16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 4806
Registration Code: S-U1462374608

SEISMIC ASSESSMENT OF A COMPLEX EARTHEN STRUCTURE: ICA CATHEDRAL (PERU)

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Abstract

This paper discusses the structural performance and addresses the seismic assessment of Ica Cathedral, which is one of the four buildings involved in the ongoing Getty Seismic Retrofitting Project, under the auspices of the Getty Conservation Institute (GCI). The aim of this work is the evaluation of the dominant mode shapes of the structure as predicted by the model, and also the comparison with the dynamic identification performed during the on-site visit conducted during May-June 2015, by a team from the University of Minho, the reproduction of the existing structural damage using advanced numerical analysis and the safety assessment of the structure in its current condition. These tasks are challenging due to the complexity of this historical building and to the numerous unknowns on the morphology and material properties. In fact, the Cathedral is composed by two sub-structures: an external (mostly) earth envelope and an inner timber frame.

The characteristics of the timber substructure have shaped the adopted procedure for its analysis. This substructure is composed by a vast number of main elements usually connected by some additional bracing elements. Another factor adding complexity to the model is the definition of the connections between the elements. This is one of the most challenging aspect to be evaluated in historical timber structures as the characteristics of the joints depends on the region, period of time and carpenters' know-how. For all these reasons, initially, just a single representative bay has been studied. Analysis under vertical loading and parametric analysis under mass proportional lateral loading have been performed using the SAP 2000 software. The structural analysis of the masonry envelope involving a finite element representation is based on geometric and inspection surveys performed. Nonlinear static under gravity loading, eigenvalue, pushover and time history analyses will be addressed. The analysis adopted a macro-modeling approach of the adobe structural parts and base course stone foundation, with a total rotating strain crack model, allowing to consider specified compressive and tensile softening behaviour, based on fracture energy concepts. Finally, the analysis of the entire structure of Ica Cathedral will be presented, with appropriate considerations on the connection between the two sub-structures. While keeping valid the considerations made on the masonry envelope and timber frame, the structure will be studied in terms of nonlinear static under gravity loading, eigenvalue, pushover and time history analyses.

Keywords: earthen masonry; timber structure; quincha technique; seismic assessment; nonlinear analysis

1. Introduction

Earth has been used since ancient times, as a structural as well as a decorative material, for a variety of uses ranging from vernacular to monumental. Historical earthen structures represent unique products of the technology of their time and place; unfortunately, they are at high risk of being heavily damaged and even destroyed due to a variety of reasons. Earthen buildings, such as the Cathedral of Ica, are typically classified as unreinforced masonry structures. Their structural performance is significantly influenced by: the material properties of masonry (high specific mass, low tensile and shear strengths and brittle behaviour); the geometry; the mass and stiffness distribution; the connections among structural elements and the lack of maintenance. Besides, they have often experienced alterations in layout and been damaged by fire, floods and other extreme events. In the last decade, earthquakes have provoked tremendous losses in built cultural heritage and the conservation of earthen architectural heritage represents a complex challenge that needs for methodology and standards [1,2].

Considered as a national monument since 1982, the Cathedral of Ica is representative of ecclesial buildings built in coastal cities during the Viceroyalty of Peru. Since its construction, Ica Cathedral has been occupied, renamed, repaired and altered by different religious orders. However, the main sources of historical damage are related to the seismic events that have occurred during its existence. The MW 7.9-8.0 magnitude inter-plate 2007 *Pisco* earthquake that occurred off the coast of central Peru provoked the partial collapse of the vaulted roof and the main dome, as well as extensive loss to its adobe walls. In 2009 another earthquake led to the total collapse of the main dome. The Cathedral was identified as a case study inside the Seismic Retrofitting Project (SRP), initiative of The Getty Conservation Institute, with the aim to investigate its performance during the earthquakes and to develop effective retrofit methods which can be used for preserving similar churches [3].

In this scenario, this work presents the study carried out on Ica Cathedral by applying a methodology including data acquisition, structural analysis, diagnosis and safety evaluation. Firstly, a deep insight into the main features that characterise the structure of Ica Cathedral is presented according to the information available in literature [3, 4], updated with the results which were derived from the geometrical survey carried out by the University of Minho [5]. The material properties are adopted assuming the experimental tests performed in [6,7] and updated on the basis of the results obtained from the dynamic identification tests [5]. Advanced structural analysis is performed for the two sub-structures, independently; consequently, the interaction of these two sub-structures is investigated by performing analyses on the entire structure of Ica Cathedral. Comparison with existing damage is carried out validating the numerical models and the structural performance is evaluated.

2. General description of the building

Oriented along the typical east-west axis, the Cathedral is adjacent to a three-story modern concrete structure and a fired brick structure to the west, while a concrete portico and a square cloister are located towards its southern side. The foundation, located over compacted silty sand and composed of rubble stone masonry with sand and lime mortar, varies significantly for the configuration and dimension throughout the Cathedral. Above it, a base course that goes all around the cathedral was constructed using fired brick masonry, rubble stone masonry or a combination of both. The Cathedral has a rectangular plan of around 22.5 x 48.5 m^2. It is characterized by changes in floor level and interior pillars, pilasters and piers that separate the different spaces, as shown in Fig. 1. At the back of the Cathedral, behind the altar and the chapels, a series of spaces is present, including the sacristy, a reception and offices. The structural system of the Cathedral can be divided into two main sub-structures from the point of structural composition: an external masonry envelope and an internal timber frame.

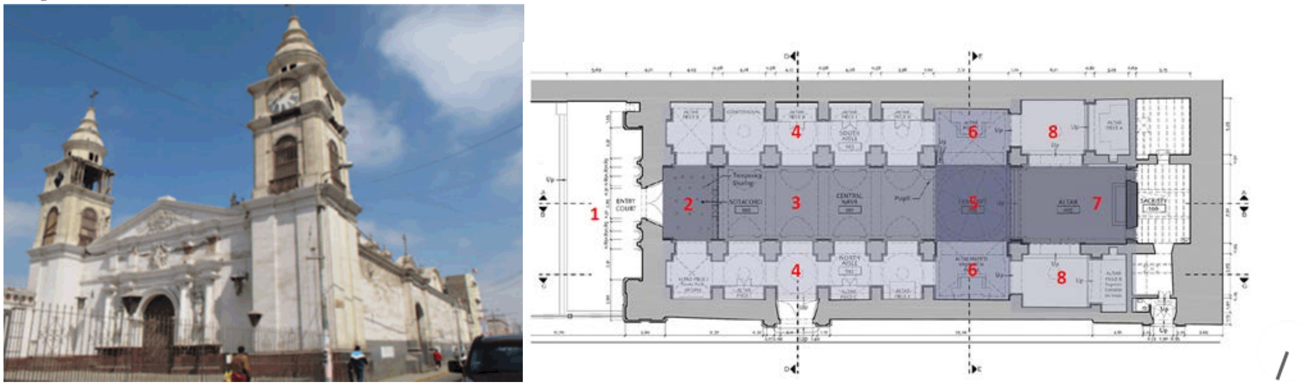


Fig. 1 – The front façade (right) [1] and the schematic of the plan of Ica Cathedral (left): (1) atrium; (2) main entrance - the so called *sotacoro* - with extensions at either lateral side; (3) main nave; (4) side aisles; (5) crossing of the transept; (6) arms of the transept; (7) altar; and (8) chapels

At the exterior, a massive and load-bearing masonry envelope surrounds the Cathedral: it consists mainly of the front façade with two bell towers, the lateral longitudinal walls and the back façade. The front façade is made of fired brick masonry with lime mortar and a thinner pediment is present at the top. Each bell tower is composed of a timber internal structure that sits on fired brick base having a cavity. The longitudinal walls consist of three different layers of masonry varying throughout the structure in terms of dimension: rubble stone, fired brick and adobe, sequentially from the base upwards. The masonry structure is typically finished with mud plaster and painted gypsum.

The internal space of Ica Cathedral is divided by a series of pillars, pilasters and piers which support a system of longitudinal and transversal beams, that in turn carry a complex vaulted roof system. These structural elements are constructed by applying the so called *quincha* technique. The pillars and the pilasters are essentially hollow structures composed of numerous posts, horizontal and diagonal elements which are wrapped with flattered cane reeds (*caña chancada*) and finished with mud plaster and gypsum. The complex vaulted roof system includes a main umbrella dome, barrel vaults, barrel vaults with lunettes and small domes. In general, the domes are composed of several ribs and two ring beams located at the top and at the bottom. The barrel vaults are constructed by a system of principal and secondary arches: because of the large spans, each of these elements consists of several segments made of nailed planks. The intrados of the vaulted roof framing system is covered by plaster and flattered cane reeds, while cane reeds finished with layers of mud plaster (*caña brava*) cover the extrados. A flat wooden ceiling is used to cover the space between the timber and masonry structures with layers of fired brick masonry and sand, lime and cement mortar. The members in each part of the timber framing are connected to the adjacent members by means of different types of timber joints, such as mortice and tenon and half–lap connections. Three wooden species are present throughout the timber structure: cedar (*Cedrelaodorata*), sapele (*Entandrophagmasp*) and huarango (*Prosopissp*).

3. Material Properties

The material properties of the different masonries – i.e. rubble stone, fired brick and adobe – were primarily derived from the experimental campaign carried out by the Pontificia Universidad Católica del Perú (PUCP) for the Getty Conservation Institute [6]. Besides, national technical building standards and existing literature were used [8, 9, 10]. The modulus of elasticity of the different types of masonry was calibrated by performing a model updating of the masonry envelope as presented in Section 4. The updated values of the modulus of elasticity of the different masonries were adopted for the combined model. A summary of the material properties assumed in the numerical models is presented in Table 1. The physical nonlinear behaviour of masonry was simulated in the numerical models by using the *Total Strain Rotating Crack* material model which is available in DIANA [11]. An implicit shear term that is used by the total strain based on rotating model provided co–axiality of the rotating principal stress and strain. The tensile softening and the compression behaviour of masonry were defined by an exponential and a predefined parabolic curves respectively, which depend on the corresponding strengths and fracture energies.

Table 1 – Material properties of the different types of masonry

Properties	Rubble Stone	Fired Brick	Adobe
Modulus of elasticity [MPa]	300 (720)*	340 (850)*	93 (220)*
Poisson's ratio	0.20	0.20	0.20
Compressive strength [MPa]	0.60	1.70	0.46
Tensile strength [MPa]	0.06	0.10	0.05
Fracture energy (compression) [N/mm]	1.50	3.50	1.00
Fracture energy (tension) [N/mm]	0.01	0.01	0.01
Specific Weight [kN/m^3]	19	19	19

* Modulus of elasticity derived from the model updating (Section 4)

The material properties of the different wood species present in Ica Cathedral – cedar, sapele, huarango – were assumed on the basis of the results obtained from the experimental campaign carried out by the Universidad Nacional Agraria La Molina (UNALM) by request of the Pontificia Universidad Católica del Perú (PUCP) and the Getty Conservation Institute (GCI) [7]. The weight of the covering layers of the timber structure was taken into account by assigning an equivalent specific weight to the timber elements carrying them. In other words, the specific weight assigned to these elements in the numerical model was not equal to the value corresponding to the wood species in which the timber element is constructed. For details, please refer to [12]. No increase in the stiffness due to the confinement effect of these covering layers was considered for these timber members and the modulus of elasticity corresponding to their wood species was adopted in the numerical models. All the timber elements were assumed to have an isotropic homogeneous and linear behaviour in the numerical models. In order to carry out the verifications for the representative bay (Section 4), the design load-carrying capacities were calculated according to Eurocode 5 [13] from the admissible load-carrying capacities which were assumed considering the classification, performed by UNALM [7], of the wood species into the structural classes recommended by the Peruvian Code [14]. In particular, huarango was evaluated corresponding to *Class A*, while sapele and cedar to *Class B*. A summary of the material properties of the different wood species is presented in Table 2.

Table 2 – Material properties of the wood species according to UNALM [7] and to the Peruvian Code [14]

Properties	Cedar	Sapele	Huarango
Density [g/cm^3]	0.38	0.49	1.04
Mean modulus of elasticity in flexure [MPa]	9380	8610	16900
Poisson's ratio	0.30	0.30	0.30
Admissible bending strength [MPa]	14.70	14.70	20.60
Admissible compressive strength parallel to grain [MPa]	10.80	10.80	14.20
Admissible compressive strength perpendicular to grain [MPa]	2.70	2.70	3.90
Admissible shear strength [MPa]	1.20	1.20	1.50
Admissible tensile strength parallel to grain [MPa]	10.30	10.30	14.20

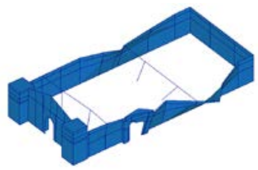
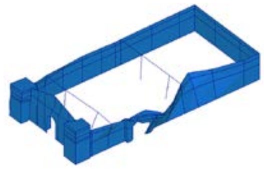
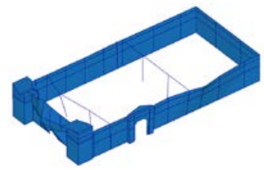
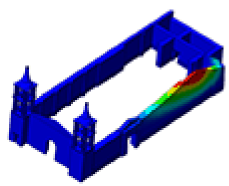
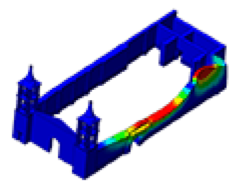
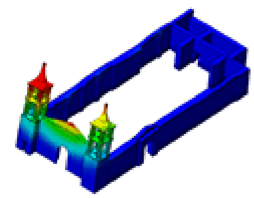
4. External Masonry Structure

On the basis of the geometric survey carried out by the PUCP [6], a 3D finite element model of the masonry structure in its current state was generated in Midas FX+ for DIANA [11]. The model of the masonry envelope included all the parts described in Section 2 with the exception of the sacristy, reception and offices on the western side as they are likely to exhibit an independent behaviour [12]. The cloister on the southern side was also not modelled but its buttressing effect was simulated by using *one-noded translation spring* elements in the numerical model. The intersecting walls as well as the horizontal layers of different masonries were assumed to have full connectivity. All the timber elements in the bell tower, made of huarango, were hypothesised to be rigidly connected. Timber lintels were located in correspondence to all the openings of the structure in order to avoid a concentration of damage that was not observed during the conducted damage survey [5]. The created FE mesh for the model of the masonry envelope was composed of 353,866 *3D isoparametric solid linear* elements and 345 *one-noded translation spring* elements, with 81,236 nodes in total.

Eigenvalue analysis was first carried out on the numerical model in order to evaluate the dynamic response in terms of natural frequencies and mode shapes. The results were compared to those obtained from the dynamic identification tests performed by the University of Minho [5]. Ambient vibration tests were performed with piezoelectric accelerometers located in several locations at the top of the lateral longitudinal walls and the results were processed using ARTeMIS software [15]. Both frequency and time domain methods were applied to process the data, specifically the *Enhanced Frequency Decomposition Domain Method* (EFDD) and the *Stochastic Subspace Identification Method (SSI)*: as more accurate results were provided by the latter, only the results obtained by using SSI are presented in this paper. Table 3 shows the first three modes of vibration of the structure which were considered for updating the model: the first mode identifies the first mode of the lateral longitudinal walls with higher intensity in the northern wall near the transept area; the second mode corresponds to second order curvature of the northern wall, with a deflection point near the middle span; the third mode identifies a complex mode including the first movement of the façade. As the average error calculated between the frequencies obtained experimentally and numerically was about 39%, model updating was carried out for the numerical model of the masonry envelope [12]. This was done by adjusting the modulus of elasticity of the different types of masonry in the numerical model within a reasonable range of values in order to match the natural frequencies obtained from the experimental investigations. The average error between the experimental frequencies and those observed for the calibrated model consequently came down to a value lower than 6%. As previously mentioned, the values of the modulus of elasticity of the different types of masonry in the calibrated model are shown in Table 1.

In order to evaluate the seismic behaviour of the masonry envelope, nonlinear static (pushover) analyses were performed on the calibrated model by applying a mass proportional approach in the primary axes of the numerical model, in both positive and negative directions. The lateral load-carrying capacity is 0.27g and 0.36g in the XX+ and XX– (longitudinal) directions, respectively. In the YY+ and YY– (transversal) directions, the maximum lateral load that can be applied to the model is 0.25g and 0.22g, respectively. It is worth noting that the comparison between the results obtained from the pushover analyses performed on the calibrated and un–calibrated models shows a moderate increase in lateral load-carrying capacity for the former (Fig. 6). The most relevant failure mechanisms of the model of the masonry envelope are identified when lateral load is applied in the XX– and YY– directions. Under the lateral load of 0.36g in the XX– direction, the failure mechanism consists of the out-of-plane mechanism of both the front façade and the bell towers. When the lateral load of 0.22g is applied in the YY– direction, the failure mechanism is identified as the out-of-plane mechanism of the northern lateral longitudinal wall. According to the obtained results, the north-western corner of the masonry envelope is identified as the most vulnerable region of the masonry envelope. Correlation was observed between the crack pattern in terms of tensile strains obtained from these pushover analyses on the calibrated model and the existing damage observed in-situ [5], validating the numerical model. It should be mentioned that these results are also confirmed by those obtained from the time history analysis performed on the calibrated model of the masonry envelope.

Table 3 – Comparison between the frequencies and mode shapes obtained experimentally and numerically

	Mode 1	**Mode 2**	**Mode 3**
Experimental Mode Shape			
	2.84 Hz	3.21 Hz	3.92 Hz
Numerical Mode Shape (Calibrated Model)			
	2.56 Hz	3.01 Hz	3.94 Hz

5. Internal timber structure

On the basis of the information available in literature [3,4], the experiment campaign performed by the University of Minho [5] and the geometrical assumptions presented in [12], the models of the representative bay and of the whole complete timber structure were constructed.

5.1. Representative bay

A 3D finite element model of the representative bay was constructed in SAP 2000 software [16]: the FE mesh was composed of 1136 nodes and 1344 *Frame* elements (Fig. 2). The model included the following structural parts: a barrel vault with lunettes; two aisles' domes; aisles' joists used for the flat roofed area; a system of longitudinal and transversal beams; two nave pillars and two pilasters; timber frameworks connected to the nave pillars and the pilasters. The base of the posts composing the nave pillars and the pilasters was pinned. The timber joints were modelled as hinges or rigid connections depending on the mechanical behaviour of the timber joints present in the representative bay.

Structural analysis of the representative timber bay was performed considering different loading conditions. In order to investigate the effect of the principal timber joints of the representative bay, parametric analyses was carried out by applying a lateral loading using a mass proportional approach. The model of the representative bay was compared to models that differed only by the assumptions of modelling for specific sets of the timber connections. The obtained results show that the timber joints between the elements of the barrel vault with lunettes are the most critical.

Global and local verifications were carried out for the all the straight elements of the representative bay in order to evaluate the compliance under self-weight, live load and earthquake load with the various criteria specified by the Eurocode 5 [13]. According to the obtained results, few timber elements are not verified for the serviceability limit state, for vertical loading. However, the values of displacements that are not verified are only slightly higher than those recommended, which seems not much of an issue for historic buildings. For the ultimate limit state under vertical load (1.35G), both the global and local verifications are satisfied for all the timber elements of the representative bay. When the ultimate limit state under seismic combination (G+E) is considered, the global verifications are not satisfied for the beams at the top of the lunettes and the posts close to the masonry walls. The local verifications for ultimate limit state under seismic combination (G+E) show that the stresses that occur in correspondence to the mortise and tenon connections of the beams at the top of the lunettes are large enough to provoke the failure of the structure. The obtained results confirm the damage observed for the timber structure of Ica Cathedral after the earthquakes.

5.2. The whole timber structure

The numerical model of the whole timber structure was constructed by using Midas FX+ for DIANA software [11]: the FE mesh consisted of 17,209 C*lass–I beams* elements with shear deformation and 15,083 nodes (Fig. 2). The model included all the structural parts of the internal timber structure, with some simplifications for the *sotacoro*, the chapels and the altar due to limited information available [12]. Besides, bracing elements made of cedar were added in the barrel vaults of the transept, of the altar as well as of the chapels in order to avoid local natural modes of vibration in the FE model. Additionally, translational displacements were restrained at the base of posts composing the nave pillars and of the pilasters, and in correspondence to the connection with the masonry walls. All the timber joints were modelled as rigid connections.

According to the results obtained from linear elastic analysis under self-weight, high displacements are observed for the transept and the barrel vault covering the altar, with a maximum value of 1.8 cm. In terms of internal forces, the crossing of the transept is identified as the most vulnerable part: in particular, while the ribs that compose the main dome show relatively low values of compressive axial force, the ring beam at its bottom is subjected to significant combined internal forces. Besides, significant compressive axial forces with biaxial bending moments are observed in the posts of the central pillars supporting the structure: the sum of the vertical reactions calculated at the bottom of these posts is more than 40% of the weight of the total structure (5955 kN). The behaviour of the whole timber structure under horizontal load was investigated by applying a mass proportional lateral loading in the XX and YY directions. The results show that the largest displacements are concentrated in the upper part of the structure – in particular in the main dome as well as in the barrel vaults, which are the less stiff parts – while the piers and pilaster remain almost un-deformed.

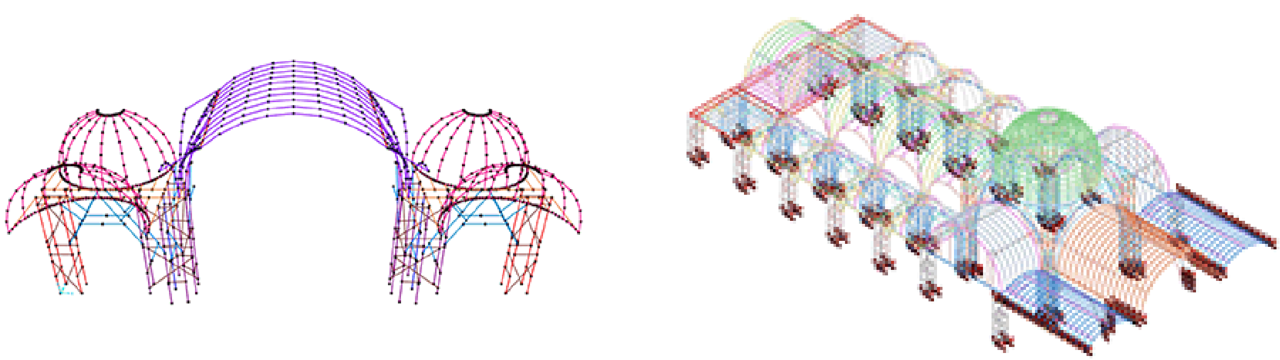


Fig. 2 – FE mesh of the representative bay in SAP 2000 (left) and of the whole timber structure in Midas FX+ for DIANA (right)

6. Combined Model

In order to investigate the interaction between the two sub-structures, a numerical model of the combined structure of Ica Cathedral was constructed in Midas FX+ for DIANA [11]. The assumptions presented for the masonry envelope and the timber structure, presented in Section 4 and Section 5, hold true also in this model. *Class–I beam* elements with shear deformation were adopted for modelling the timber structure, while *3D isoparametric solid linear* elements were used to model the masonry structure. In addition, *one-noded translation spring dashpot* elements were used to simulate the influence of the adjoining cloister. A crucial aspect was the definition of the connections existing between the two structural systems, for which little information was available. These connections were assumed to exist between: (1) the wooden beams in the upper part of the *sotacoro* and the fired brick façade and the longitudinal masonry walls; (2) the transversal wooden beams of the bays and the longitudinal masonry walls; and (3) the wooden beams supporting the barrel vaults of the chapels and altar and the masonry walls. In particular, these connections were modelled in the numerical model by merging the nodes of *class–I beam* elements with those of *3D isoparametric solid linear* elements. Timber plates made of huarango were assigned to the upper part of the masonry walls flanking the altar to avoid undue concentration of damage. Fig. 3 shows the FE mesh of the combined model including both timber and masonry structures.

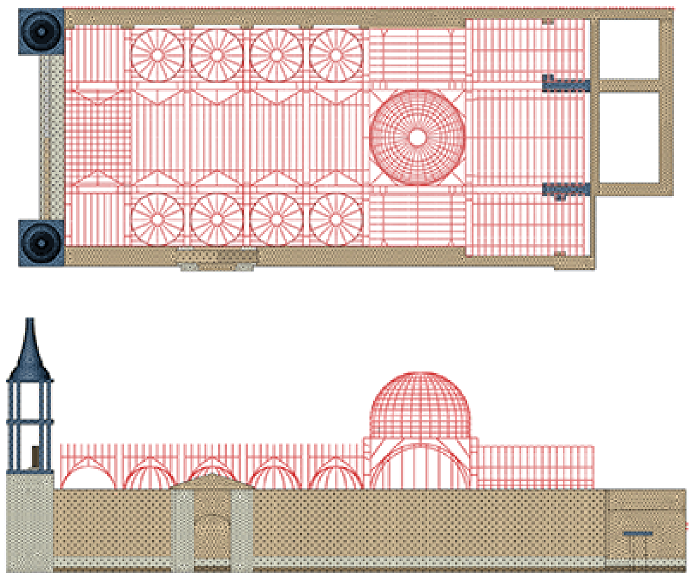


Fig. 3 – Numerical model of the combined structure of Ica Cathedral

The structural behaviour of the complex model was studied by performing several analyses, including eigenvalue, nonlinear under self-weight, mass proportional pushover and time history analyses. The mode shapes obtained from the eigenvalue analysis identified the main dome, the central part of the barrel vault with lunettes, the upper part of the barrel vault covering the chapels and the altar as the most likely vulnerable regions of the structure – these parts correspond to the regions where more damage was observed after the earthquakes. Besides, both the longitudinal walls get activated simultaneously in the obtained modes of vibration, providing slightly better correlation between numerical and observed dynamic response.

As the most critical failure mechanisms in the model of the masonry envelope were identified in the XX– and YY– directions, the seismic behaviour of the combined structure was evaluated by performing two mass proportional lateral loading analyses in these two directions, after having introduced the self-weight. The results obtained from the nonlinear analysis under self-weight show high value of displacements in the barrel vaults and in the main dome ranging from 1.0 cm to 2.0 cm. No cracking is observed throughout the masonry envelope, indicating that there is no onset of plasticity in the masonry under self-weight. The distribution of displacements observed for the combined model when lateral load is applied in the XX– and YY– directions is presented in Fig. 4.

The maximum lateral load that can be applied to the numerical model in the XX– direction is 0.45g. The failure mechanism in the masonry envelope is identified as the out-of-plane failure of both the front façade and the bell towers. The crack pattern in terms of tensile strains shows vertical separation cracks occurring in the connection between the bell towers and the lateral longitudinal walls, and diagonal cracks propagating throughout the adobe wall adjoining the southern bell tower. Besides, flexural cracks can be observed at the base of both the bell towers and the front façade. Extensive tensile damage is also seen where the walls of the altar are connected to other adjoining walls (Fig. 5). When the lateral load is applied in the YY– direction, the maximum lateral load-carrying capacity was calculated with a value of 0.28g. The failure mechanism in the masonry under this loading consists of the out-of-plane failure of the northern lateral longitudinal wall. Extensive tensile damage is observed in these regions: flexural cracks occur throughout the length of the longitudinal wall – at the rubble stone base course and in the interface between adobe masonry and the fired brick base course – and cracking progresses significantly from the connection between the northern bell tower and the front façade through the base of the former (Fig. 5).

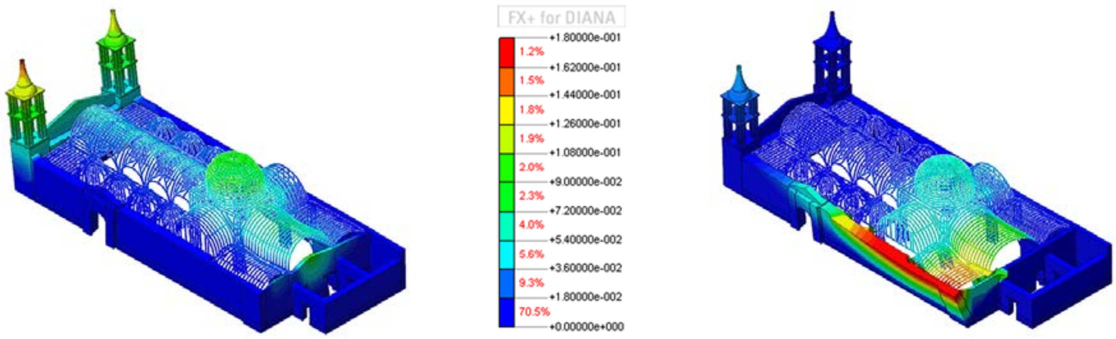


Fig. 4 – Distribution of displacements observed for the combined model under the maximum lateral loads applied in XX- (right) and YY- (left) directions. Unit: m

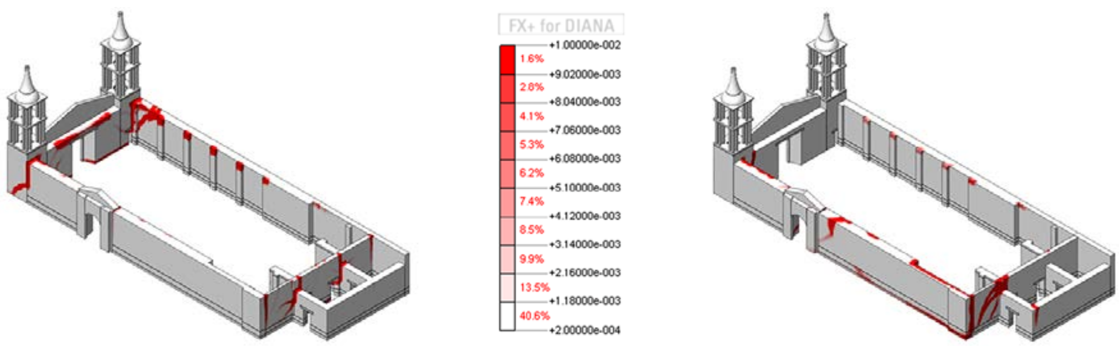


Fig. 5 – Crack pattern in terms of tensile strains observed for the combined model under the maximum lateral loads applied in the XX- (right) and YY- (left) directions

The behaviour of the combined model (MC) under lateral loading in the XX– and YY– directions is summarized in Fig. 6 in terms of load displacement diagrams. The respective load displacement diagrams obtained from the models of only the masonry envelope, calibrated (ME – C) and uncalibrated (ME – UC), are also plotted with them. In particular, the control points were assumed as the nodes where high values of displacements were observed: a node at the top of the northern bell tower and one at the top of the north-western corner were considered for plotting the load displacement curves of the different models under lateral load in the XX– and YY– directions, respectively. According to the obtained results, the combined model presents higher values of lateral load-carrying capacity when compared to the models of only the masonry envelope, in both the directions. A similar initial stiffness is observed for ME – C and MC under lateral load in the XX– direction. On the other hand, an appreciable decrease in the initial stiffness is seen for MC as compared to ME – C when lateral load is applied in the other direction. While a moderate difference of lateral load-carrying capacity is observed for ME – C and ME – UC (about 10%), MC shows an increase in seismic capacity of about 25% as compared to ME – C, in both the directions considered. The obtained results point out that the interaction between the two sub-structures affects significantly the seismic behaviour of the entire structure of Ica Cathedral.

A time history analysis was performed on the combined model considering Rayleigh viscous damping. The Rayleigh damping parameters for the combined structure were calculated by considering all the natural modes of vibration until 80% mass participation was reached. Explicit time integration using the Hilber – Hughes-Taylor method – which also introduces numerical damping of noise resulting for abrupt change of masonry from elastic to fully cracked state with zero stiffness change – was adopted. Using the elastic response spectrum according to the Peruvian Code [17] for the region of Ica, two artificial accelerograms were generated using the software SeismoArtif v2.1 [18] and the Butterworth filter was applied to them for eliminating noise by means of the software SeismoSignal v5.1 [19]. In order to apply a single earthquake to the combined model, the artificial accelerograms – which were uncorrelated, i.e. peaks in acceleration do not occur at the same time with respect to each other – were applied to the combined model in two orthogonal directions to each other (X and Y). The loading-unloading diagrams plotted for the masonry of the combined model in the two orthogonal directions in which the accelerograms were applied indicate that masonry lost its loading-unloading capacity and severe damage occur in the structure under the applied dynamic loading. A correlation of cracks obtained in the numerical model subjected to dynamic loading was carried out with the existing damage observed in-situ. Critical cracking present in the structure after the earthquakes were obtained in the numerical model, such as parallel cracking in the adobe masonry of the northern lateral longitudinal wall, diagonal cracks in the front façade and separation cracks at the base of the pediment. It should be mentioned that the nonlinear dynamic analysis reproduced cracks existing currently in the structure which were not obtained from pushover analyses (Fig. 7).

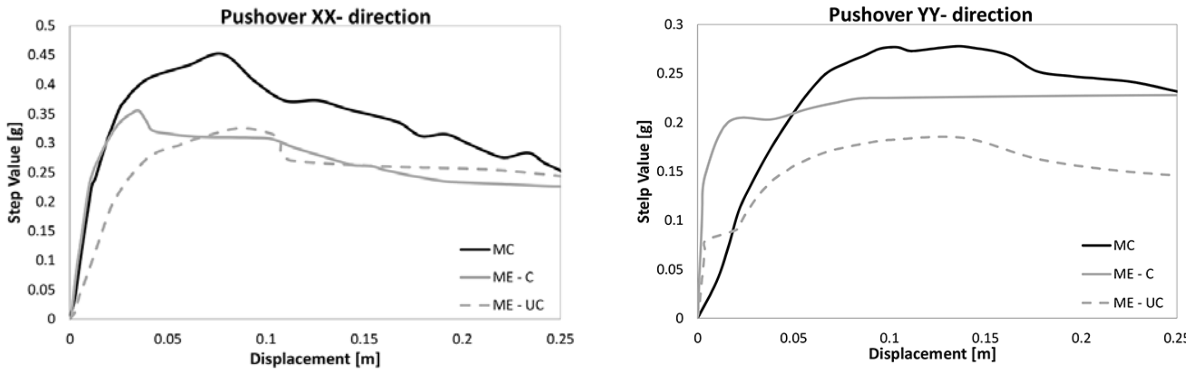


Fig. 6 – Comparison of the load displacement diagrams obtained for the combined model (MC) and the model of only the masonry envelope, calibrated (ME – C) and uncalibrated (ME – UC)

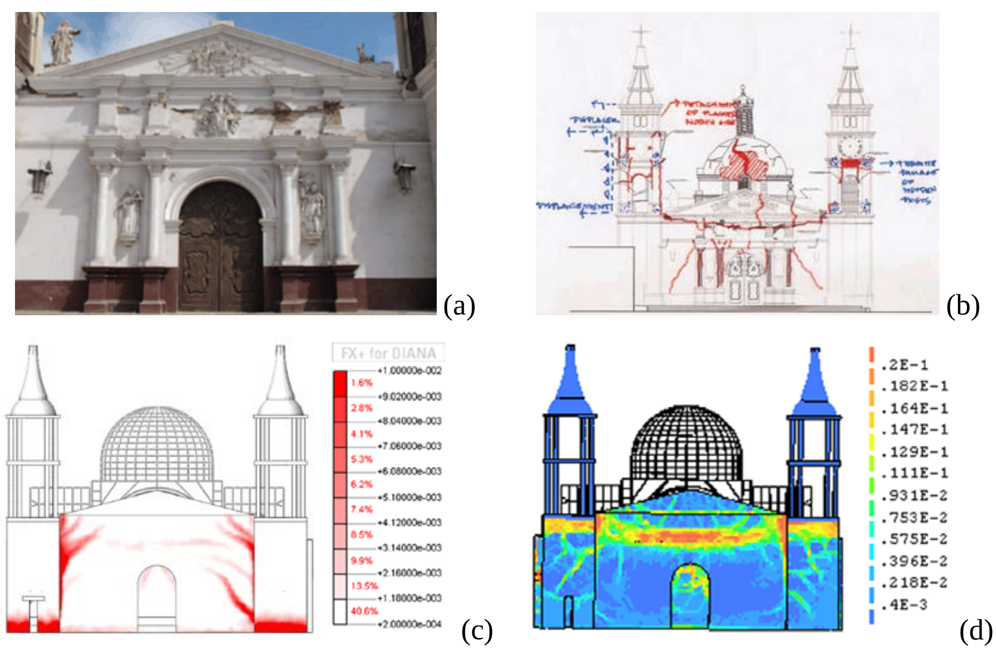


Fig. 7 – Correlation of cracks observed in the numerical model and in-situ: (a) detail of the damaged front façade [1], (b) crack survey [1], (c) crack pattern from pushover analyses and (d) crack pattern from nonlinear dynamic analysis

7. Conclusions

The seismic assessment of Ica Cathedral, a complex earthen structure constructed by applying *quincha* technique, was presented in this paper in order to reproduce the existing damage and to assess safety in its current condition. To reach this aim, several numerical models were created on the basis of the information available in literature [3, 4], experimental campaign [5] and experimental tests [6,7]. The structural behaviour of the two sub-structures composing the Cathedral – i.e. an internal timber structure and an external masonry envelope – were first evaluated independently. Afterwards, the interaction of these two sub-structures was investigated by performing structural analysis on the entire structure of Ica Cathedral.

Model updating was carried out comparing the dominant modes of the structure predicted by the model of the masonry envelope and the results obtained from the dynamic identification tests [5]. The moduli of elasticity of the different types of masonry were adjusted in the numerical model in order to match the natural frequencies derived from the experimental investigations. According to the different pushover analyses performed on the calibrated model of the masonry envelope, the most relevant failure mechanisms were identified in the XX– and YY– directions. In particular, the lower bound lateral load-carrying capacity of only the masonry envelope was calculated in the YY– direction, with a value of only 0.22g. The north-western corner was identified as the most vulnerable part of the structure, as also confirmed by the results obtained from the time history analysis.

Afterwards, the structural performance of the representative bay was investigated by performing linear elastic analysis under different loading conditions and verifying the compliance with the various criteria specified by the applicable normative. While the timber members are mostly verified for the ultimate limit state and for the serviceability limit state when vertical loading is considered, under horizontal loading the verifications for ultimate limit state are not satisfied for the beams at the top of the lunettes and the posts close to the masonry walls. These parts represent the most vulnerable regions of the representative bay, as verified by the damage observed in-situ after the earthquakes. According to the qualitative study carried out on the whole timber structure, the posts and the main dome composing the crossing of the transept represent the most critical parts, confirming the existing damage.

Although the structural behaviour seems to be reasonably well represented by independent structures, the linear and nonlinear analysis performed on the combined model of the Cathedral demonstrated that the building is influenced by the interaction of the timber structure with the masonry envelope. The connections between the two sub-structures are a critical factor influencing significantly the structural behaviour of the Cathedral. An increase in seismic capacity of about 25% was observed for the combined model under lateral load in both the directions, when compared to the calibrated model of only the masonry envelope. The lower bound capacity of the combined model was found in the transversal direction, with a value of only 0.28g – much lower than the PGA considered in the code for the region of Ica [17]. Global strengthening is therefore required in order to avoid the out-of-plane mechanisms identified for the most vulnerable regions of the structure, in particular for the north-western corner of the Cathedral, by means of tying, collar beams and/or proper connections between the two sub-structures.

8. Acknowledgements

This work was carried out with funding from the Getty Seismic Retrofitting Project under the auspices of the Getty Conservation Institute (GCI). This work is also partially financed by FEDER funds through the Competitivity Factors Operational Programme - COMPETE and by national funds through FCT – Foundation for Science and Technology within the scope of the projects POCI-01-0145-FEDER-007633 and PTDC/ECM-EST/2777/2014.

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