13TH CANADIAN MASONRY SYMPOSIUM
HALIFAX, CANADA
JUNE 4TH – JUNE 7TH 2017

ICA CATHEDRAL, PERU: SAFETY ASSESSMENT AND STRENGTHENING

Ciocci, Maria Pia[1]; Sharma, Satyadhrik[2] and Lourenço, Paulo B.[3]

ABSTRACT

The Cathedral of Ica, Peru, is an 18th century church that was heavily damaged by the earthquakes experienced in 2007 and 2009. Considered representative of religious buildings built in coastal cities of Peru, Ica Cathedral was selected as one of the prototype buildings in the Seismic Retrofitting Project, a collaborative project between the Getty Conservation Institute, the University of Minho, the Pontifical Catholic University of Peru and the Ministry of Culture of Peru. The Cathedral is characterized by two sub-structures: an external masonry envelope made of rubble stone, brick and adobe masonries, and an internal timber framing system constructed by applying the *quincha* technique. This paper deals with the safety assessment of the Cathedral in its current condition and the proposal of effective strengthening to reduce its vulnerabilities. Nonlinear behavior is assumed for the different types of masonry by using the *Total Rotating Strain Crack* model, while isotropic homogeneous and linear behavior is adopted for timber with adequate considerations for the connections. Advanced structural analysis is first carried out on the two sub–structures, independently. Afterward, several analyses including nonlinear static and nonlinear dynamic loading analyses are performed on the FE model of the entire structure of Ica Cathedral in order to: (1) validate the numerical model; (2) investigate the interaction of the two sub-structures; (3) evaluate the lateral load–carrying capacity and (4) identify the main failure mechanisms. Thereafter, a global strengthening of the structure is proposed guaranteeing the principles of minimum intervention and reversibility. Finally, the FE model including the strengthening is studied in order to test the efficiency of the strengthening proposal.

KEYWORDS: *half-timber construction, quincha technique, nonlinear analysis, numerical modeling, safety assessment, strengthening*

[1] PhD Student, ISISE, Department of Civil Engineering, University of Minho, Campus de Azurém, Guimarães, Portugal, mariapiaciocci@gmail.com
[2] PhD Student, ROSE Programme, UME School, Istituto Universitario di Studi Superiori (IUSS), Pavia, Italy, satyadhrik.sharma@iusspavia.it
[3] Full Professor, ISISE, Department of Civil Engineering, University of Minho, Campus de Azurém, Guimarães, Portugal, pbl.@civil.uminho.pt

INTRODUCTION

Earth has been used in Peru to construct both monumental and vernacular buildings for almost four thousand years. Since then, these adobe constructions were often combined with the use of a traditional construction technique known as *quincha,* see [1]. However, it was during the Hispanic Viceroyalty and the early Republic periods (1534-1821 CE) that the combination of these indigenous construction techniques reached its technological climax in Peru. In particular, ecclesiastic and other representative buildings were increasingly being built by the Spaniards with half-timber structures characterized by an external masonry envelope surrounding a complex *quincha* vaulted roofing system [2].

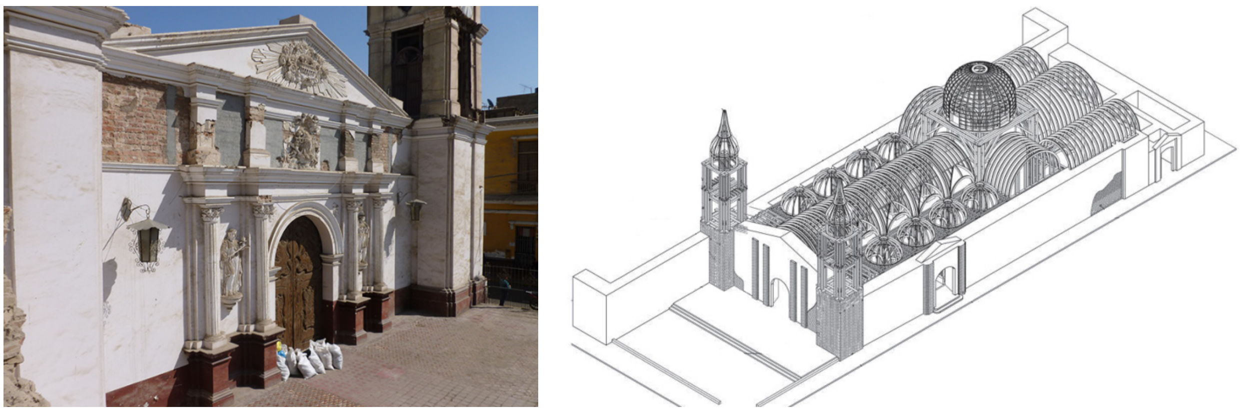
Today, these Peruvian historical structures possess a great value for the society; however, they are also at a high risk of being heavily damaged and even completely destroyed due to several threats, including earthquakes. Their conservation and preservation needs for a multi-disciplinary approach with an organization in steps which are similar to those used in medicine: condition survey (anamnesis), identification of causes and decay (diagnosis), choice of remedial measures (therapy) and control of the efficiency of the interventions (controls). In such a scenario, a full understanding of the structural behavior and material characteristics is essential for any project in these structures [3]. However, the application of this methodology to cultural heritage buildings presents a number of challenges for a variety of reasons: (1) intrinsic complexity due to their history, materials and assembly; (2) difficulty to apply modern codes and building standards; (3) adoption of respectful and efficient interventions [4].

Declared as a national monument since 1982, the Cathedral of Ica is representative of ecclesial buildings constructed in coastal cities during the Hispanic Viceroyalty. The recent earthquakes which occurred in 2007 and 2009 provoked widespread damage to the cathedral including the collapse of the vaults and of the main dome, as well as extensive loss to its adobe walls. As part of the Seismic Retrofitting Project (SRP), initiative of the Getty Conservation Institute (GCI), this work on the Cathedral of Ica aims to apply a methodology that can be used to investigate the seismic performance and to develop effective retrofit techniques for similar constructions.

DESCRIPTION OF THE BUILDING

A deep insight into the main features responsible for the intrinsic complexity that characterizes the structure of Ica Cathedral was necessary before delving into any form of simulations on such a structure. Such information was primarily obtained from historical research and condition surveys carried out by GCI [5] and additional knowledge derived from an experimental campaign performed by the University of Minho [6].

Located at the corner of an urban block in the historic centre of Ica, the one story cathedral has a rectangular plan that consists of a main entrance with a choir loft, a main nave, two lateral aisles, a transept, an altar and two chapels. A series of spaces including a sacristy, a reception and offices is present to the west, while a concrete portico and a square cloister are located towards its southern side. Figure 1 shows the overall structural scheme of the cathedral. The fired brick front façade is flanked by two bell towers which are composed of timber frames resting on fired brick bases. The thick lateral walls are constructed with adobe masonry over a fired brick base course and rubble stone foundations. The internal timber frame is composed of a series of pillars which support a complex vaulted timber roofing system including a main dome, barrel vaults and small domes. In general, the pillars are composed of numerous posts braced by means of horizontal and diagonal elements, while the domes and the barrel vaults are constructed with wooden ribs and arches. These timber members are connected to the adjacent members by means of different types of timber joints and are covered by *quincha* layers.



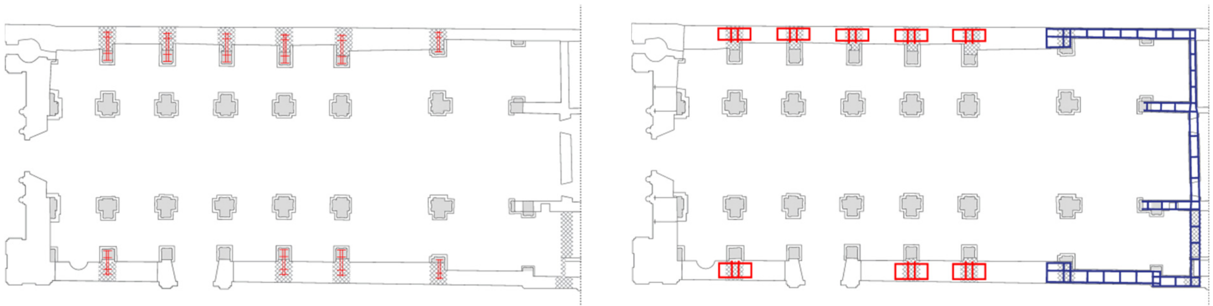
(a) (b)

Figure 1: Ica Cathedral. (a) Eastern front façade; (b) overall structural scheme [5]

DESCRIPTION OF THE PROPOSED STRENGTHENING

Based on the criteria recommended by [3], strengthening measures were kept to the minimum necessary to guarantee safety, durability and the least damage to the historic fabric. Other factors influencing the design methodology adopted here included the state of the art research being performed by GCI and the Pontifical Catholic University of Peru (PUCP) on traditional, low tech yet highly effective methods that have been historically used for the retrofitting Peruvian structures. A brief overview of the proposed strengthening actions is presented in this paper, more details can be found in [7].

A new brick masonry wall was proposed at the north-western corner of the structure, connected to the adjacent walls by interlocking the new brick masonry with existing masonry. The strengthening recommendations also included new brick masonry columns at the location of every connection between the internal timber structure and the masonry envelope along the lateral walls. Timber anchoring systems were proposed to be embedded in these new brick masonry columns to improve the connection between the two sub-structures. In general, each timber anchoring system is composed of keys and ties connected by means of half-lap joints with nails, and has four levels along the height of the lateral walls (Figure 2). It should be noticed that the anchoring systems at the uppermost level have also vertical keys in order to improve the resisting mechanism. A U-shape timber collar beam was proposed to be located at the back of the structure surrounding the altar and the chapels with a frame composed of timber elements joined with nailed half-lap connections (Figure 2). Four steel anchoring systems were proposed to improve the connection between the front façade and the internal timber structure. Each anchoring system consists of: (1) a square steel plate with stiffening elements; (2) a steel tie passing throughout the whole thickness of the front façade; (3) system of steel profiles connected to the timber structure by means of plates and bolts.



(a) (b)

Figure 2: Timber strengthening. (a) Anchors at the lower levels (red); (b) anchors at the upper level (red) and the collar beam (blue)

MECHANICAL AND MATERIAL CHARACTERIZATION

Masonry

Nonlinear behavior of masonry was simulated in the numerical models by using the *Total Strain Rotating Crack* (TSCR) material model, which allowed to specify compressive and tensile softening behavior [8]. The material properties of existing and new masonry – i.e. rubble stone, fired brick, adobe and new fired brick – were derived from bibliographic resources and national technical building standards, considering also the results of the experimental campaign performed by PUCP [7, 9]. It should be mentioned that the moduli of elasticity (MOE) for the existing typologies of masonries which were obtained from the preliminary model updating carried out for the masonry envelope alone were assumed in this numerical model. A summary of the material properties adopted for the different typologies of masonry is shown in Table 1.

Table 1: Summary of material properties for masonry

Properties	**Adobe**	**Fired Brick**	**Rubble Stone**	**New Fired Brick**
Specific weight (kN/m^3)	19	19	19	19
Modulus of Elasticity (MPa)	98 (220)*	340 (850)*	690 (720)*	1200
Poisson's ratio (–)	0.20	0.20	0.20	0.20
Compressive strength (MPa)	0.46	1.70	1.00	6.00
Tensile strength (MPa)	0.05	0.10	0.06	0.25
Fracture energy (compression) (N/mm)	1.00	3.50	1.50	12.40
Fracture energy (tension) (N/mm)	0.01	0.01	0.01	0.02
*First estimate of MOE vs value obtained from model updating (in parentheses)				

Timber

Isotropic homogeneous and linear behavior was assumed for the elements composing the internal timber frame and the upper part of the bell towers in the numerical models. The density and the MOE of existing timber – i.e. *huarango*, *cedar* and *sapele* – were assumed on the basis of the results obtained from the experimental campaign carried out by PUCP. As regards the internal timber structure, it should be mentioned that an equivalent specific weight was assigned to most of the elements in the numerical model to take into account the weight of the *quincha* covering layers, as discussed in [6]. The strengthening timber elements were initially assumed to be made of *tornillo*, for which the density and the mean value of MOE were assumed according to the recommended values provided for the corresponding structural wood class (CLASS C) in the Peruvian Code [10]. A summary of the material properties adopted for the different typologies of timber is shown in Table 2. Isotropic homogeneous and linear behavior was assumed for all the strengthening timber elements with the exception of the connecting elements modelled by using *enhanced truss* elements for which Von Mises criterion was used. In order to apply the von Mises criterion to these *enhanced truss* elements, and then perform the verifications, the expected mean values of the load-carrying capacities were calculated for *tornillo* applying the probabilistic model code developed by the Joint Committee on Structural Safety (JCSS) [11]. Further details can be found in [7].

Table 2: Summary of material properties for timber

Properties	Huarango	Cedar	Sapele	Tornillo
Density (kg/m^3)	1040	380	490	550
Modulus of Elasticity (MPa)	16900	9380	8610	8826
Poisson's ratio (–)	0.30	0.30	0.30	0.30

SAFETY ASSESSMENT IN ITS CURRENT CONDITION

Preliminary analysis

As an initial step towards understanding the global behavior of the structure, the timber frame structure and masonry envelope were first analysed separately. In this section, the main results of this study are presented, see [6] for further details.

Checks for compliance with the criteria specified by Eurocode 5 were carried out for the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) performing global and local verifications on the timber members and connections composing a representative bay of the cathedral. In particular, the beams at the top of the lunettes and their mortise and tenon connections were not verified for ULS under earthquake load combination. The structural performance of the whole timber structure was also evaluated considering vertical and horizontal actions and the transept was identified as the most vulnerable part. Moreover, advanced structural analyses were performed on the FE model of the masonry envelope alone. Eigenvalue analysis was performed in order to predict the dominant modes and the FE model was calibrated on the basis of the results derived from the dynamic identification tests performed by the

University of Minho. Nonlinear static (pushover) analyses were carried out in order to investigate the seismic behavior of the masonry envelope assuming mass proportional lateral loading. The most relevant failure mechanisms were identified as the out-of-plane mechanisms of the front façade including the two bell towers and the northern lateral wall, with associated values of lateral load-carrying capacities of 0.36g and 0.22g respectively.

Definition of the model

On the basis of the assumption made for the partial models of the two sub-structures, a 3D FE model of the entire structure of Ica Cathedral was created in Midas FX+ for DIANA [8]. *3D isoparametric solid linear* four-noded elements were used to model the masonry structure as well as the bell towers, while the internal timber frame was modelled by using *class–I beam* elements with shear deformation. The effect of the cloister adjacent to the building was included in the numerical model by using *one-noded translation spring dashpot* elements along the southern lateral wall. As regards the boundary conditions, all nodes were fixed at the foundation of the masonry envelope, while the base of the posts composing the timber structure was pinned. Full connectivity was assumed between intersecting walls, horizontal layers of different masonries and the timber members. The connections existing between the two sub-structures were assumed on the basis of the available information, as discussed in [6]. The connecting elements were modeled by using *class–I beam* elements and merging their nodes with those of *3D isoparametric solid linear* elements.

Structural analysis

Pushover analyses under mass proportional lateral loading were carried out in order to evaluate the seismic capacity and the failure mechanisms of the structure. The maximum lateral load-carrying capacity obtained from the pushover analysis in the XX– direction was 0.45g, corresponding to the out-of-plane failure of both the front façade and the bell towers. When the lateral load was applied in the YY– direction, the maximum lateral load-carrying capacity was calculated to be a value of 0.28g and the failure mechanism consisted of the out-of-plane failure of the northern lateral wall. Figure 3 shows the load-displacement diagrams obtained for the combined model (MC) and the calibrated model of the 'only masonry' envelope (ME) under lateral loading in the XX– and YY– directions. The obtained results showed that the interaction between the two sub-structures significantly influences the seismic behavior of the entire structure: in both directions, the combined model presented an increase of about 25% in lateral load-carrying capacity. However, it should be noted that the seismic capacity of even the combined model calculated when the lateral load was applied in the YY– direction was found to be considerably lower than 0.45g, which corresponds to the peak ground acceleration (PGA) recommended by the Peruvian Code for the region of Ica.

Nonlinear dynamic (time-history) analysis was performed on the combined model in order to: (1) study its dynamic response; (2) validate it with respect to damage observed in-situ; (3) compare with results obtained from pushover analyses. Rayleigh damping was adopted and the associated parameters were calculated considering all the natural modes of vibration until 80% mass participation was reached (with a damping ratio of 3%). Two artificial accelerograms, compatible with the elastic response spectrum provided by the Peruvian Code, were applied in the principal orthogonal directions of the numerical model, while explicit time step integration was performed using the Hilber-Hughes-Taylor method. Severe damage and loss of its loading-unloading capacity were observed for the model at the end of the applied loading. Additionally, a better correlation was observed between the damage observed in-situ and from the numerical model as compared to pushover analyses. Further discussion on the results can be found in [6].

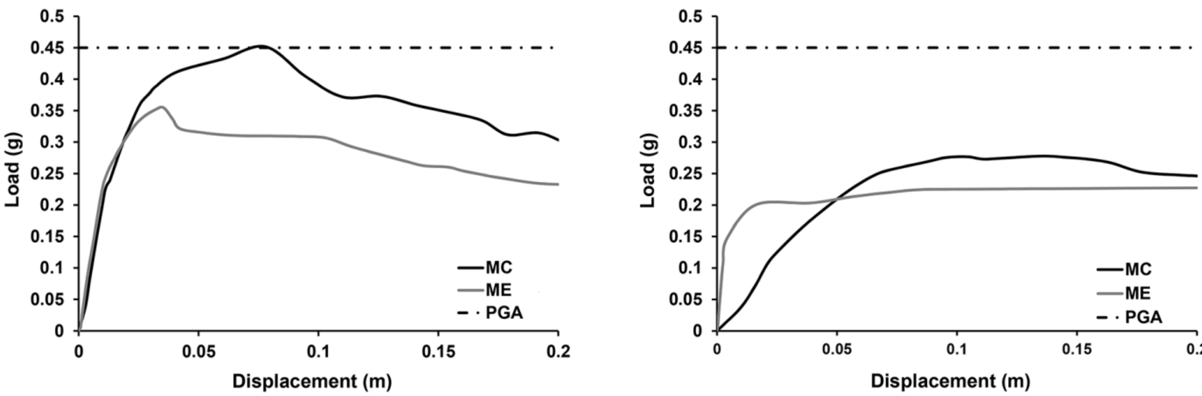


Figure 3: Load-displacement diagrams obtained for the model of only the masonry envelope (ME) and the combined model (CM). (a) XX– direction; (b) YY– direction

SAFETY ASSESSMENT IN STRENGTHENED CONDITION

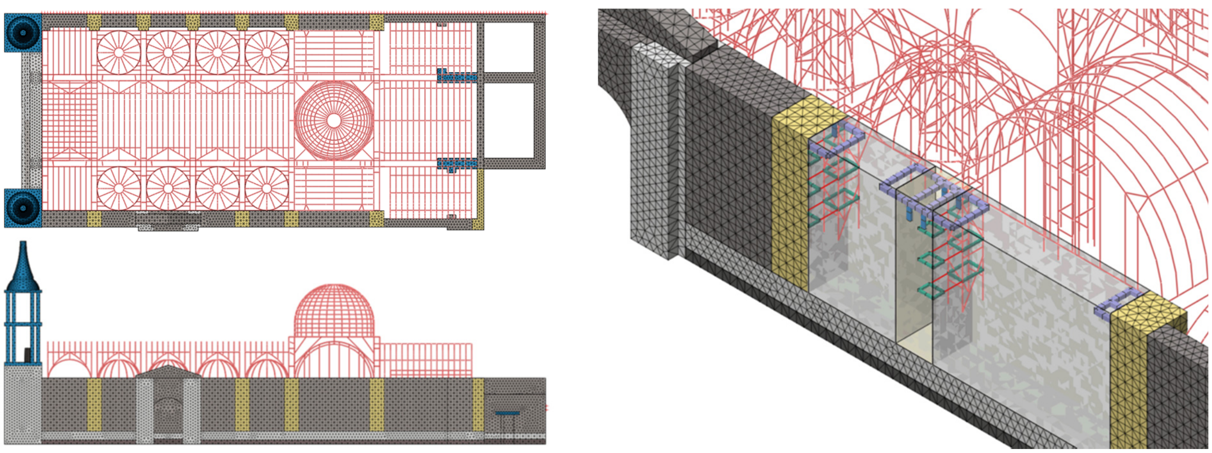
Preliminary analysis

A stronger connection was provided between the two-substructures by means of the strengthening. Consequently, the modeling hypotheses considered for the connections could have had a major influence on structural behavior. An initial study was carried out in order to have a better sensitivity regarding the modeling of the connecting elements between the two sub-structures. The main outcomes of this study are presented in this paper, more details can be found in [7].

The previous model of the structure not including the strengthening was compared to a model that differed only by the modeling of these connecting elements: while they were modeled by using *class-I beam* elements in the previous model, *enhanced truss* elements were used in the alternative one. Pushover analyses were performed by applying a mass proportional lateral loading in the XX– and YY– directions. When the lateral load was applied in the YY– direction, no significant difference was observed since these connecting elements worked structurally as trusses in both the models, not being dependent on the elements used to model them. On the other hand, the values of initial stiffness and capacity obtained when the load was applied to the model with *enhanced truss* elements in the XX– direction were lower than those observed for the model with *class-I beam* elements. In fact, assuming no shear resistance for these connections, the longitudinal movement of the internal timber structure was allowed, and the façade took the maximum possible lateral effect provided by it. Therefore, the updated version of the combined model using *enhanced truss* elements was assumed to provide a more conservative scenario, and is referred in this paper as the unstrengthened model (UM).

Definition of the model

On the basis of the assumptions made for the previous model and the proposed remedial actions, the numerical model of the strengthened structure was created in Midas FX+ for DIANA [8], as shown in Figure 4. The timber anchoring systems as well as the elements composing the collar beam were modelled by using c*lass–I beam* elements which were fully embedded in the masonry with perfect bond. *Flat shell* elements were used to model the steel plates of the anchoring system on the front façade. On the basis of these results obtained from the preliminary analysis, the elements connecting the *fully embedded class–I beam* elements and the *class–I beam* elements modeling the internal timber structure were modelled by using *enhanced truss* elements.



(a) (b)

Figure 4: Strengthened model. (a) FE mesh; (b) detail of the embedded elements

Structural analysis

The efficiency of the strengthening proposal was mainly evaluated by performing mass proportional pushover analyses on the strengthened model. The results were compared to those obtained for the unstrengthened model and the behavior of the strengthened model was particularly investigated for a lateral load equal to the recommended PGA or the ultimate state. Figure 5 shows the load-displacement diagrams obtained for the unstrengthened and strengthened models under lateral loading in the two considered directions. While the maximum lateral load-carrying capacity that could be applied to the unstrengthened model in the XX–direction was 0.39g, a maximum capacity of 0.45g was calculated for the strengthened model. Compared to the out-of-plane mechanism of the front façade and the bell towers observed for the unstrengthened model, the failure mechanism of the strengthened model was identified as the out-of-plane mechanism of only the southern bell tower. Under the lateral load in the YY–

direction, the maximum lateral load that could be applied to the unstrengthened model was 0.25g and the failure mechanism consisted of the out-of-plane failure of the north-western corner. When a lateral load of 0.45g was applied to the strengthened model in the YY– direction, though a progression of a flexural cracks could be observed in the north-western corner for the strengthened model, no decrease in lateral load carrying capacity could be seen.

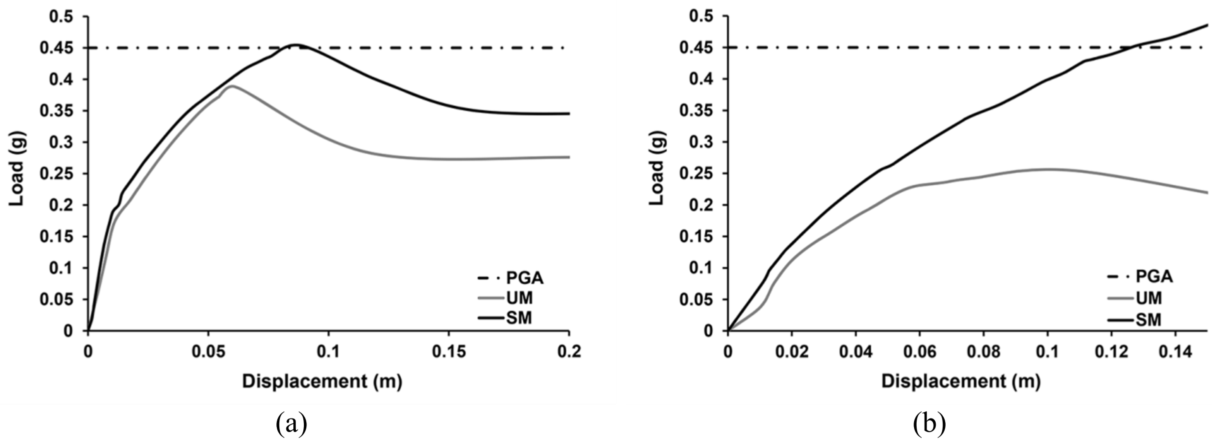


Figure 5: Load-displacement diagrams obtained for the unstrengthened model (UM) and the strengthened model (SM). (a) XX– direction; (b) YY– direction

Verifications of all the timber strengthening elements were carried out to validate the assumption made on their linear elastic behavior considering the internal forces which occurred as the strengthened model was subjected to a lateral load equal to the PGA. The obtained results showed that all the strengthening timber elements where verified when the lateral load of 0.45g was applied in the XX– direction. On the other hand, when this lateral load was applied in the other direction, the elements of the collar beam were subjected to tensile axial and shear forces higher than the corresponding capacities. Therefore, a higher wood structural class (Class B) was adopted for the strengthening timber elements. Although no relevant difference was observed in terms of load-displacement diagrams and internal forces, the load-carrying capacities obtained for a wood species classified as Class B were much higher than those obtained considering Class C and the verifications were satisfied for all the strengthening timber elements, in both the directions.

CONCLUSION

Using the case study of Ica Cathedral, a complex earthen structure constructed by applying *quincha* technique, this paper presents the application of a multi-disciplinary approach to preserve and conserve similar historical constructions. On the basis of historical research, condition surveys, experimental and in-situ test campaigns, several numerical models were created and advanced structural analysis was performed to fully understand the structural behavior of the existing structure. The safety assessment in its current conditions pointed out the effective need for strengthening the structure and allowed identification of the most vulnerable regions of the structure. Therefore, remedial measures were proposed to guarantee the safety and the durability of the structure, guaranteeing minimal, reversible and compatible interventions. The strengthening proposal included: (1) replacement of existing masonry with new brick masonry in selected parts of the structure; (2) timber anchoring systems; (3) a timber collar beam and (4) steel anchoring systems. The results obtained from the safety assessment carried out on the strengthened numerical model showed a reduction of the out-of-plane vulnerabilities and an increase in seismic capacity – which is higher than the PGA required for new buildings in the region of Ica.

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/ECMEST/ 2777/2014.

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