



Structural Performance and Seismic Vulnerability of Adobe Historical Constructions. The Kuño Tambo Case Study.

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Summary

The current study focuses on the structural assessment and seismic vulnerability of adobe historical constructions. Under the scope of reproducing the existing structural damage, the importance of retrofitting measures is signified.

Adobe buildings, due to their low mechanical properties and overturning resistance, are subject to early structural damage, such as cracking, separation of structural elements and, possibly, collapse in areas of high seismic hazard. Under the auspices of the Getty Conservation Institute (GCI) and the ongoing Seismic Retrofitting Project, the current paper, aims at the assessment of the structural performance and seismic safety of the Church of Kuño Tambo, a religious adobe structure of the 17th century, in Cusco region, in Peru.

The inspection and diagnosis involved sonic testing and damage mapping, while conducted ambient vibration tests revealed the modal response of the structure and served to validate the built 3D FE model. Through nonlinear static and pushover analyses, the seismic vulnerability was defined, together with the necessity of retrofitting measures.

Introduction

Adobe is one of the oldest and most widely used natural materials, which accounts for a significant part of the built heritage and is associated with early building techniques, widely dispersed material availability and low-cost construction.

In general, the overall seismic performance of masonry buildings is highly related to the level of redistribution of seismic loads between vertical elements, enhanced by stiff floors, ring beams and joists [1]. Adobe historic buildings, often lack this level of confinement, and the corresponding structural damage is composed of in-plane failure in parts of the skeleton (rocking, sliding, diagonal tension and toe crushing), as well as out-of-plane, under flexural hinge lines, formed usually at the base or on intermediate heights [1, 2]. Often, aspects, such as the lack of maintenance and adequate retrofitting techniques, further contribute to the loss of historic fabric [3].

The current paper discusses the structural performance and seismic assessment of the Church of Santiago Apóstol de Kuño Tambo, in Acomayo, Cusco, in Peru and is part of the Seismic Retrofitting Project of the Getty Conservation Institute (GCI). The correlation with the existing structural damage and capacity were obtained through nonlinear pushover analyses, based on a macro-modeling approach, with a total rotating strain crack model, of compressive and tensile softening behavior, incorporating fracture energy [4]. The uncertainty of confinement provided by the system of anchored ties was accounted, by conducting analyses with no ties and with ties anchored to the external part of the lateral walls. The assigned material properties were retrieved from the experimental campaign provided by GCI, the onsite testing campaign, conducted by the University of Minho in May 2015, and literature.

1. DESCRIPTION OF THE BUILDING AND DAMAGE IDENTIFICATION

The church of Kuño Tambo is a religious structure of the 17th century (1681), representative of churches built in the Andes, during the period of the Spanish Viceroyalty. Its historic significance lies in the limited number of past interventions, and the mural paintings, present in the interior finishes (Fig. 1). It consists of a single nave, leading to an elevated Presbytery and Altar, with an adjacent sacristy and baptistery (Fig. 2). The walls and buttresses are of adobe masonry, with a single gable timber roof and a roof cover composed of canes, a layer of mud and straw, and clay roof tiles. The structure is built on a base course plinth of rubble stone masonry with mud mortar, over a sloping natural rock, with varying layers of compacted clay.



Fig. 1 Kuño Tambo church: (a) aerial view; (b) 3D CAD model.

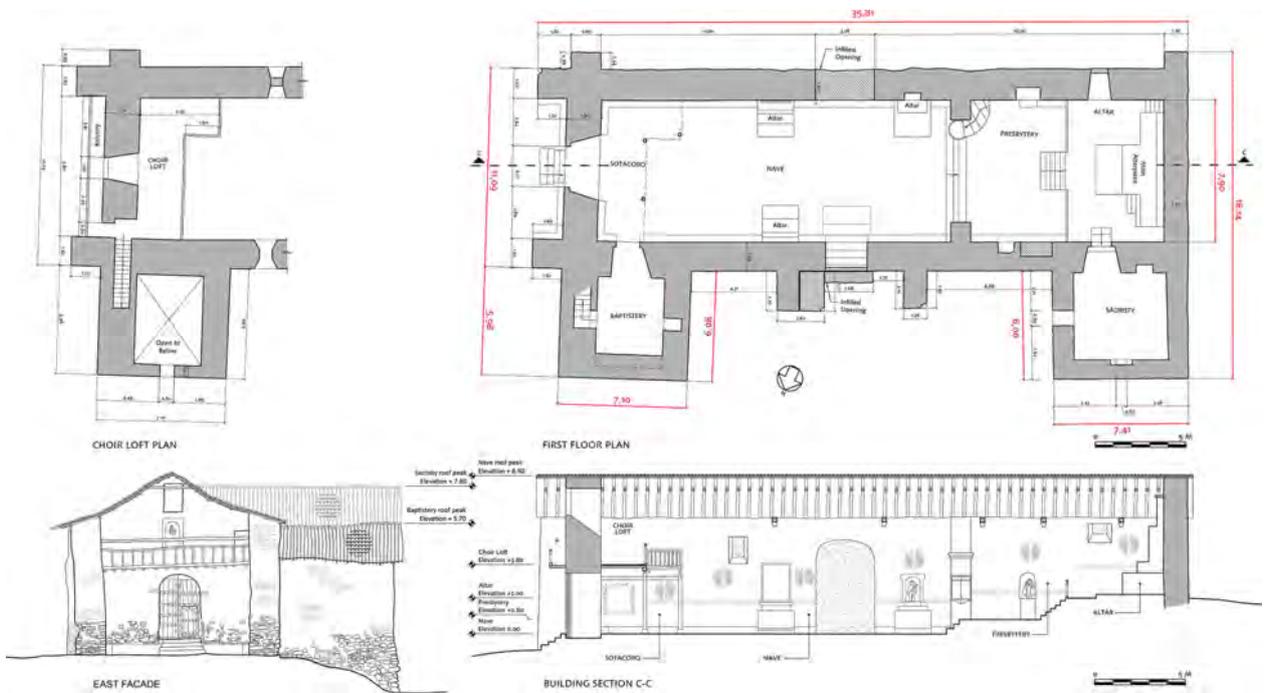


Fig. 2. Architectural plans and elevations (North indication in main plan) [3].

Due to the sloping ground, the base course rubble stone masonry varies in height typically between 1.2 and 1.5 m, with extremes of 3.5 m at the northeast corner. The lateral nave walls have a thickness of 1.6 to 1.9 m and a maximum height of 6.6 m, from ground level.

The east and west façade have a thickness of 1.9 m and 1.3 m respectively, and their height, from ground level to the gable ends, is 8.7 m. For the baptistery and sacristy walls, the thickness varies between 0.65 and 2.0 m, with the east wall of the baptistery having a difference in thickness, due to the presence of the choir loft staircase. Buttresses of adobe masonry are placed on either side of the currently in filled opening, in the north lateral wall. In total six timber ties run along the span of the nave and altar, at the eave level of the lateral walls, placed on top of embedded corbels and in between timber wall plates. Yet, the whole system is discontinuous and the level of confinement is questionable.

The damage configuration, is highly influenced by the low mechanical properties of adobe masonry, the poor level of confinement of structural parts, and the high level of exposure of exterior surfaces to erosion, from improper drainage and maintenance. The east façade shows signs of discontinuity with the lateral walls, with evident vertical cracks at both ends. The south lateral wall exhibits outward displacements, loss of mortar and units, while the base course has evident signs of erosion and deterioration in many exterior parts, with loose stones and missing mortar from joints.

The baptistery exhibits several vertical cracks, located in the middle span of the sidewalls and at the east corner of the gable wall, which appear to be related to seismic action or settlements, triggered by the difference in the thickness. The crack at the corner is characteristic of a separation mechanism of orthogonal walls, from pounding during earthquake events or soil settlements [2]. The presence of damage in all the facades is presented in Fig. 3 [5].



Fig. 3 Damage maps in each facade [5].

2. IN SITU TESTING AND FE MODEL VALIDATION

The material properties were determined from bibliographic resources, national technical building standards [6, 7, 8], considering also the results of the experimental campaign, conducted for the SR Project [9], by the Pontificia Universidad Católica del Perú (PUCP) in 2011 and 2012, and the in situ sonic testing in May 2015, by the University of Minho [10]. The compressive strength of adobe masonry, was accounted from compression tests on adobe wallets, taken from case study buildings in the center of Lima and is equal to 0.45MPa [9]. For the modulus of elasticity E , which is associated with the compressive strength f_c by $E_m=af_c$, a value of a closer to the lower bound specified by Tomazevic [11] is chosen, which corresponds to a Young's Modulus for the adobe masonry of 100 MPa [12].

According to the Mohr-Coulomb yield condition, the tensile strength of adobe masonry, with the cohesion c , equal to 0.0445 MPa and the internal friction angle θ , equal to 0.540 rad, values obtained from shear compression tests on adobe triplets [13, 14], is equal to 0.05 MPa [8, 15], in agreement with literature, even if it is much questionable if the present calculation provides a good measure of the tensile strength of the masonry composite.

Regarding the rubble stone base course, the mechanical properties were specified from reference values [16]. The physical nonlinear compressive and tensile behavior of masonry is described through a total strain rotating crack model [11, 17], and specified values of fracture energy G_f for tension and G_c for compression [18]. The elastic mechanical properties of timber ties, wall plates and lintels are determined from characterization tests in two case study buildings [9], with the highest values assigned to the timber lintels, in order to avoid excessive concentration of stresses in the proximity of the openings.

During the experimental in situ campaign in May 2015, direct and indirect sonic tests were conducted, regarding one indirect setup in the rubble stone masonry, and two indirect and one direct setups for the adobe masonry, in specified locations [10]. The sonic tests in the adobe masonry exhibit good correlation, whereas the values for the stone masonry are more disperse, mainly due to the irregular bond. The implemented mechanical properties, and the recommended fracture energy values [19], for the adobe and rubble stone masonry are presented in Table 1.

Table 1. Mechanical properties of structural elements.

Mechanical properties		Adobe masonry	Rubble stone masonry	Timber	
				Lower	Higher
Compressive strength	f_c (MPa)	0.45	0.60	33.4 ⁵ / 4.7 ⁶	38.1 ⁵ / 6.3 ⁶
Modulus of elasticity	E (MPa)	100 ¹ / 270 ²	300 ¹ / 1570 ²	7830	10680
Poisson's ratio	ν (-)	0.2	0.2	0.2	0.2
Tensile strength	f_t (MPa)	0.05	0.06	47.8 ⁵ / 1.6 ⁶	64.1 ⁵ / 1.8 ⁶
Fracture energy Mode I (tension)	G_f (N/mm)	0.01	0.01	-	-
Fracture energy (compression)	G_c (N/mm)	1.0	1.5	-	-
Specific weight	ρ (kN/m ³)	19 ³	19 ⁴	3.9	4.8

¹ Literature review

² Sonic tests [18]

³ Vecchi, S.S. (2009) Peru

⁴ Table 11.D.1 of OPCM 3431, 2005 [19]

⁵ Parallel to grain [11]

⁶ Perpendicular to grain [11]

A 3D FE model was built in Midas FX+ Version 3.3.0 for DIANA software (Fig. 4). The base connection was modelled as pinned, while all intersecting adobe walls and the connection between the plinth and adobe were modelled with full connectivity. The configuration of the anchoring consists of a combination of three to four single tie beams of 0.6 m length, fixed in the exterior surface of the walls. The tie trusses, of 20 cm in diameter, share no bonding along their embedment in the adobe walls. This simulation takes into account the capacity of the adobe walls under compression, under lateral loads.

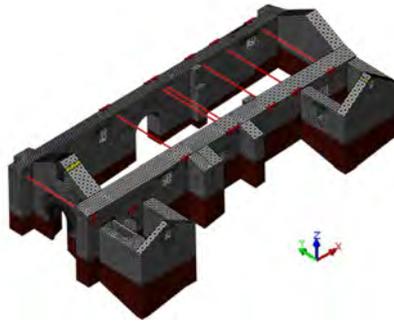


Fig. 4 FE model in 3D view.

The lateral roof thrust, is assigned as a uniform distributed load, with a vertical and horizontal component. The floor level is configured in three elevations, reaching a clear height of 1.6 m in the north part and corresponds to a significant volume of compacted clay. The counteracting earth pressures are assigned as a distributed surface load in three zones, approximating the triangular distribution of the Coulomb earth pressure [19].

The dynamic behavior of the church of Kuño Tambo, as identified from ambient tests conducted in May 2015 [10], is characterized by out-of-plane bending modes, in single and second order curvatures, for the lateral and gable walls of the nave. These configurations provide feedback on the low level of connectivity at the corners, approximating true cracks.

Experimental mode 1 (1.59 Hz) consists of independent orthogonal bending modes, of single curvature, for the nave walls, configured in an out-of-phase symmetry in both axes, with high peaks and similar amplitudes, clearly due to similar stiffness, mass and damping properties (Fig. 5a). Due to the presence of damage, an evident decrease of the natural frequencies and an alteration of the mode shape vectors are found, in comparison to the results of modal analyses, with total continuity in the wall connections, and no ties present.



Fig. 5 Mode shapes from dynamic tests, of the first three modes, with interpolation [10].

As far as the modelling is concerned, the nonlinear behavior of the interfaces, representing the cracks, as well as the elastic behavior of the soil-structure interface, would introduce many uncertainties in the numerical model and therefore they were not included in the

nonlinear analysis. A homogeneous modelling approach was followed, with the structure pinned at base, a decision found better suited for large-scale models, which allows discussing the current condition.

3. PUSHOVER ANALYSIS

The global structure was first subjected to increasing dead loads up to the full gravity. The nonlinear static analysis, revealed lateral displacements in order of millimeters and almost no cracking. Here, two models are presented; Model GM1, a more conservative one, with material properties taken from literature and Model GM2, with material properties retrieved from the in-situ testing campaign.

Regarding the seismic effects, the progression of tensile and compressive damage and the corresponding lateral capacity, were evaluated by applying a mass proportional lateral load. In total, four pushover analyses were conducted, in the x-x and y-y main axes, both in positive and negative directions. Additionally, the effect of the lateral restraint due to timber ties, was investigated by two modelling approaches: one with the presence of timber ties, hinged on a system of timber anchors; and the other without any lateral restraint. Load-displacement curves and damage patterns from load steps near collapse, correlated with the documented damage, are presented in Fig. 6 and Fig. 7.

Regarding the pushover analysis in the positive x-x direction, the minimum lateral load applied is 0.22g for Model GM1_{no ties} (Fig. 6a). The failure mechanism consists of vertical separation cracks, along the corners, in the west part of the church, with horizontal hinge lines at the bottom of the base course foundation. Model GM2 is stiffer and the lateral capacity is around 30% higher than Model GM1, with the ties not offering any strengthening in the longitudinal direction.

The structure experiences the lower lateral capacity in the positive y-y direction, equal to 0.19g (GM1_{no ties}) (Fig. 6b). With the presence of ties, the obtained capacity values are 47% and 37% higher than the ones without ties, for models GM1 and GM2 respectively. Regarding the ties, are all under tension and their ultimate capacity is specified from the pull-out failure of their anchored connection. From tests on timber ties and wood key anchors, on adobe specimens of various thicknesses, conducted at PUCP [9], and given the thickness of the lateral walls, the maximum pull-out force is 96 kN, but partial safety factors should be considered. For ties close to the corners, the ultimate axial force is reached at the early stages of post-peak behavior. Due to transversal restraint, interfaces between the east lateral wall, the baptistery, the buttresses and the sacristy suffer extensive tensile damage and are almost separated.

In the negative x-x direction, the failure mode consists mainly of the out-of-plane overturning of the east façade, together with adjacent fragmented parts of the lateral walls, as shown in Fig. 6c and Fig. 7. The minimum obtained lateral capacity is 0.28g in the less stiff model (GM1_{no ties}). With effective ties, the structure presents higher capacity and the ties appear to provide some restraint to the lateral walls and limit flexural out-of-plane deflections. Nevertheless, this is considered a local effect.

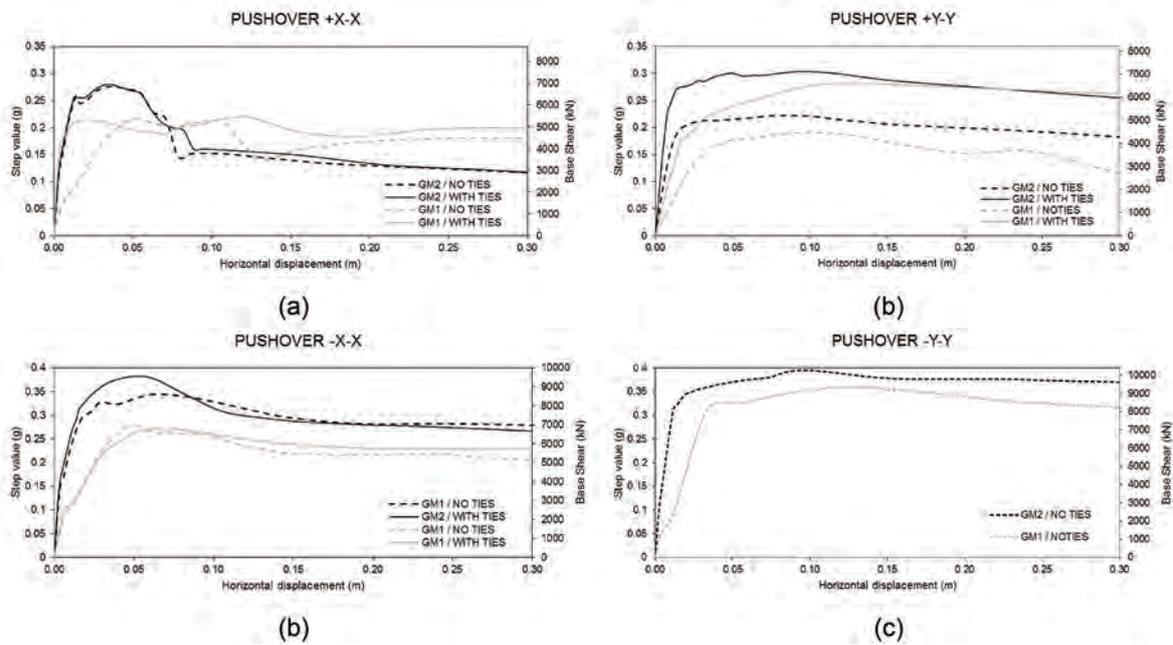


Fig. 6. Response of structure in all directions and axes, for Models GM1 and GM2.

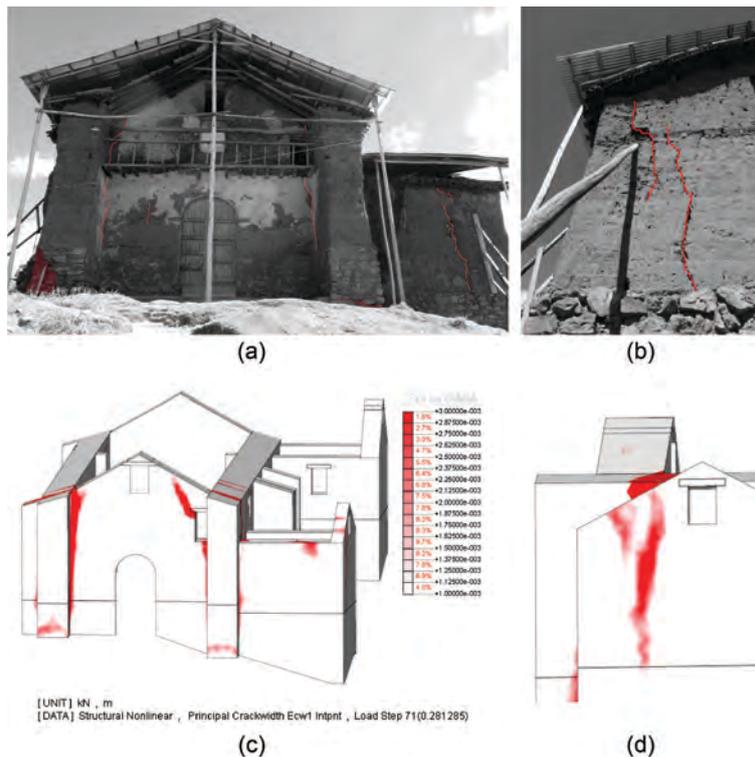


Fig. 7. Crack pattern correlation: (a) east façade; (b) gable wall of baptistery; (c), (d) distribution of principal crack width at load step near collapse (Model GM2_{no ties}).

Lastly, from the results of the pushover analysis in the negative direction of the y-y axis, the structure presents the highest lateral capacity, in this case 0.36g and 0.39g (Fig. 6d), for Models GM1 and GM2 respectively, with the abutments on the north part contributing much more to the overall stiffness. The tensile damage is mostly concentrated in the sacristy, which fails early at 0.16g and 0.25g for models GM1 and GM2 respectively. Yet, no such damage exists on current state. Regarding the behavior of the tie beams, their axial force is gradually reversing from tension to compression, due to the difference of

stiffness, between the lateral walls. Nevertheless, in order for the ties to work in tension and compression, anchor keys at both sides of the walls are needed or an internal timber truss. Since no such connections were identified, the behavior of the ties is questionable.

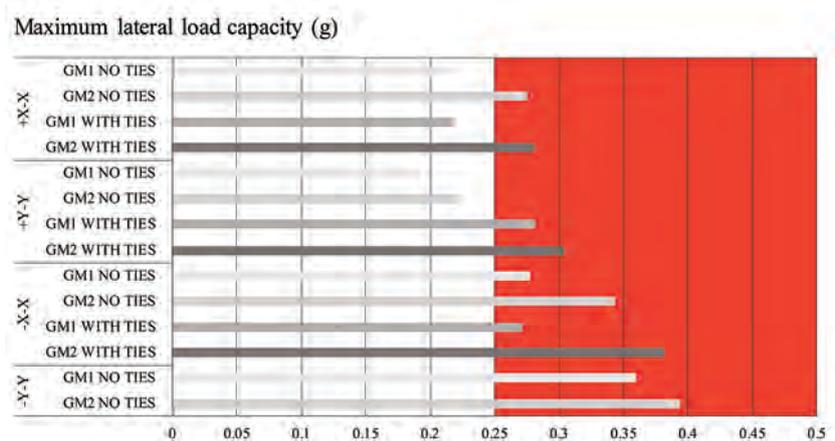
CONCLUSIONS

The seismic vulnerability of the Kuño Tambo Church was studied, aiming at reproducing the existing damage patterns and assessing safety under current conditions. Tensile crack patterns were obtained and correlated with the observed ones, indicating that the current damage have been induced partly by past seismic actions and settlements. In-situ inspection, together with dynamic and sonic testing, offered substantial information regarding the structural behavior, under the current damage.

The system of tie beams and their connection with the adobe walls of the nave can have a significant effect on the transversal behavior. Under the current condition, given the visual inspection and the results of the dynamic identification tests, the system of ties is considered inactive. Thus, the model with no ties is considered representative of the structure at current condition.

According to the pushover analyses, strengthening is needed in multiple directions, since the overall capacity does not reach the peak ground acceleration for the Cusco region (0.25g) [20], as depicted in Table 2. One of the structural deficiencies of the church is the large asymmetric distribution of stiffness in the transverse direction of the nave. The failure mode with the lowest capacity, the out-of-plane overturning of the south lateral wall, controls the response, and the obtained minimum value is 0.19g, for the less stiff model. The possible reintroduction of the buttresses in the east part will increase the stiffness and global capacity.

Table 2. Maximum lateral capacity in each direction, in Models GM1 and GM2.



The ties have a significant effect to the lateral capacity in the positive y-y direction. Yet, the capacity of the connections is considered low and given the dynamic character of the seismic forces, their performance is questionable. In order for the ties to work both in tension and in compression, anchor keys at both sides of the walls are needed or an internal timber truss is required. This would be a required practice in the definition of strengthening measures. New ties should also be added to reduce the large spacing between existing ones, as well as in the top of the baptistery and sacristy sidewalls, in order to limit out-of-plane collapse modes. The connectivity in the intersections between

walls can also be enhanced with the use of corner keys, placed in intersections between orthogonal walls.

Lastly, several areas of the building are in need of stabilization actions, due to mortar joint loss and loss of adobe and stone units. Immediate action is recommended to prevent damage increase and reduce the risk of fabric loss.

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