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SOIL-STRUCTURE MODELING AND UPDATING OF THE "REGINA MONTE REGALIS" BASILICA AT VICOFORTE, ITALY

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Abstract. *The paper describes one of the most important structural health monitoring program in Italy, which started in 1983 for the conservation of the "Regina Montis Regalis" Basilica of Vicoforte. The Basilica is a building of great historical, architectural, and structural significance, owing its fame primarily to its big masonry elliptical dome, the world's largest of this shape. The dome-drum system has suffered over the years from significant structural problems, partly due to soil settlements, and, largely, arising from its bold structural configuration. The program is developed in coordination with national conservation institutions, considering Vicoforte Basilica as a notable case study for the application and assessment of the guidelines recently formulated at national and international level for the protection of cultural heritage in seismic regions. Currently, the program addresses the evaluation and reduction of the seismic risk of the monument.*

This paper reports the progresses achieved in the last years of monitoring and analysis of the structure. Past efforts (long term monitoring, non-linear FE masonry modelling and seismic input characterisation) were disseminated also in the previous SAHC editions (2006 and 2012). This time the paper describes the progress in the characterisation of the numerical model developed for the simulation of seismic response of the structure, by taking in account soil-structure interaction and model updating on the basis of experimental modal analysis.

1 INTRODUCTION

Forecasting the response to dynamic and especially seismic events, for safety assessment as well as for control purposes, may benefit from vibration measurements, which are indispensable to update a numerical model. This paper reports and updates the results of a long-term monitoring and research project conducted in order to evaluate the state of conservation and the structural safety of the Sanctuary of Vicoforte, near Mondovì, Italy. This is a monument of great architectural significance, characterised by the largest elliptical dome in the world (axes 37.23 m x 24.89 m). Conceived in 1596 by Duke Carlo Emanuele I to serve as the mausoleum of the Savoy dynasty, since the earliest stages of its construction the monument was affected by the settlements of foundations resulting in the opening of extensive cracks. In 1987 its dome-drum system was strengthened for fear of an imminent collapse.

An extensive experimental campaign has been conducted on the Sanctuary since 2001 by the Politecnico di Torino [1,2], the University of Genoa [1], the University of Pavia [3] and the Nagoya City University [4]. The impressive amount of data collected in these years is all aimed to establish new advanced bases for the conservation and protection of the monument. The main scopes of the program are the improvement of the knowledge of the construction, the interpretation of the cracking phenomena through an advanced modelling in the static domain on the basis of non linear damage models, the estimation of the structural safety under gravity loads on the basis of limit analysis, and the updating of numerical models through the comparative examination of the structural response in the dynamic domain. Finally, the program addresses the evaluation of the seismic risk with reference to the recent guidelines [5,6] formulated at the national and international level for the evaluation and the reduction of the seismic risk of the cultural heritage.

In this particular paper, the dynamic numerical modelling of the structure is expanded with respect to previously presented works [2], taking into account the complex soil-structure interaction present in the building [7,8,9]. Therefore, a much more refined version of the numerical model is presented, which takes into account both for different materials within the structure and the soil underneath the building. Moreover, the model is calibrated on the basis of experimental modal analysis carried out by using ambient vibration dynamic measurements.

2 THE BASILICA "REGINA MONTE REGALIS"

2.1 Description of the structure

The Basilica "Regina Montis Regalis" is located in Vicoforte, near Mondovì, in Northern Italy, and is a building of great historical, architectural, and structural significance, owing its fame primarily to its big masonry elliptical dome (Figure 1). With its internal axes of 37.23 m and 24.89 m, respectively, the dome is one of the largest in the world, and by far the largest elliptical one. The construction of the basilica began in 1596, based on a project by architect Ascanio Vittozzi (1583–1615). Since the earliest stages of construction, important absolute and differential settlements of the foundations affected the building, due to an unfortunate selection of the site. 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. In the early eighteenth century, works continued with the architect Francesco Gallo (1672–1750), who demolished the previous part of the drum, levelled its base because of excessive deformations, and erected a new slender drum with large window openings. The Basilica was inaugurated in 1735, on the completion of the lantern top [10].

Figure 1 - External view of the Basilica (left-side) and the interior of the dome (right-side).

Eight huge masonry pillars supports the elliptical drum-dome system. In the lower portion of the pillars, the radial walls separating the chapels increase their thickness. A slender and transparent drum connects the dome to the underlying structures. Eight ribs reinforce the extrados of the dome, and these ribs transfers their load to eight rather slender buttresses (two of them with stairwells inside). A huge and heavy sandstone lantern tops the dome. Eight oval windows are located, between the buttresses, in the region of the dome and tiburio. The space between the tiburio and the vault is filled, up to a variable level, with light-weight materials, while full masonry sections at the buttress locations ensure structural continuity. The top of the tiburio is connected to the dome by a reverse vault (see Fig. 6). Two sets of relieving arches where built in the thickness of the dome and the drum, respectively above the oval openings in the dome and at an intermediate height between the triple windows of the drum and the dome impost. The openings of the oval windows are built by means of thick oval masonry rings, while the two rectangular lateral openings of the triple windows are surmounted by stone architraves. The dome is equipped with three sets of original annular iron ties totalling a cross-sectional area of 14000 mm^2, positioned immediately above and below the oval windows and at the top of the tiburio.

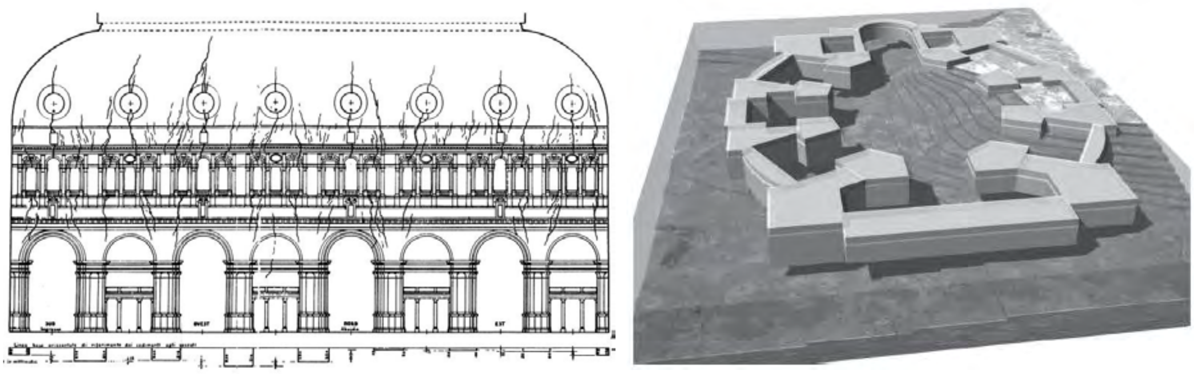

Figure 2: Cracking pattern on the dome (left side) and axonometric view of the foundations (right side).

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 (Figure 2), and largely arising from the bold and shallow structural configuration of the dome-drum system itself [11]. In 1983, concerns over the severe settlement and cracking phenomena affecting the structure prompted the decision to undertake inspection, monitoring, and strengthening interventions. 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 steel tie-bars, totalling a cross-sectional area of 3200 mm^2, placed within holes drilled in the masonry at the top of the drum along 14 tangents around the perimeter. The bars were slightly tensioned at the moment of their placing. A monitoring system was set up to measure strains and stresses in the structure, crack propagation, as well as stresses in the reinforcing bars. The bars were re-tensioned in 1997 to compensate for stress losses.

2.2 Structural monitoring and dynamic characterisation

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. Earlier studies conducted on the basilica 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 basilica and the investigations previously conducted, see Chiorino et al. [1]). 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.

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 building were designed first to assess seismic risk, but they also provided an opportunity to test out the application of the new Italian regulations. In fact, the Sanctuary of Vicoforte has been recently chosen as a case study for the evaluation and application of the Directive PCM 2011 [5], in the frame of an agreement protocol with the Italian Ministry for Cultural Heritage, the administration of the Sanctuary, the Politecnico di Torino, the University of Genoa and the University of Pavia.

The first steps of this investigation and research process, which are illustrated in the following, regarded the definition of the seismic input at the site and the dynamic characterisation of the complex soil-structure interaction. Work is still in progress in what concerns the use of these results in the subsequent phases of evaluation and, the case being, mitigation of the seismic risk of the monument.

3 GEOTECHNICAL INVESTIGATIONS AND DEFINITION OF SEISMIC INPUT

A wider attention has been given in recent years to the conservation of cultural heritage in seismic areas. Preserving or upgrading the seismic response of monuments is a challenging task, due to the complex phenomena involved, especially in case of large masonry architectures, and to the need to conserve as much as possible the original material and structural features. Guidance documents have been developed for countries characterised by an exceptional cultural heritage and by significant seismicity.

Italian directive PCM [5] underscores the importance of an assessment and mitigation of seismic risk based on good structural knowledge of cultural heritage. This need was well evidenced by the tragic consequences, in terms of severe losses of architectural masterpieces, of earthquakes that in recent years hit Umbria-Marche (1997) Abruzzo (2009) and Emilia (2012) regions. In the last two events, like in other previous seismic events along the history in Italy, the high vulnerability of dome-drum systems was demonstrated by the collapse or great damage of quite a few of them. A vulnerability which is intrinsically associated to the configuration of these architectural objects, standing isolated above the underlying built masses.

3.1 Geotechnical investigations and foundation soil

Geotechnical investigations were conducted in the past years to characterize the foundation soil [3]. The campaigns, carried out in 1976, 2004 and 2008, revealed that the soil underlying the sanctuary is composed of three different materials having thickness variable point-to-point. A layer of marl slanted in southwest direction (dip angle 5°- 6°) surmounted by a more recent formation of a fine-grained deposit of fluvial origin, with some old manmade filling in the upper part. This formation, mainly consisting of a clayey-silt layer with a low sand content, in lent form at some points, is approximately 1 to 5 m thick on the west side of the monument, and virtually disappears on the east side. Therefore, the east side of the monument is mostly founded directly on the bedrock, whereas the foundation structures of the southwest zone of the monument, especially those of the pillar supporting the dome-drum system, rest on a silt-clay layer reaching a maximum thickness of 3 m. The subsoil characterization showed that the bedrock formation has on average a very high strength and a very low deformability. Therefore, from the geotechnical point of view, it can be considered as a rigid support for the upper formation. The latter is slightly over-consolidated mostly due to seasonal groundwater oscillations, desiccation, etc.

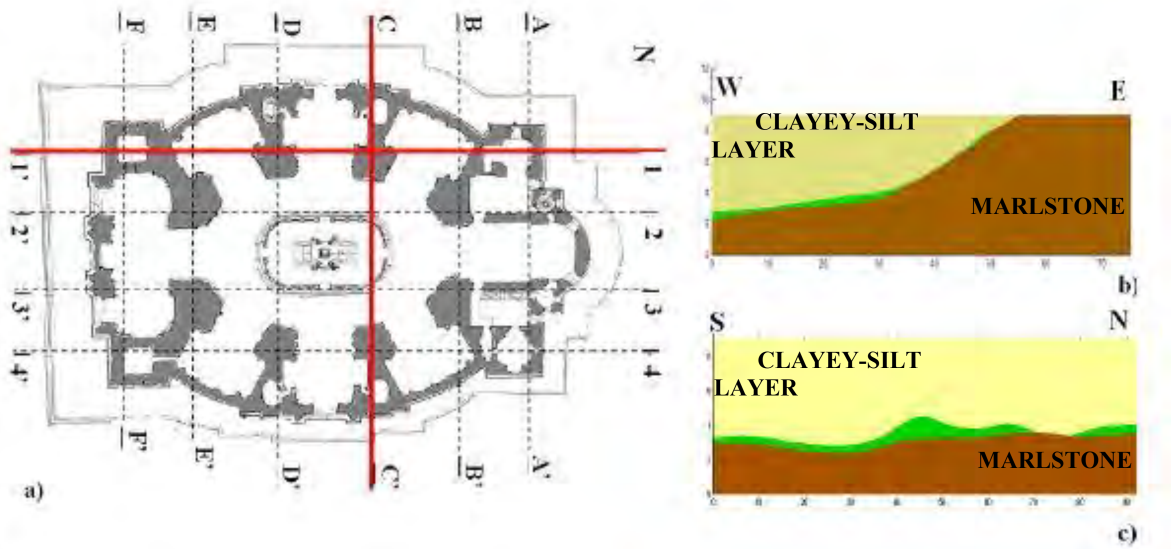

Figure 3: (a) position of the generated vertical cross-sections; (b) section C-C’ West-East; (c) section 1-1’ South-North [3].

The Sanctuary foundation is constituted by the four bell tower and the eight pillars foundations. Their depth below the ground level, as revealed the drilling and the axonometric view of the foundation carried on 1974, changes from about 2 m to 4 m [3].

The focus of the cross-hole test is the evaluation of 1D shear and compression wave velocity (S-P wave velocity) profiles at the site. The position of the boreholes drilled to carry out the cross-hole tests is shown in Figure 4. The small-strain shear modulus G_0 has been calculated from the values of the shear wave velocity. Values of density equal to 1900 kg/m^3 and 2100 kg/m^3 have been used for the clayey-silt and marlstone layers respectively, in order to perform the computations. The obtained values of G_0 for both CHT1 and CHT2 are shown in Figure 4. It can be seen that, for the first 5 meters, the layer is characterised by low values of G_0 ($\approx$45 MPa), then G_0 increases, meaning that there is a change in the elastic material features. This trend agrees with the results obtained from the stratigraphic study [3].

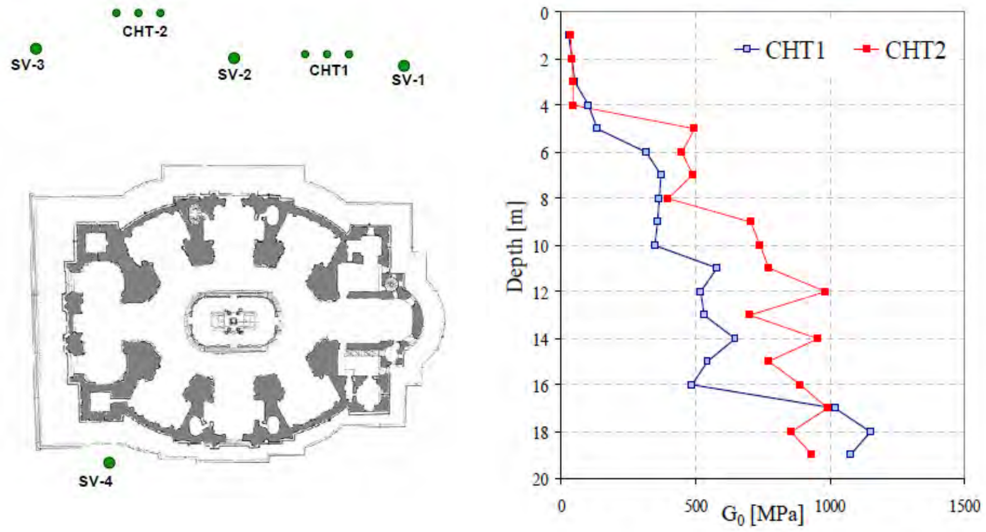

Figure 4: Position of cross-hole tests (CH) performed in 2004 during the geotechnical investigations campaign.

3.2 Definition of seismic input

The principal aim of the study of Scandella, Lai et al [3] was to define the seismic input at the site where the Basilica is located, to be used for the future structural dynamic analysis of the Cathedral. The research was articulated in two phases. The first phase was devoted to the execution of the site-specific Probabilistic Seismic Hazard Analysis (PSHA) and the Deterministic Seismic Hazard Analysis (DSHA), both under the assumption of stiff ground.

The PSHA has been performed using the classical Cornell-McGuire and the zone-free approach by Woo [12]. The output of the PSHA consists of horizontal and vertical probabilistic uniform hazard acceleration spectra at the site, for different reference return periods of 72, 475, 975 and 2475 years, on stiff soil and at the surface ground level. The horizontal PGA expected at Vicoforte, for the return period of 475 years, resulted 0.096 g, whereas for the return period of 2475 years resulted 0.160 g. These are values corresponding to a site of low seismicity with sources located distant from the site.

The DSHA was adopted to define the worst shaking scenario which would occur in the future, compatibly with the tectonic and seismic setting of the region. To this aim, the main seismic sources in the area of interest were identified on the bases of past earthquakes. In particular, a parametric study has been computed to identify the most critical rupture activations and the consequent ground response at Vicoforte. Three main seismic faults have been defined and characterised on the basis of seismic events and a tectonic study: the Monferrato, Western Alps and Western Liguria faults. A parametric study has been implemented performing a total of 144 low frequency and 50 high frequency simulations, to identify the most critical rupture activation scenarios and the corresponding ground shaking at Vicoforte.

The probabilistic approach provided more severe ground shaking scenarios with respect to deterministic methods, with PGA values which exceed the ones computed through the deterministic methods by a factor from 2 to 5. Nevertheless, PSHA and DSHA are both important as they provide complementary information to the predicted hazard.

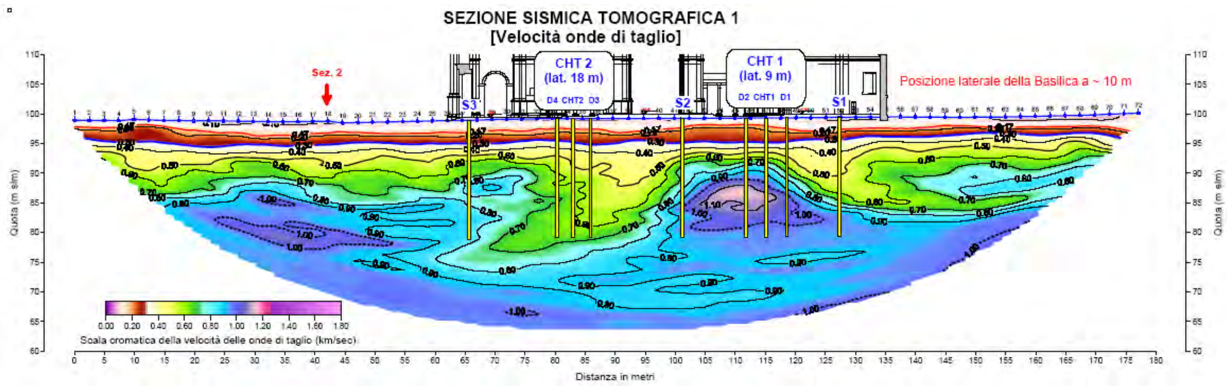

Figure 5: Seismic tomography for S-waves carried out at the Cathedral site of Vicoforte

The second part of the research treated the investigation of the seismic site response through 1D stochastic and 2D deterministic approaches, in order to evaluate possible amplification effects due to localized lithostratigrafic characteristics at the site. To this aim, a 3D subsoil model was constructed integrating the results of advanced geotechnical investigation campaigns including seismic tomography (Figure 5). Four different sections have been chosen to be analysed with both 1D and 2D site response analyses in an effort to capture the behaviour of the subsoil. From the 1D stochastic approach, the mean PGA at the free surface for the 475 years return period results equal to 0.2 g. This corresponds to an amplification of about 2 with respect to the PGA at the outcropping bedrock. The same trend is noticed for the other return periods. The response has also been assessed at the foundation level in order to provide the foundation input motion to be used in future SSI analyses, which resulted equal to 0.15 g for the 475 years return period. Some amplification phenomena are observed also from the results of the 2D deterministic ground response analysis. Finally, as an output of the study, dynamic impedances at the foundations of the Basilica were computed to be used in soil structure interaction analysis [3].

4 FINITE ELEMENT MODEL CALIBRATION

4.1 Experimental testing campaign

The data acquisition campaign at the Sanctuary of Vicoforte was performed by Eucentre, Pavia, in June 2008 [2]. The tests were executed by means of 4 Lennartz 3D/5s triaxial geophones and 5 PCB 393B31 ICP piezoelectric accelerometers. 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 with a sampling frequency of 128 Hz or 512 Hz.

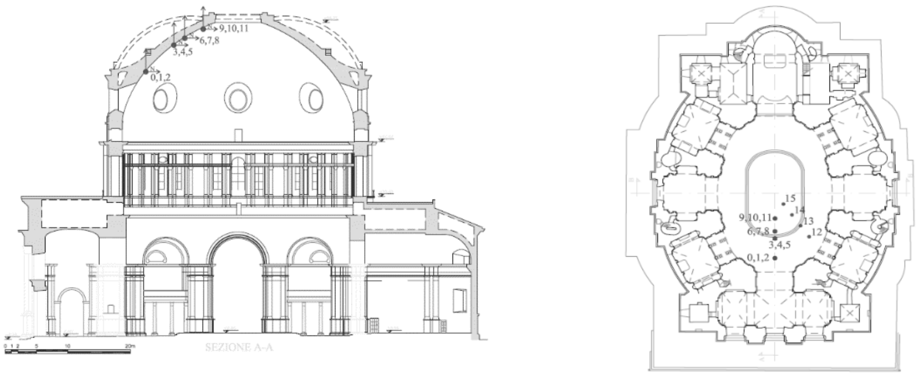

Figure 6: Arrangement of accelerometers and geophones in one of the various setups.

The instruments were positioned according to different acquisition setups to arrive at a global identification of the structure (Figure 6). In the various setups, a number of 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 results of the experimental modal analysis procedure are listed in Table 1. For further details about the identification campaign see Chiorino et al. [2].

4.2 Finite element model

The realisation of a FE model (Figure 7) was conducted by using the Ansys FE commercial code. The geometric data comes 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. The cracks present in the structure were not taken into account in constructing the FE model, based on the assumption that their effects on the natural modes of the structure may be incorporated in the equivalent elastic modulus.

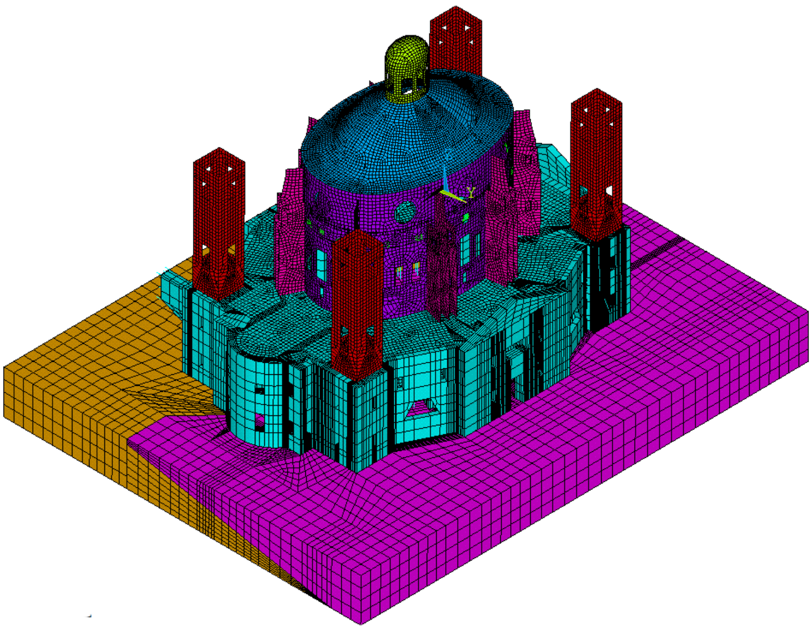

Figure 7: The finite element model inclusive of foundation soil (clay-silt in purple and marlstone in light brown). The other colours identify the different material used for the different macro-elements within the model.

8-nodes hexahedral solid elements were used to mesh the bottom part of the model: the soil, the foundations and the piers. In fact, this part of the building is really massive, and it is not easily modelled by resorting to computationally lighter elements. For what concerns the drum-drome system, the lantern, the bell-towers and the buttresses they were all meshed using 4-nodes shell elements. Moreover, in order to model the interaction between the Sanctuary structure and the nearby convent, spring elements were used to model the boundary condition.

Table 1 and Figure 7 highlight the subdivision in homogeneous portions used for the FE model. The first attempt subdivision was based on typological, historical, structural or modelling reasons (e.g. the relative importance of the dynamic of different zones of the model). The initial values of the mechanical properties are listed in Table 1. For what concerns the density, it was set to 1750 kg/m^3 for the masonry, to 1900 kg/m^3 for the clay-silt and to 2100 kg/m^3 for marlstone based on experimental data [4].

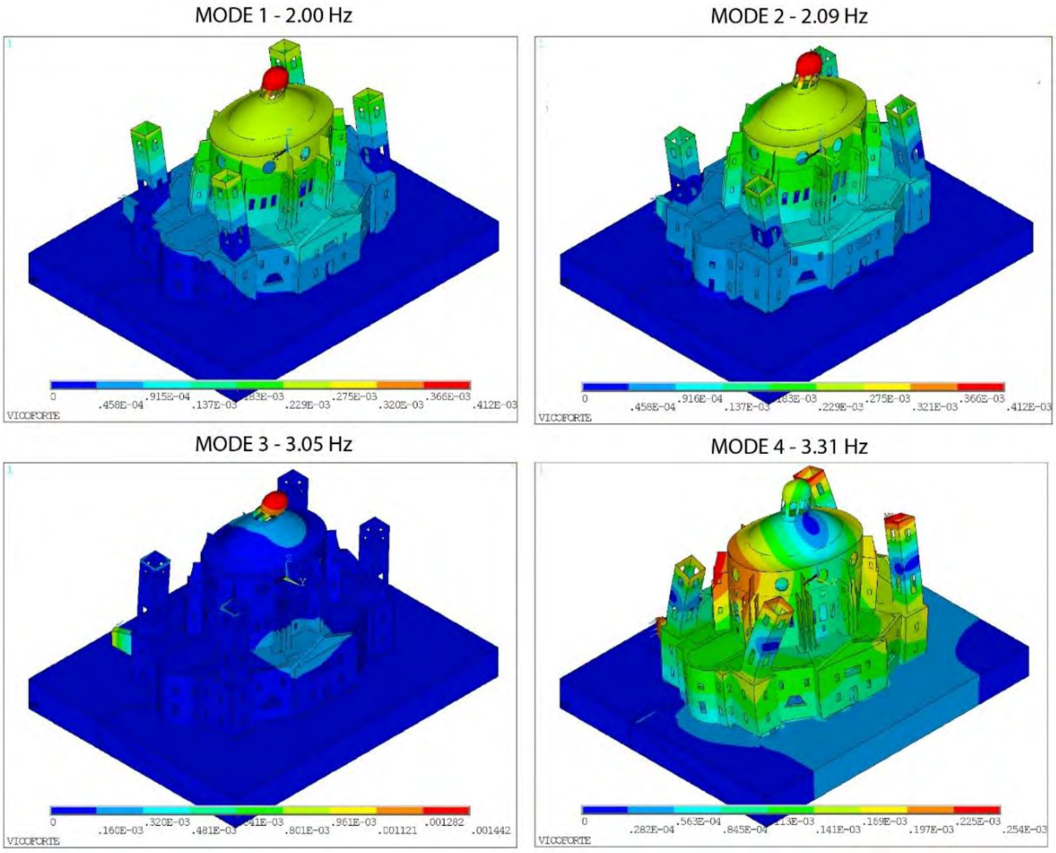

Figure 8: The first four FE modes: the two first flexural modes, the second flexural (or the first mode of the lantern) and the first torsional mode (the frequencies are already referred to the calibrated model).

Finite element mesh boundary conditions for soil modelling are usually: (a) elementary boundary with zero displacement; (b) local boundary with viscous dashpots; (c) lumped-parameter consistent boundaries (mass, spring and dash-pot). In this case, it was chosen, for this first analysis, to use the boundary condition (a). In future, with more experimental tests to be carried on the structure, conditions (b) and (c) will be calibrated. 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.

Figure 8 shows the first four natural frequencies of the model. Bell-towers prevalent modes are not showed in this paper, because they were not identified with the experimental modal analysis campaign (for lack of spatial resolution). The selection of the modes was made based on modal shapes and participation factors: the modes with a high participation were rated as global modes, and those with a low participation were rated as local modes.

4.3 Model calibration

The model updating procedure used in this paper is an indirect one [13]; therefore, it is based on the definition of a robust penalty function. Bakir et al [14] firstly proposed the procedure described hereinafter. One can define a residual vector as the weighted difference between the measured quantities $\mathbf{v}_{EXP}$ and calculated quantities $\mathbf{v}(\boldsymbol{\theta})$, as follows:

$$\mathbf{r}(\boldsymbol{\theta}) = \begin{bmatrix} \mathbf{r}_e(\boldsymbol{\theta}) \\ \mathbf{r}_s(\boldsymbol{\theta}) \end{bmatrix} = \mathbf{W}\left(\mathbf{v}_{EXP} - \mathbf{v}(\boldsymbol{\theta})\right) = \mathbf{W} \begin{bmatrix} \lambda_{1,EXP} - \lambda_{1,FEM} \\ \vdots \\ \lambda_{m,EXP} - \lambda_{m,FEM} \\ \phi_{1,EXP} - \phi_{1,FEM} \\ \vdots \\ \phi_{m,EXP} - \phi_{m,FEM} \end{bmatrix} \tag{1}$$

where $\lambda_{i,FEM}$ and $\lambda_{i,EXP}$ are the i-th analytical and experimental eigenvalues, respectively, whilst $\phi_{i,FEM}$ and $\phi_{i,EXP}$ are the i-th analytical and experimental modal displacements. Finally, $\boldsymbol{\theta}$ describes the parameter vector that usually contains the values of the mechanical properties to be updated (such as elastic and shear moduli, densities, Poisson ratios, etc.). Anyway, one can virtually update any kind of parameters, including boundary conditions, boundary constraints, geometric dimensions, etc. The other term $\mathbf{W}$ is a diagonal weighting matrix which normalises the residue of eigenvalues and eigenmodes because they can be of different order of magnitude. The $\mathbf{W}$ matrix can be defined as follows:

$$\mathbf{W} = diag\left(1/\lambda_{1,EXP}, \ \dots, \ 1/\lambda_{m,EXP}, w_{\phi_1}, \ \dots, w_{\phi_m} \right). \tag{2}$$

and m indicates the number of identified frequencies. For the determination of the weights, consult [14]. Finally, one can update the FE model by minimising the residual $J(\boldsymbol{\theta})$:

$$J(\boldsymbol{\theta}) = \min \frac{1}{2} \mathbf{r}(\boldsymbol{\theta})^T \mathbf{W} \, \mathbf{r}(\boldsymbol{\theta}) \tag{3}$$

through different minimisation procedures, such as a genetic algorithm.

4.4 Model calibration results

The calibration procedure required three different iterations. In fact, at first, only the flexural frequency were calibrated (mode 1, 2, 3 and 5 of Table 2) in order to calibrate the Elastic moduli of the Dome, the buttresses, the lantern and of the two soil layers. In fact, this modes are particularly sensible to the Elastic moduli of these macro-elements. The second iteration, on the hand, mostly involved shear behaviour: a calibration of the fourth (torsional) modes allowed for a further calibration of both elastic moduli and Poisson ratios. A final updating iteration was then performed with all the five modes. Each iteration listed in Table 2 corresponds to the minimisation of the penalty function shown in Eq. (3). This multi-step procedure allowed to overcome local minima in the solution, minimising the number of the parameters to be updated each time.

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 [4]. The Elastic moduli resulting from this test campaign spanned from 1.3 to 4.8 GPa and an average Poisson ratio of 0.38 was obtained, which is accordance with to the results shown in Table 1.

Table 1: Model calibration of the different materials in the different iterations.

Macro-element	Initial value		IT 1	IT 2		IT 3	
	E [GPa]	v	E [GPa]	E [GPa]	v	E [GPa]	N
Basement	3.0	0.35	3.0	3	0.35	2.9	0.35
Drum	2.5	0.35	2.5	3	0.30	2.6	0.30
Bell towers	3.0	0.35	3.0	3	0.35	2.0	0.35
Dome	3.0	0.35	5.9	9.9	0.35	5.9	0.35
Buttresses	2.0	0.35	2.7	2.7	0.30	2.7	0.30
Lantern	1.8	0.35	1.8	1.8	0.35	1.8	0.35
Marlstone	3.18	0.35	4.17	4.17	0.35	4.15	0.35
Clay-silt	0.45	0.35	0.45	0.45	0.35	0.55	0.35

Table 2: Model updating results.

Experimental frequency [Hz]	Updated FE model frequency [Hz]	Error [%]	Classification
1.99	2.00	0.50%	1st flexural Y
2.08	2.09	0.48%	1st flexural X
3.08	3.05	-0.97%	2nd flexural Y
3.42	3.31	-3.22%	1st torsional
3.77	3.52	-6.63%	2nd flexural X
4.11	3.91	-4.87%	1st dome/drum (ovalisation)
5.16	5.17	0.19%	3rd flexural Y
6.02	6.00	-0.33%	1st vertical

5 CONCLUSIONS AND PERSPECTIVES

The paper presented the results, at the date, of a wide investigation and research program, intended to propose the case study of Vicoforte as a reference case at national and international level for the conservation of monuments of large cultural and structural significance, with specific attention to the evaluation of seismic risk. The dynamic characterisation of the building has allowed the identification and classification of 10 vibration modes and the calibration of a FE model that takes into account soil-structure interaction and dynamic effects.

Although several solutions based on active and semi-active control technology for seismic protection of ordinary buildings have been developed and validated in the last two decades, the state of scientific knowledge does not allow yet a direct adoption of this control systems and devices to the seismic protection of cultural heritage buildings. This is due to the several additional requirements that historical building interventions require and to the multitude of geometrical and mechanical uncertainties that characterise these structures. The availability of a reliable numerical model, updated on the basis of experimental modal analysis taking in to account also the soil-structure interaction, allows to evaluate in advance the performance of the control system and any possible negative effect or side-effect.

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