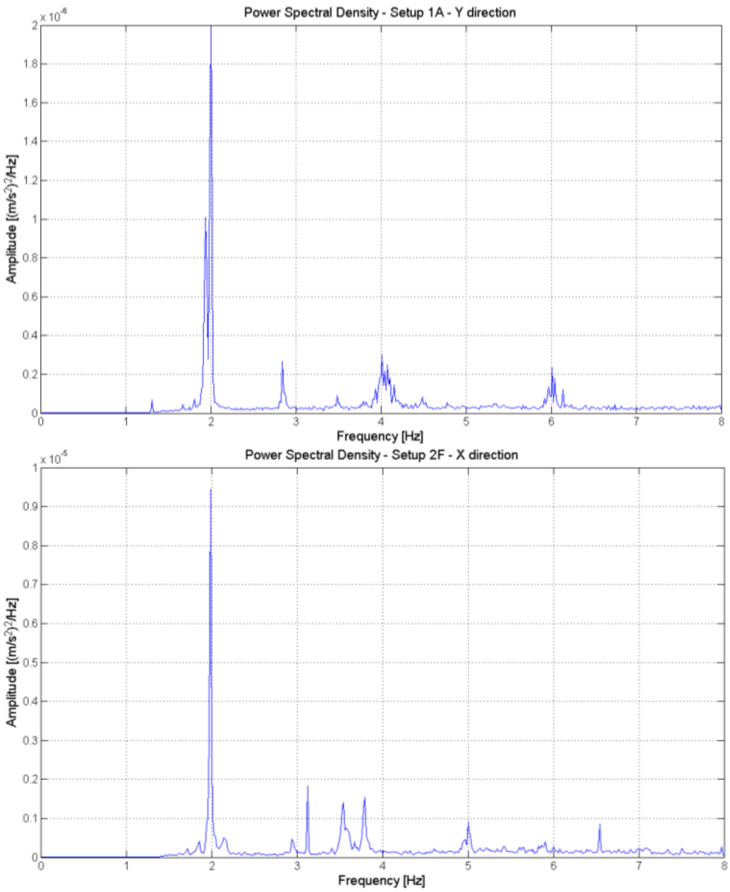


Figure 4.19 - Examples of average Welch power spectral density diagrams: (a) on the X direction channels of acquisition 1A and (b) on the Y direction channels of acquisition 2F. Length of Hamming window: 1/8 of signal, overlap: 50%, frequency resolution: 16 Hz.

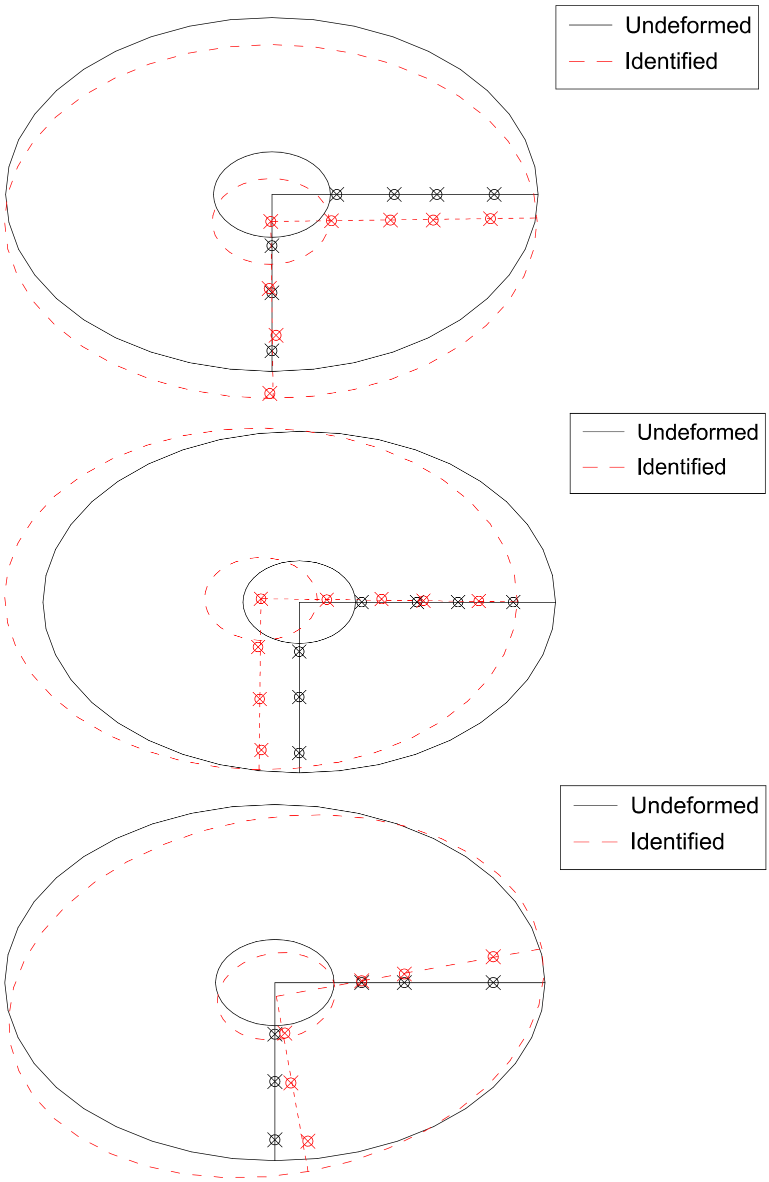


Figure 4.20 - Identified modal shape of: (a) the 1st bending mode along y direction at 1.99 Hz, (b) the 1st bending mode along x direction at 2.08 Hz, (c) the 1st torsional mode of the dome at 3.42 Hz. Each mode is normalized to its maximum value.

The signals acquired were conditioned using filters, de-trending and sub-sampling. After a pre-analysis (e.g. Figure 4.19) with Welch power spectral representations [31], several time-domain identification sessions were executed. Computational modes were systematically discarded by using modal assurance criteria (MAC). In more detail, all signals coming from different acquisitions were segmented and a great number of SSI identification sessions were performed. Stabilization diagrams were used to identify frequencies in each segment (stabilization criterion: maximum frequency deviation: 2%), then additional tolerance criteria were used for MAC (5%) and for damping ($0 \leq \zeta \leq 20\%$).
By executing a statistical recurrence of the system's natural frequencies identified by the SSI algorithm and by averaging values, it proved possible to distinguish between the real modes of the structure and modes that appeared occasionally, being possibly due to exogenous components. In this manner, three modes were identified with certainty (Figure 4.20) and seven more were rated as "suspect". Other modes were also observed which were not computational modes, though some doubts still exist as to the type of modal shape involved. All modes extracted are complex valued, though they are plotted by forcing 0 or π phase difference between channels. Modes are possibly affected by frictional phenomena and non-linearity. In particular, tests of linearity would be desirable for a more accurate dynamic characterization of this monument.

Identified Frequency [Hz]	**Type**
1.99	1st bending, Y
2.08	1st bending, X
3.42	1st torsional

Table 4.3 - Experimental frequencies identified in the preliminary analysis.

4.2.3.4 FEM model construction

The next step was an identification process assisted by a FEM model. Mode attribution was, in fact, based on a comparison between the modal shapes extracted from the signals and those obtained from the solution of a FEM eigenvalue problem. By means of the finite element code it is possible to work out a classification of the modes, which otherwise would be virtually impossible when dealing with highly complex structures (such as the Sanctuary of Vicoforte). In standard FEM codes, a given dynamic math model is reduced to one with fewer degrees through ad hoc reduction techniques. Degrees of freedom can be selected as a function of the points where the acquisitions have been performed so as to obtain comparable modal shapes. Similarly, in a further stage, FEM models may support the expansion of the identified modal shapes. After that, a FEM model of the structure was constructed using the ANSYS computation code, with tetrahedrical solid elements and lumped mass matrices. As for the parameters of the materials and the constitutive law, the initial values were selected in accordance with Directive PCM [27] for historic masonry, as representative of the historic masonry of the building Table 4.4.

Elastic modulus, E:	1635 MPa
Poisson's ratio, v:	0.4

Table 4.4 - Initial parameters of the FEM model.

Geometric data were obtained from an examination of the building performed with a laser scanner by the Nagoya City University research team coordinated by T. Aoki, and from surveying measurements (Figure 4.21). The cracks formed in the structure were not taken into account in constructing the FEM model, based on the assumption that their effects on the order of presentation of the natural modes of the structure were virtually negligible. It is expected, however, that a FEM model designed to assimilate damage and non-linearity will be produced in future [32].

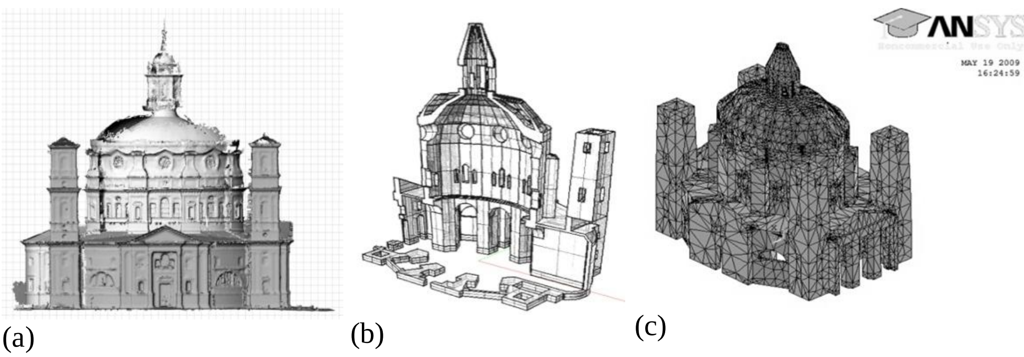
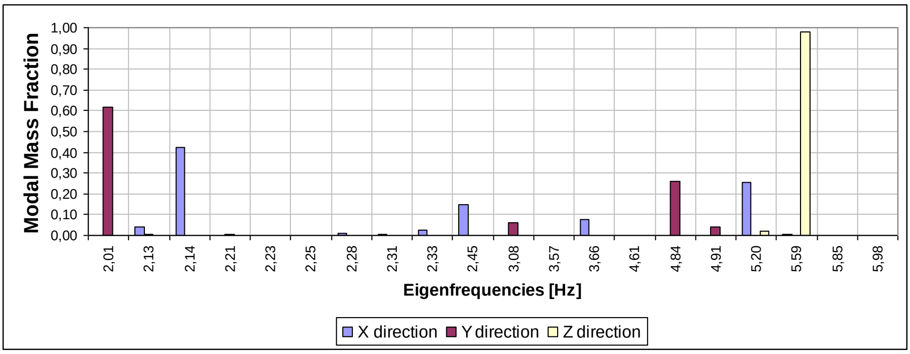
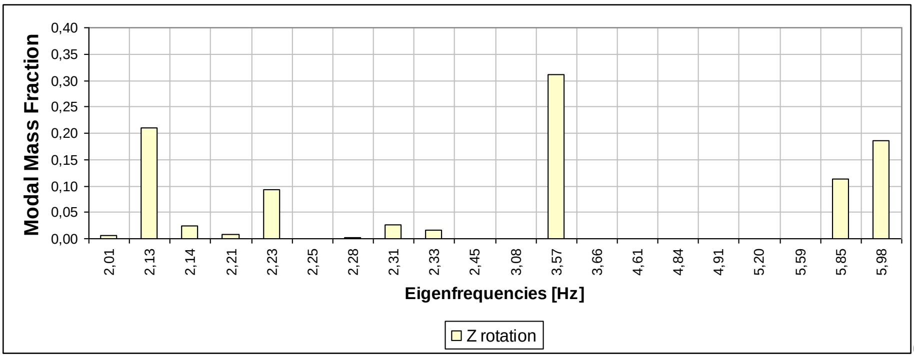


Figure 4.21 - (a) Cluster of points obtained with a laser scanner [33], (b) Axonometric split view of the geometric model, (c) FEM model.

The structure was reasonably assumed to be clamped at the base. Furthermore, in this linear FEM model, the continuum underlying the structure was disregarded, based on the assumption that in ambient vibration conditions soil-structure interaction has no effects on vibration modes. The criterion used to classify modes was the percentage of mass participation along horizontal (X, Y, Z) directions and torsion round Z axis. This made it possible to determine which vibration modes of the structure should be compared with the results of the identification process.



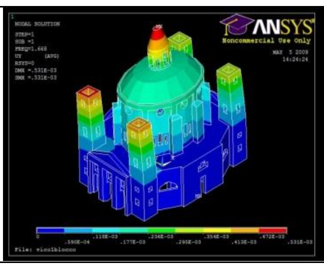
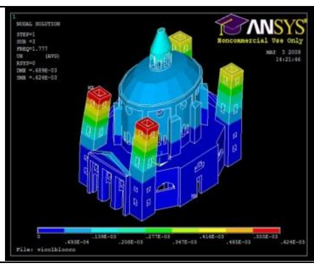
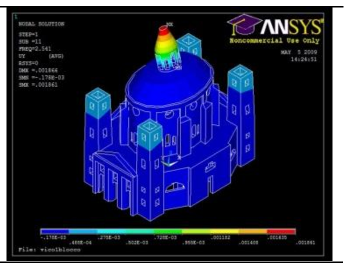
(a)



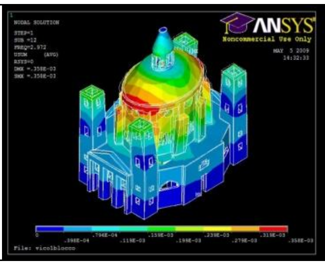
(b)

Figure 4.22 - Modal mass participation in the FEM vibration modes (a) directions X , Y e Z, (b) torsional.

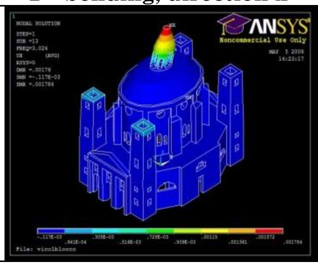
The global modes extracted from the model are listed in figure 4.23. The selection was made based on modal shapes and on the charts shown in figure 4.22(a) and (b) which illustrates the modal mass fraction with respect to translations and torsion, respectively: the modes with a high participation are rated as global modes, and those with a low participation are rated as local modes.

Mode 1 (1.99 Hz) **1st bending, direction y**	**Mode 3 (2.12 Hz)** **1st bending, direction x**	**Mode 11 (3.01 Hz)** **2nd bending, direction y**
		

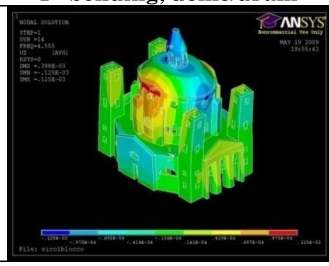
Mode 12 (3.55 Hz)
1st torsional



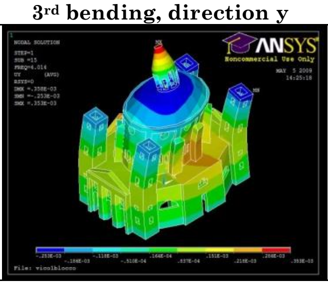
Mode 13 (3.59 Hz)
2nd bending, direction x



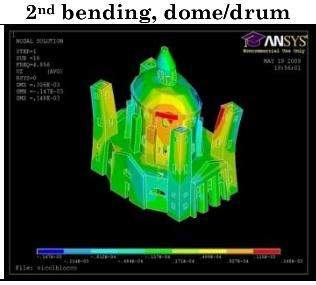
Mode 14 (4.56 Hz)
1st bending, dome/drum



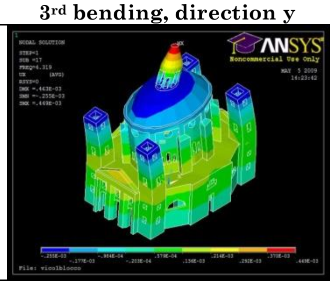
Mode 15 (4.86 Hz)
3rd bending, direction y



Mode 16 (4.87 Hz)
2nd bending, dome/drum



Mode 17 (5.14 Hz)
3rd bending, direction y



Mode 18 (5.55 Hz)
1st vertical, dome

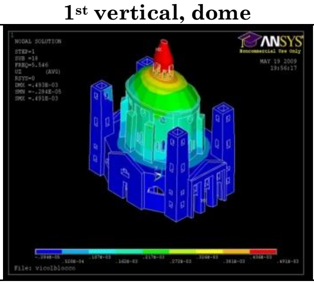


Figure 4.23 - Modes of the FEM model of the structure.

4.2.3.5 Model updating, damping and frequency identification

Through an optimisation of Young's modulus and Poisson's ratio of the FEM model, a first updated model was obtained. The model updating was guided by the minimization of a simple cost function of modal frequencies [34]:

$$J(\boldsymbol{\theta}) = \sum_{i=1}^{n} \left[\lambda_{exp,i} - \lambda_{FE,i}(\boldsymbol{\theta}) \right]^2 \tag{4.1}$$

where $\boldsymbol{\theta}$ is the vector of the parameters to be optimized, n is the number of modes considered in the minimization, $\lambda_{exp,\,i}$ is the i-th experimental eigenvalue, and $\lambda_{FE,\,i}$ the i-th eigenvalue supplied by the FEM model.

By considering only two frequencies in the cost function (1st bending direction Y, 1st bending direction X), the updated parameters of the model turned out to be as follows:

Property:	First attempt value:	Updated value:	Test value (2004):
Elastic modulus, E:	1635 MPa	2330 MPa	1300 - 4800 MPa
Poisson's ratio, ν:	0.4	0.38	0.39

Table 4.5 - Updated parameters of the FEM model and comparison with the experimental data determined by Aoki *et al* [35].

It is also possible to compare updated parameters with experimental data obtained by previous tests. In 2004, non-destructive tests were performed to evaluate the compressive strength and Young's modulus of mortar and bricks by means of a scratch tester (scratch width) and a Windsor Pin System (penetration resistance). These tests were flanked with compressive tests performed on small brick and mortar cylinders (Ø 33 mm × 50 mm) according to Japanese Standards JIS A 1108. Good correlations were found between the outcomes of two non-destructive tests and between the results of the latter and those of the compressive tests. In particular, penetration resistance and compressive strength were seen to decrease with increasing scratch width. Conversely, with increasing penetration resistance, compressive strength was seen to increase. The relationship between compressive strength and Young's modulus was fairly linear [35].

Table 4.5 shows a substantial consistency of the FE updating process, though the same elastic parameters were used for the whole structure. A more realistic FE model calibration would require the use of different types of materials, so allowing the optimization procedure to correct different mechanical parameters. To this aim, in the future, the model will be divided in regions with homogeneous mechanical characteristics, also based on historical data (i.e. the four bell towers were completed in 1884).

By means of this model it proved possible to identify the remaining natural frequencies from the experimental data. Due to high interaction between modes, identification was seen to be easier if done by frequency bands. Therefore the signals were filtered beforehand by means of band-pass filters to free the signals of the frequency components that did not have to be identified at the moment. Then the results of the identification procedure were plotted in frequency vs. damping diagrams, so that the natural frequencies would form clusters of points and could be readily identified for modal uncoupling, if needed. Identified modes corresponding to the first two bending modes are clearly identified by the two clusters in figure 4.24, where also a frequency at 2.86 Hz is discernible.

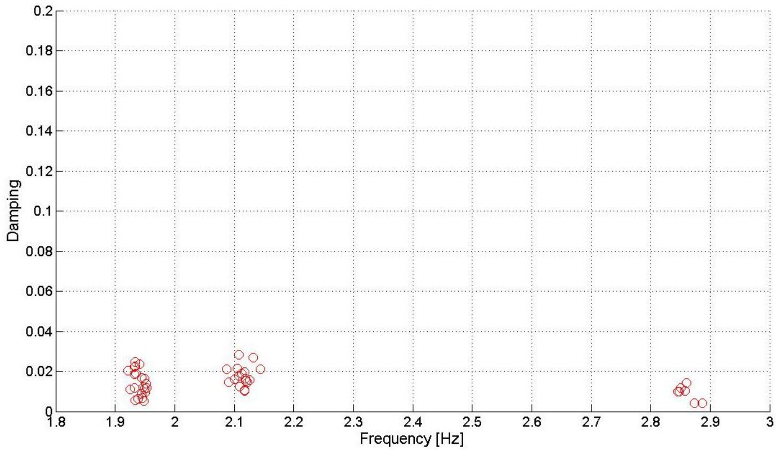


Figure 4.24 - Frequency vs. relative damping in signals of acquisition 1B: values extracted from different signal segments.

Frequency at 2.86 Hz might be associated to a mode that is regulated by the four bell towers Figure 11). Figure 4.25 depicts the identified shape for this particular mode. In the dynamic tests performed by Eucentre the towers were not instrumented, but modes of the bell-towers appear with strong energy on bell-excitation measurements.

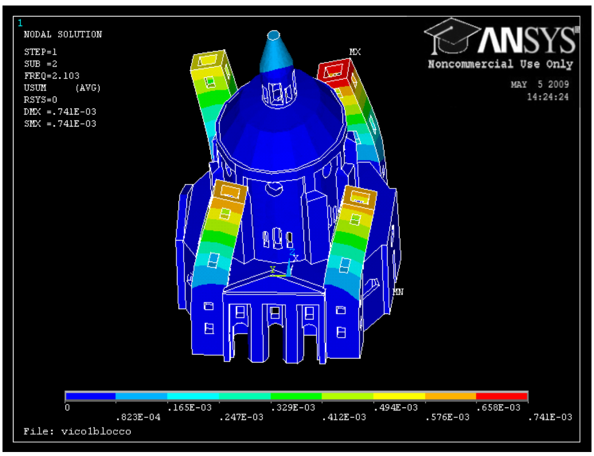


Figure 4.25 - Mode 2 of the FE model as regulated by the bell towers.

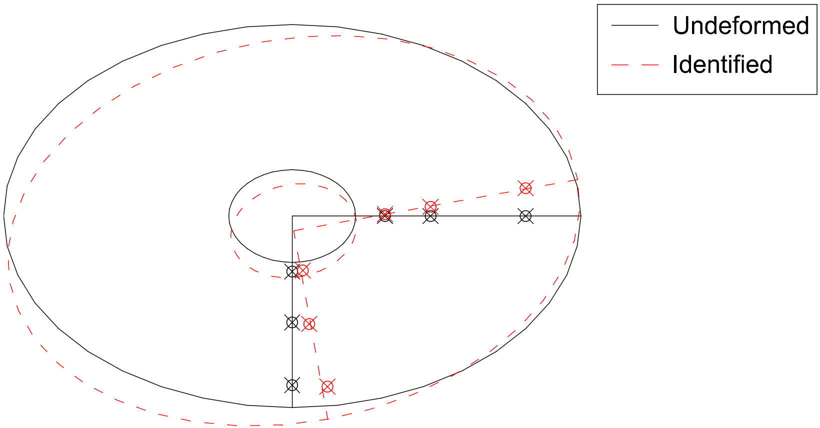


Figure 4.26 - Experimental modal shape of the mode identified at 2.86 Hz.

Then, higher frequencies, between 3 and 3.5 Hz, were analysed, by filtering the signal again, which made it possible to identify the second bending mode in direction Y, in addition to the torsional mode of the dome. These modes are particularly intense in the setup 6, as is apparent in the time-frequency representation of figure 4.27, which refers to acquisition 2F (channel 1, reference to figure 4.18c). Figure 4.28, which refers to acquisition 1B, reports a stabilization diagram referred to an acquisition where a strong modal component appears at 6.1 Hz, which was seen to correspond to the first vertical mode of the dome (Figure 4.29).

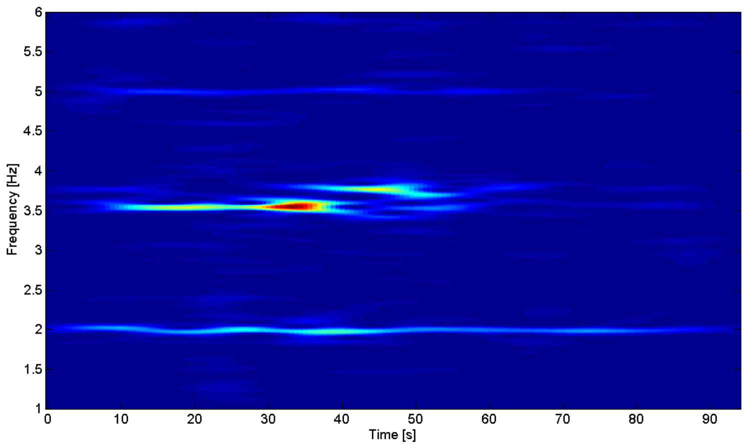


Figure 4.27 - Spectrogram of a sub-sampled signal from acquisition 2F (channel 1, reference at Figure 4.18c). Length of Hanning window: 320 samples, FFT length: 2048, overlap: 80 %.

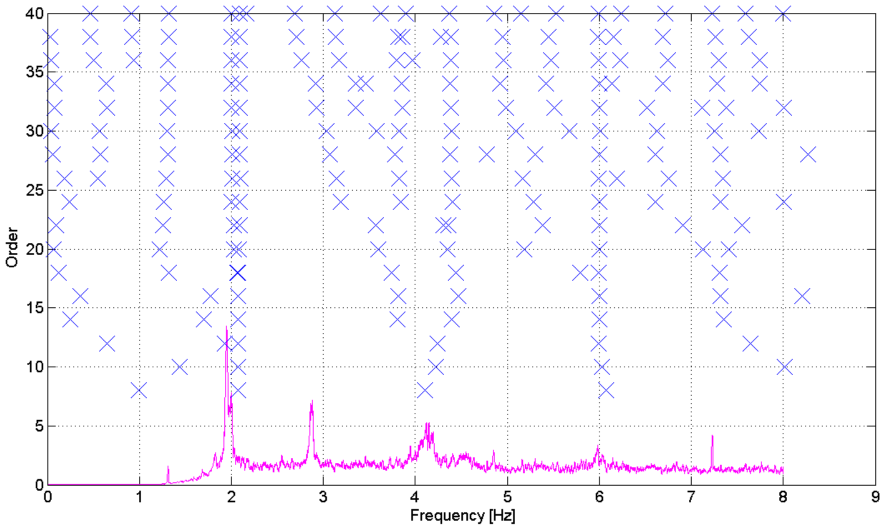


Figure 4.28 - Sample of stabilization diagram associated to acquisition 1B (reference at Figure 4.18b). The stabilization criteria are: 2% for frequencies, 5% for damping and 5% for vectors.

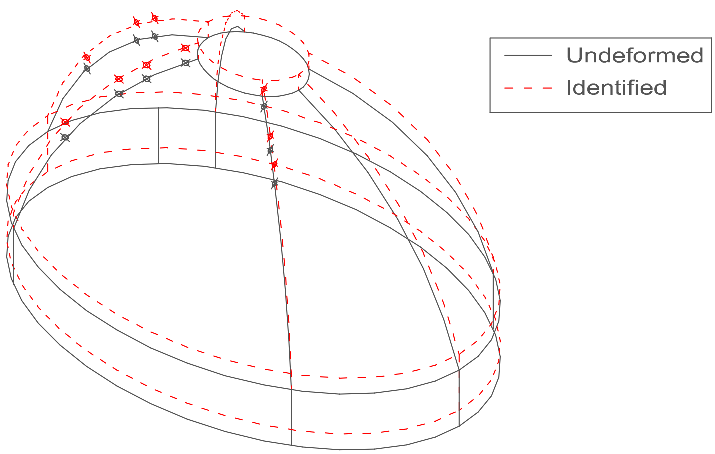


Figure 4.29 - Experimental modal shape of the vertical dome mode at 6.02 Hz, normalized to its maximum value. The signals used are from acquisitions 1A and 1C.

The frequencies identified at the end of the FEM model assisted identification procedure described above are as shown in table 4.6.

Updated FE model frequency [Hz]	Experimental frequency [Hz]	Type of mode:
1.99	1.99	1st bending, Y
2.12	2.08	1st bending, X
3.01	3.08	2nd bending, Y
3.55	3.42	1st torsional
3.59	3.77	2nd bending, X
4.56	4.11 (*)	1st dome/drum
4.86	5.16	3rd bending, Y
4.87	4.36 (*)	2nd dome/drum
5.14	5.96	3rd bending, X
5.55	6.02	vertical dome

Table 4.6 - Modes identified using the FEM model assisted procedure. (*) Suspect mode.

4.3 The oval dome of the church of Santa Caterina in Casale Monferrato

The origin of the church is linked to a donation of the marquis Anna d'Alençon to the Dominican nuns of S. Caterna da Siena, occurred in 1528. The religious relocated in the palace and built an oratory (the current choir). During the eighteenth century the nuns wanted to renew the church, so they commissioned Giovanni Battista Scapitta of the new project. The new Church was built in 8 years, and it was finished in the 1726, and it was placed side by side to the old oratory [36].

The body of the church is mainly characterised by an oval dome, built on a drum with a height of 7 meters. The dome dimension are 15 m by 10 m, the remaining part of the building are a little atrium and the apse (communicating with the oratory). The dome height is 4.5 m and it presents 8 buttresses which subdivide the dome in 8 masonry groins. The interior of the dome is richly frescoed even if the deterioration of the frescoes is in progress.

The dome presents a slender lantern 5 m tall which recalls the structure of the dome; in fact it has 8 pilasters, one for each buttress. The roof of the lantern is again a little masonry dome. The richly decorated Baroque façade has a jut which exceeds the height of the church body of 6 m.

The old choir has dimensions in plant of 10 m by 22 m and it is covered by a barrel vault, reinforced through the use of regularly distanced round arches in correspondence of the pilasters and by using metal chains.

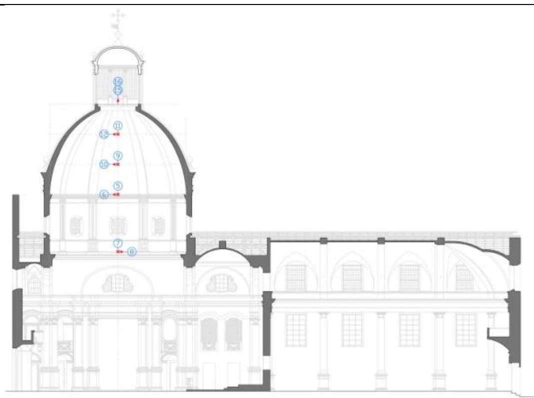
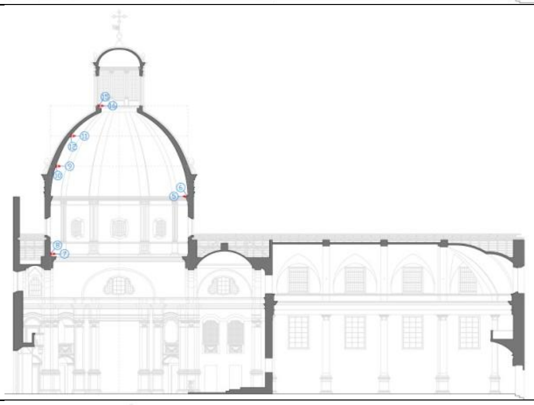
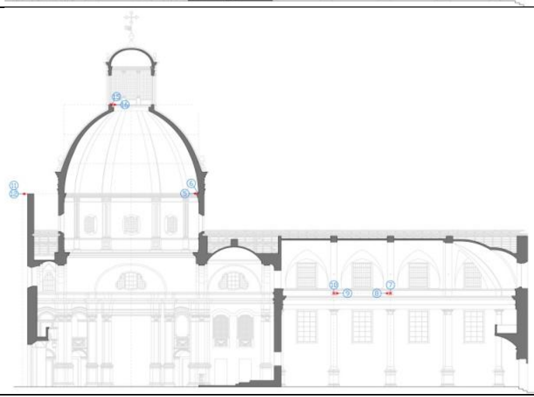
Figure 4.30 – Santa Caterina in Casale Monferrato.

4.3.1 Testing campaign

A series of tests were carried out on the church of Santa Caterina between the 23rd and the 29th of September 2010. Among these tests a dynamic characterisation of the building was scheduled. In this case the dynamic setups were designed using a linear FE model realised in Ansys. The modal analysis allowed determining that the most flexible zones of the buildings were the lantern and the jut of the façade. Therefore these zones have to be particularly monitored in order to easily distinguish among the lower frequencies of the building by using the modal shapes. Moreover, using the modal assurance criteria (MAC) it has been verified that the number of positions acquired were enough in order to distinguish among modes. In fact it may happens that different modes, if measured only in certain locations may have high MAC values. The threshold, in this type of analysis has been set on the MAC index to 0.8.

The façade was easily accessible; therefore it has been monitored in its higher part. On the other hand the lantern has proved to be practically impossible to access; therefore the accelerometers were located at the connection between the dome and the lantern.

The 4 setups (Table 4.2) were composed by a total of 18 acquisition channels and they were divided a link setup, which comprises a link setup, with 8 positions linking all the setups and other 3 different setups used to characterise the various zones of the building, in particular the dome, the choir and the jut of the façade.

Setup	**Excitation**	**Notes**	
Dome YZ	Ambient vibration	The Y-Z plane of the dome is monitored	
Dome XZ	Ambient vibration	The X-Z plane of the dome is monitored	
Global	Ambient vibration	The X-Z plane of the dome is monitored, together with the choir	

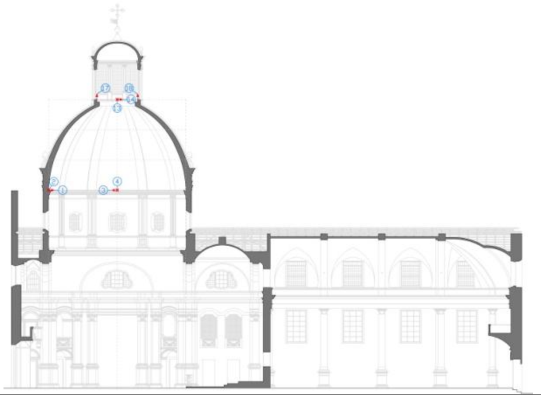
Link	Ambient vibration	Linking setup	

Table 4.7 - List of the setups and excitations used in the application.

Figure 4.31 - (a) The lantern (b) The jut of the façade and the instrumented locations.

The sensors used in the acquisitions were PCB Piezoeletronics with a sensitivity of 1 V/g, measuring range between 0 and 3g and a resolution of 30 µg. The accelerometers were connected using coaxial cables and a data acquisition system LMS Difa-Scadas connected to a notebook. Whenever a triaxial acquisition was required a metallic cube was placed and the accelerometers fixed by using screws and wax.

The data were acquired with a sampling frequency of 400 Hz and with an average length of 1200 s.

Figure 4.32 - Metallic cube used to locate the accelerometers in triaxial position.

4.3.2 Dynamic identification

The first step in the structural identification of the church of Santa Caterina was a preliminary analysis of the signals acquired, conducted by means of SSI (output-only) methods [37,38], in consideration of the use of unknown ambient excitation tests. Also in this case, the code used to perform the identification is the SDIT code developed at the Politecnico di Torino [30], which runs in the MatLAB environment.

In order to improve the robustness of the identification the signals where subdivided in shorter signals and a statistical analysis of the identified frequency was performed. The signals were treated with subsampling, filters and de-trend algorithms. The parameters used to select real modes from spurious modes were frequency, damping and modal shapes. In the latter case the previously defined modal assurance criteria (MAC) was used to discriminate among modes. Frequency versus damping plots allowed distinguishing clearly among modes, as can be notice from figure 4.33.

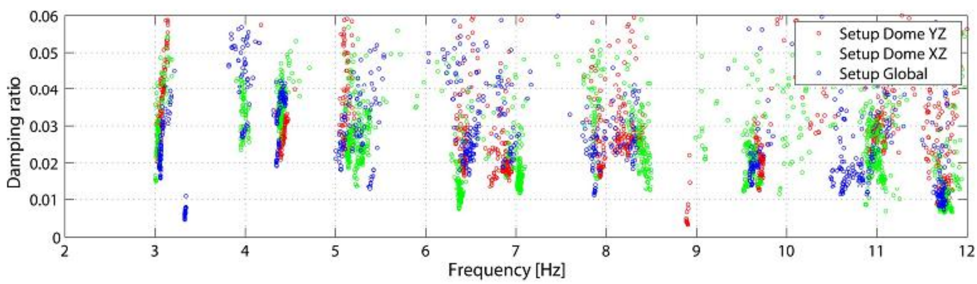


Figure 4.33 - Clustering of the results in the frequency range between 2 and 12 Hz.

The identification has been conducted with a model-driven approach. In fact, after the first preliminary identification, the obtained modes were compared with the modes obtained by a FEM model realised in Ansys. The model has been realised using shell and beam elements, using a linear isotropic material with an elastic modulus of 2500 MPa, a density of 2000 kg/m^3 and a Poisson ratio of 0.4. Four main factors that may strongly influence the dynamic behaviour of the finite element model have been identified:

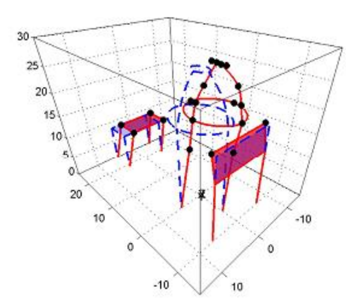
- The lantern;
- The jut of the façade;

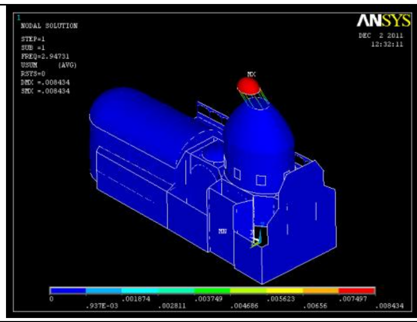
- The oval dome;
- The adjacent structures (the choir and the palace of the trade union).

For what concerns the latter point, two different schematics have been considered: rigid adjacent bodies (elastic modulus of 5000 MPa) or flexible adjacent bodies (elastic modulus of 1000 MPa). The comparison of these results highlighted that the difference of the two behaviours is negligible: the discrepancy between natural frequencies is comprised between 0.01 and 0.06 Hz.

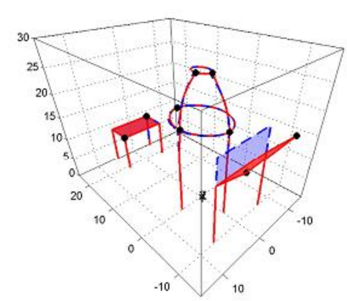
The toolbox SDIT allows for a graphical representation of the modal shapes, in order to compare the identified modal shapes with the modal shapes of the FEM model. The first four modes identified match the modes determined analytically

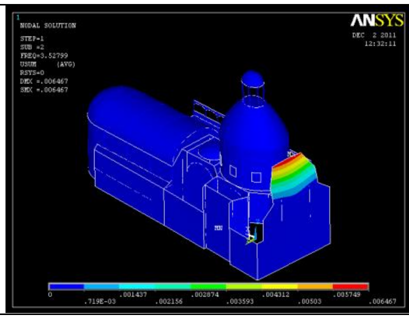
Mode 1: 3.03 Hz



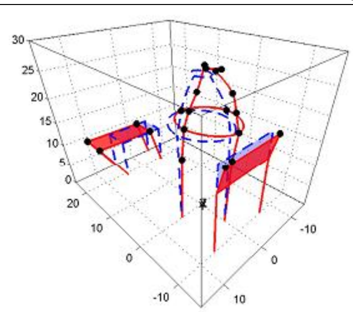


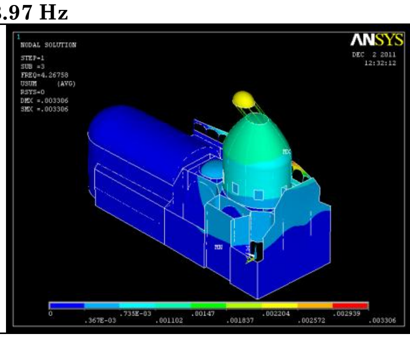
Mode 2: 3.33 Hz



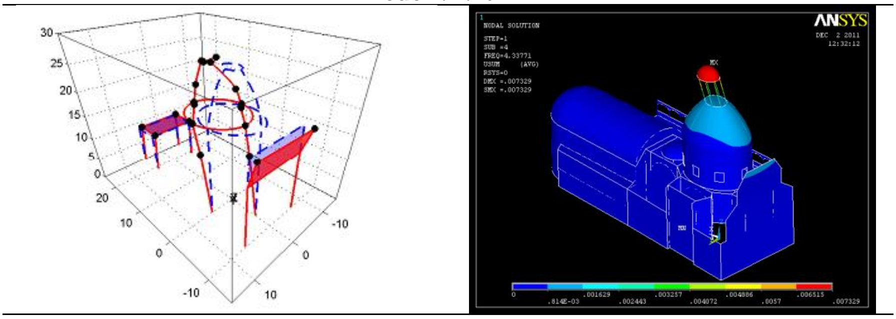


Mode 3: 3.97 Hz

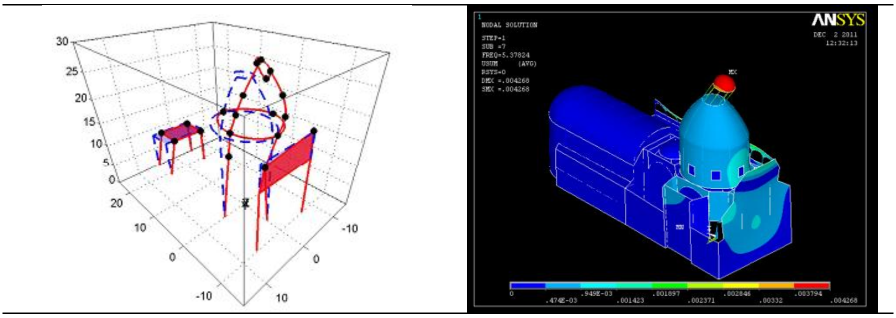




Mode 4: 4.40 Hz



Mode 5: 5.11 Hz



Mode 6: 5.39 Hz

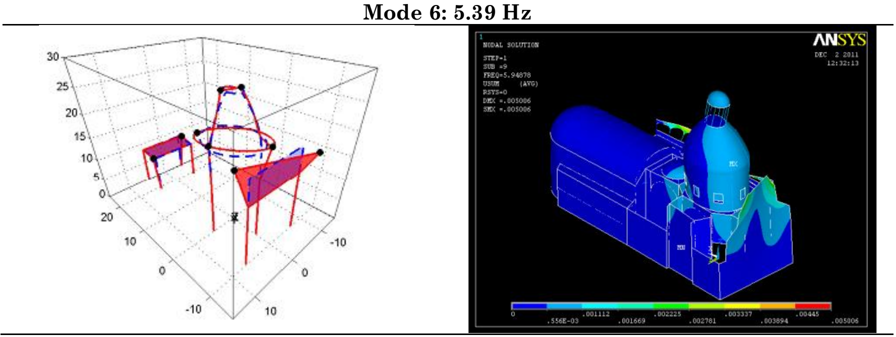


Figure 4.34 - Comparison between the first six identified modal shapes and the linked modes determined with the FEM model.

4.3.3 Model updating

Before starting the modal updating procedure the data gathered during a visual inspection of the building were used to define four different zones of the model: the lantern, the façade, the basement and the drum-dome system. These zones had 4 different linear isotropic materials assigned to be updated during the procedure. The parameters to be updated were the elastic modulus, the Poisson's ratio and the density of the material. In this case the optimisation function has been defined as follows:

$$\upsilon = p_f \cdot \left(\frac{f_s - f_i}{f_i} \right)^2 + p_m \cdot \left(1 - MAC\left(\phi_s, \phi_i\right)\right)^2 \qquad (4.2)$$

where p_f and p_m are weighting factor referred to frequencies and modal shapes, f_s and f_i are, respectively, the identified and analytical frequencies and MAC is the modal assurance criteria between identified and analytical frequencies.

Elastic moduli, Poisson's coefficients and densities were optimised iteratively in three different moments. This was done in order reduce the number of simulations to be performed, reducing the range of variation of the parameters each time. The values of the updated parameters are listed in table 4.8. It can be seen that particularly weak zones are the lantern and the façade as it was expected after visual inspections. On the other hand, the basement and the drum-dome system shows a very high elastic modulus, indicating a good quality of the materials in structurally relevant zones of the building, as it has usually been found in analogous structures.

Material	Elastic Modulus [MPa]	Density [kg/m3]	Poisson's ratio
Basement	3625	1733	0.40
Drum-dome	4500	1733	0.40
Lantern	1025	1733	0.30
Façade	1250	1733	0.40

Table 4.8 - Values of the updated parameters.

After the updating procedure the discrepancy in terms of frequency of the model is decreased significantly, and table 4.9 shows the final values.

Mode	FEM Frequency [Hz]	Identified Frequency [Hz]	Error [%]
1	2.95	3.03	2.67
2	3.54	3.33	6.16
3	4.30	3.98	8.04
4	4.30	4.40	2.25

Table 4.9 - Values of the updated frequencies.

The updated model allowed noticing that the first mode of the structure does not depend on the interaction with adjacent buildings but is referred to the dome and the choir which oscillate synchronously, instead the façade is out of phase. Mode 2 is a local mode related to the jut of the façade, which is completely free to oscillate with respect to the rest of the building. Mode 3 is still governed by the same elements: dome and choir, but in this case they oscillate with out of phase motion. The model updating has also shown the presence of ovalisation modes at relatively low frequencies (Mode 6).

4.4 The oval dome of the church of Sant'Agostino in L'Aquila

4.4.1 *Technical history of the building*

The eighteenth-century church of Sant'Agostino, was rebuilt after the 1703 earthquake destroyed the pre-existent medieval church. The church is located in the city centre of L'Aquila, contiguous to the more known Palazzo del Governo (almost completely destroyed by the recent earthquake of April 2009). A series of tests conducted on the foundations showed that the structure is completely independent from pre-existent buildings. The most recent intervention on the church was the repairing of the lantern roof: the old cover was substituted with a new metallic one, linked to the original stone columns using a concrete ring [39,40].

The church is composed by three parts: the atrium, the body and the presbytery, closed by a circular apse. The body is surmounted by an impressive oval dome. It is worth notice that the dome can be considered built directly on the piers, without a drum. Nevertheless the oval dome presents an octagonal dome cladding (*tiburio*) which has not structural role. In correspondence of the vertex of the octagonal cladding, there are eight buttresses, linked directly to the dome through masonry walls in correspondence of the internal pilaster. The dome is therefore subdivided by these walls in eight groins. The dome has a wooden roof supported by the buttress walls.

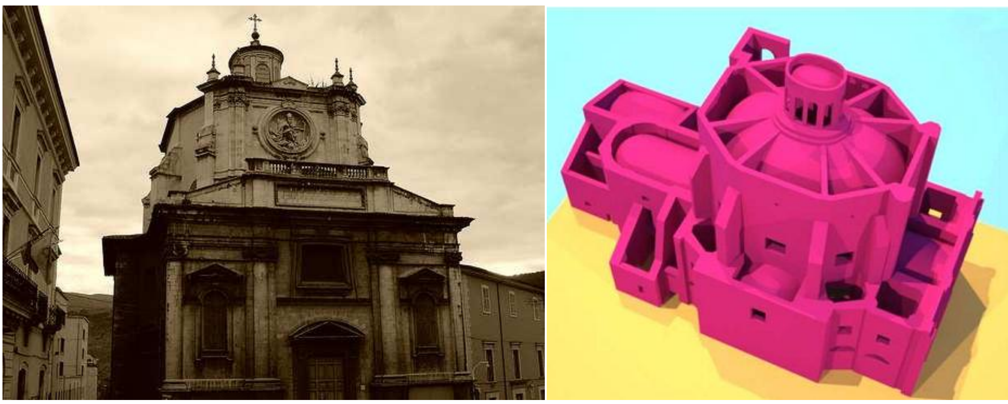


Figure 4.35 - Sant'Agostino church: (a) baroque façade, (b) 3D geometric model.

4.4.2 *Damages caused by the 2009 earthquake*

The earthquake of April 2009 has severely damaged the church. In accordance with the Directive PCM a survey of the damage has been conducted by Prof. Lagomarsino and his team [41]. The whole structure suffered heavy damage by the earthquake but, in particular, the lantern was completely destroyed by the earthquake (see figure 4.36a). The roofing of the lantern was rebuilt during the 80s using steel trusses and an edge beam in concrete to connect it to the masonry structure.

The dome-*tiburio* system has been also heavily damaged and showed in plane shear failure (diagonal-cracking) in each groin of the dome and on the walls of the octagonal *tiburio*.

The bell-tower, which has a peculiar L-shape, was heavily damaged (see figure 4.36a).

Other damage mechanisms were also activated: the façade shows the initiation of a tilting mechanism and several cracks are present in the apse, the presbytery and in the lateral chapels. The annexed bodies were heavily damaged too, such as in the case of the sacristy vault and its walls.

Figure 4.36 - Damage due to the 2009 earthquake: (a) Lantern completely destroyed, the damage on the tiburio and on the bell tower, (b) Damage on the dome's groins, (c) detachment between the main body and the dome, (d) Vault of the sacristy.

4.4.3 Dynamic analysis of the church

A finite element model of the church has been realised using the software Ansys®. The model has been built by using solid elements (see figure 4.35) and the material mechanical properties defined in table 4.10. These material characteristics have been chosen in accordance with the Italian technical codes in absence of in-situ tests.

E [MPa]	**P [kg/m3]**	**ν**
1000	1900	0.25

Table 4.10 - Mechanical parameters used for masonry

It has already been mentioned that the Sant'Agostino church has an adjacent building (*Palazzo del Governo*) which interacts with the dynamic behaviour of the church. There are several walls of the *Palazzo del Governo* which are orthogonal to the main body of the church that can absorb the horizontal forces (see figure 4.37 where the walls with a thickness greater than 50 cm are highlighted).

Therefore it has been chosen to model these walls as boundary conditions and assume the behaviour of the walls as rigid in their plane.

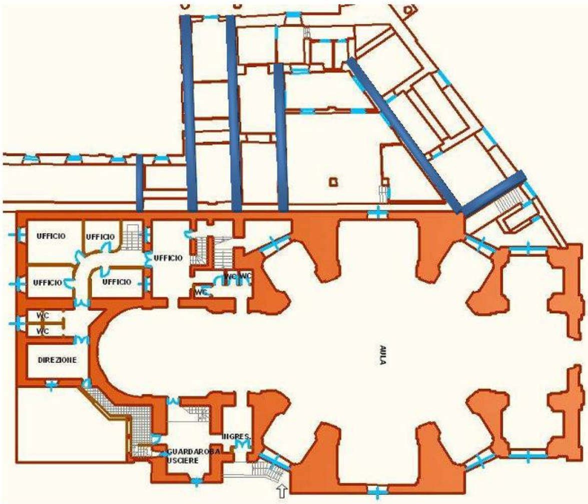


Figure 4.37 - Plan of the church and seismic resistant walls of adjacent building (in blue).

In first place, a modal analysis of the model has been carried out. The first four modes, which have the higher mass participation fraction, are listed in figure 4.38. All the other modes have a mass participation fraction lower than the 4% and are negligible in a first analysis.

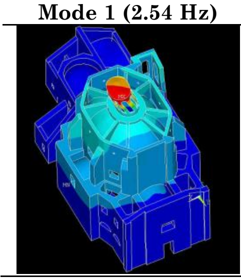
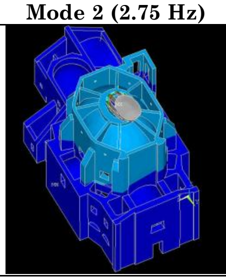
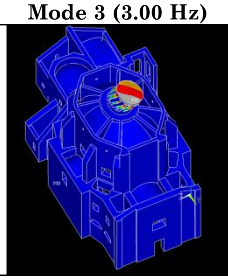
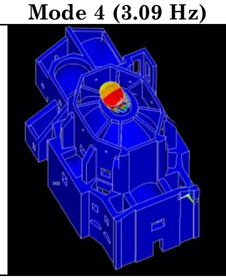
Mode 1 (2.54 Hz)	**Mode 2 (2.75 Hz)**	**Mode 3 (3.00 Hz)**	**Mode 4 (3.09 Hz)**
			

Figure 4.38 - The first four modal shapes.

Another relevant mode is the mode of the bell tower, which has been heavily damaged by the earthquake. As it can be noticed in Figure 4.39, the mode at 5.18 Hz is a local mode related to the bell tower. This particular mode has a relatively high participation factor and therefore must be taken in account.

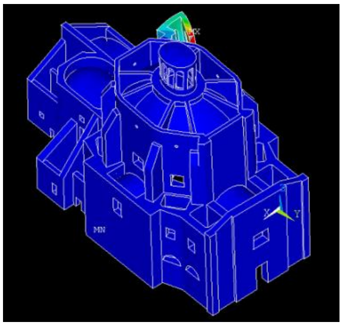
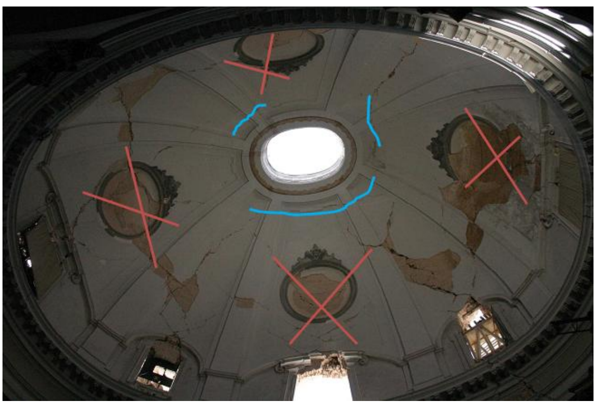


Figure 4.39 - Mode 11 (5.18 Hz) of the bell tower.

A time-history analysis of the model has been performed, using the acceleration record of the L'Aquila earthquake (of the closer seismograph to the church) and applying it to the model accordingly to the spatial orientation of the church. This type of analysis allowed determining the different mechanisms which caused damage to the church.

The most relevant damage is in correspondence of the groins of the dome, where the typical x-shaped cracks occurred in proximity of the openings (red lines in figure 4.40a). The first hypothesis of the cause of the appearance of these cracks was a torsional effect induced by asymmetric boundary conditions (due to adjacent buildings). Instead, the time-history analysis did not show any signs of torsional effects induced on the dome. Actually, the cracks were due mainly to the presence of the opening at the top of the dome (in correspondence of the lantern) and of the buttresses that link the *tiburio* with the dome (see figure 4.35).



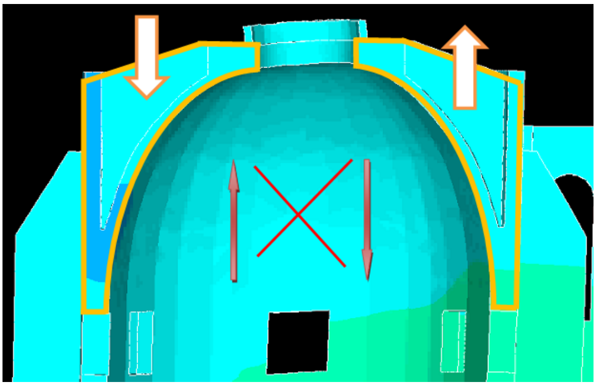


Figure 4.40 - Damage mechanisms of the groins of the dome due to shear deformation [42].

In fact, as it is shown by figure 4.40(b) the sides of the dome moves in opposite directions, inducing in the weak zones (namely near the openings) a shear stress that caused the failure of the groins in their plane.

On the other hand, the cracks shown in figure 4.40(a), highlighted in blue, near the opening at the top of the dome are related to another mechanism. These cracks are related to the oscillations of the lantern, which is particularly flexible with respect to the rest of the building. Figure 4.41a shows that the deformations induced by the seism to the lantern were significant. The lantern was particularly slender and with significant openings. This resulted in the complete collapse of the lantern itself, as it is shown in figure 4.41b.

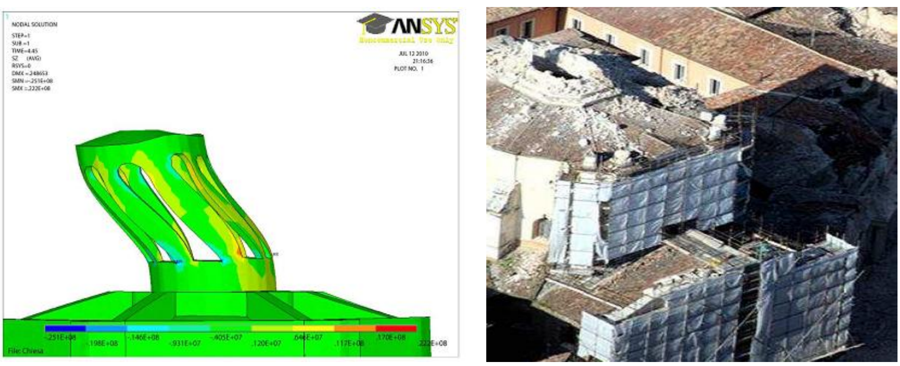


Figure 4.41 - (a) Deformation of the lantern under seismic excitation, (b) collapse of the lantern [42].

Last, but not least, the bell tower has been investigated. As it has been said, the L-shape of the tower, connected to the main body of the church only on one of its sides, does not behave well if subject to horizontal loadings. This was counterchecked in the finite element model, has it can be shown by figure 4.42, where a part of the bell tower is substantially in overhang.

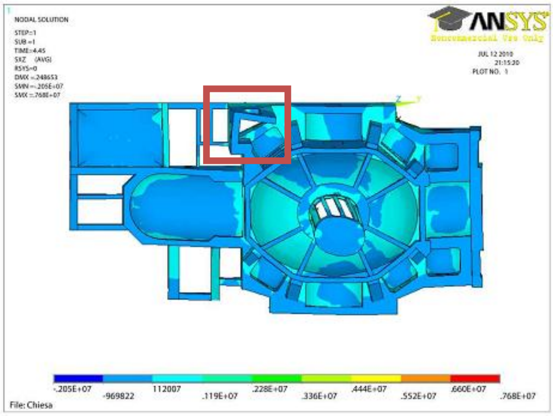


Figure 4.42 - Time history analysis, notice the deformation of the bell tower (emphasised in red).

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