MITIGATION OF SEISMIC VULNERABILITY IN EARTHEN HISTORIC STRUCTURES WITH TRADITIONAL STRENGTHENING TECHNIQUES.

Georgios KARANIKOLOUDIS¹, Paulo B. LOURENÇO², Daniel TORREALVA³, Claudia CANCINO⁴, Kelly WONG⁵

ABSTRACT

Earthen constructions, account for a significant portion of the build heritage. The structural system mainly consists of heavy and thick walls, with low overturning resistance and insufficient connections. In areas of high seismic hazard, traditional strengthening elements are mostly in terms of buttressing and timber framing. Yet, in many historic sites, due to the lack of maintenance and loss of intangible building techniques, strengthening elements are often insufficient, in bad condition or totally absent. Under the auspices of the Getty Conservation Institute (GCI) and the ongoing Seismic Retrofitting Project (SRP), the current paper aims on the mitigation of seismic vulnerability in the Church of Kuño Tamboco, a religious adobe structure of the 17th century, in the Peruvian Andes. From May 2016, after an extensive structural assessment, from the University of Minho, targeted traditional strengthening measures were designed by a team of engineers. The strengthening plan is currently implemented, under the local conservation directorate. A compilation of modelling strategies, from simplified analytical to complex numerical, were incorporated, in order to verify the strengthening plan, while issues of nonlinearity were accounted in every stage. Complimentary results, on the behavior of traditional strengthening elements were provided through an extensive testing campaign, from the Pontificia Universidad Católica del Peru (PUPC). Despite the design verification demands for modern structures, the current framework of strengthening was based on nominal values, thus, avoiding excessive and unnecessary strengthening measures. Besides strengthening in macro-scale terms, a priority was set on increasing durability, regarding material consolidation, stabilization and repair.

Keywords: Earthen historic structures; Traditional strengthening techniques; Seismic vulnerability; Nonlinear pushover analysis; Macro-block limit analysis;

1. INTRODUCTION

Earthen constructions, account for a significant portion of the build heritage and consist mainly of heavy and thick walls, with corresponding high inertia forces. In areas of high seismic hazard, traditional strengthening elements, mostly in terms of buttressing and timber framing, were incorporated in early stages, as part of the structural system. Yet, in many of historic earthen buildings, those systems are often found insufficient, in bad condition or totally absent.

In general, earthen structures, due to their low mechanical properties, poor level of connectivity between adjacent parts, exhibit low overturning capacity. Failure modes involve continuous cracking in elevation, separation of structural elements and, possibly, disproportional out-of-plane failure of

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individual parts, in areas of high seismic hazard. The absence of retrofitting or even the incompatibility in terms of consolidation and strengthening interventions can highly compromise their structural integrity and durability. Also, the lack of maintenance, combined with strong environmental agents, such as entrapped humidity and poor drainage, especially in areas of structural cracks and stress reversals, can escalate degradation rates. In a long-term scale, material properties are affected and global performance can diminish substantially.

Under the auspices of the Getty Conservation Institute (GCI) and the ongoing Seismic Retrofitting Project (SRP), the current paper focuses on the process of strengthening implementation in the Church of Kuño Tambo, a religious adobe structure of the 17th century, in the Peruvian Andes. In-situ inspections and diagnosis; mainly damage mapping, sonic and dynamic identification tests, conducted by the university of Minho in 2015, revealed the structure’s modal response, pathology, some indication on material properties and the level of interaction between structural members. The seismic vulnerability was verified through nonlinear pushover analyses, complemented with the results under a macro-block limit analysis. Furthermore, a study of a more realistic response, from dynamically induced ground motions was performed; i.e. a nonlinear time history analysis, according to the EC8 framework. Conclusive capacity and hysteresis curves revealed the high seismic vulnerability and the necessity of retrofitting measures. The lateral capacity, in many directions, does not comply with the seismic capacity demand of the Peruvian seismic code.

The strengthening and rehabilitation plan for the Church of Kuño Tambo was developed under a collaborative concept, by a team of engineers within the SRP and the staff responsible for implementing the plan; i.e. the Directorate of Conservation of Immovable Cultural Heritage of Cusco. Main strengthening solutions involve buttressing and a compilation of timber bracing systems. Consolidation measures, in damaged and deteriorated parts, were also set in high priority. The total approach accounts for practices and recommendations from several seismic building codes and norms. The main objective was set on achieving a high level of connectivity between transversal and longitudinal structural parts, thus approximating as much as possible a so-called “integral behavior”, under a flexible diaphragm. As well as for the assessment of the current state, a 3D macro-modeling FE approach, with total continuity between parts, was adopted. Timber elements where modelled as embedded beams and as trusses, only when axial deformation was dominant. Nonlinearity in solid parts was accounted by a total strain rotating crack model, with compressive and tensile softening, by means of fracture energy. Ideal plasticity was accounted for timber elements, to limit their contribution in capacity terms, by the Von Mises yield stress criterion.

The strengthening design and implementation were verified through partial and global numerical models, of increased complexity. Complementary results on an early basis were provided through simple analytical tools; i.e. a macro-block limit analysis. Nonlinear capacity curves, together with damage propagation characteristics at post-peak, concluded the adequacy of the strengthening measures, under the local seismic demand. Lastly, an extensive testing campaign on material characterization and capacity of traditional strengthening elements, by the PUPC, complimented the approach.

The basis for the evaluation of the strengthening scheme was to assess the structure’s lateral capacity, under the local seismic demand (PGA). Historic masonry structures, having mostly uniformly distributed mass in elevation, ideally the behavior under a flexible diaphragm and in general, many local modes of excitation, are not prone to spectral amplification. Despite also the design demands for modern structures, the current framework of assessment and strengthening was based on nominal values for the material properties, thus, avoiding excessive and often unnecessary strengthening measures.

2. SEISMIC VULNERABILITY OF EARTHEN HISTORIC STRUCTURES

In earthen historic structures, under the presence of seismic forces, the most common structural damage patterns are depicted in Figure 1. Here, structural damage induced by earthquakes, is mainly the result of high inertia forces, low mechanical properties, low ductility and insufficient diaphragmatic stiffness at the roof level or at intermediate floors. In the absence of buttressing and bracing systems, structural failures are not considered global and are dominated by the out-of-plane rigid body motion of individual parts (Lourenço et al. 2011, Tolles et al. 1996). Given the magnitude of seismic forces, the structural damage can have the following progressive pattern; connections between orthogonal walls are subject
to early separation (Cancino et al 2013); horizontal flexural hinge lines are formed at rotation levels, in the base of walls or at intermediate heights; lateral deformations at the top of walls are continuously increasing and corresponding flexural cracks progress in elevation and thickness; stresses at the compressed part of the rotation plane keep increasing, and further at the softening phase, the material is crushed under compression (toe crushing); crack patterns are united and large structural parts enter disproportional collapse modes at an exponential rate (Tomazevic 1999, NTC 2008, 617 C.S.LL.PP 2008).

With the addition of buttressing and bracing systems, out-of-plane failure modes are more ductile and higher lateral capacity values are reached. Damage patterns can involve only portions of walls, and consist of diagonal bending and shear cracks at the corners and in the wall’s middle span. Other in-plane shear cracks are also formed in the proximity of openings and spandrels, but those are easily repairable. Instead, in-plane horizontal and shear cracks, due to the interaction between structural elements of different stiffness, are extremely intense; i.e. rocking of the main facades in cathedrals and circumscribed by bell towers.

Lastly, the dynamic character of seismic loads, often underestimated, affects durability (Cancino et al 2013, Karanikoloudis et al. 2018). In areas of high stresses or high strain reversals, the material can disintegrate. Micro-cracks, under the presence of strong environmental agents are intensified and partial collapses can occur suddenly long after a seismic event.

3. THE CASE STUDY OF KUNO TAMBO

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. Is considered of high artistic value, due to the wall paintings, present in the interior finishes (Figure 2a). It consists of a single nave, leading to an elevated Presbytery and Altar, with an adjacent sacristy and baptistry. The walls and buttresses are of adobe masonry. 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 roof is a single gable timber roof, with 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. 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 anchoring is not sufficient to provide lateral confinement.

The present damage, as depicted in Figure 2b, appears to be inflicted partly by past earthquakes, also settlements and is amplified under erosion, from improper drainage and maintenance. Large vertical cracks, present in the corner junctions of the east façade, along the entire thickness and elevation, establish the actual discontinuity with the lateral walls. 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. Several vertical cracks, due to pounding and partial sliding of the base are also present in the gable and sidewalls of the baptistery (Tolles et al. 1996, Zanotti 2015).

The dynamic behavior of the church of Kuño Tambo was identified from ambient tests conducted in May 2015, by the University of Minho (Lourenço et al. 2016). For the lateral and gable walls of the nave, it consists mainly of out-of-plane and out-of-phase bending modes, in single and second order curvatures, under low frequencies (Figure 2d). These configurations justify the low level of connectivity at the corners, approximating true cracks and the lack of confinement from the present tie beams.

The material properties were determined, from literature, prior testing campaigns and in-situ sonic testing, from May 2015, conducted by the University of Minho (FEMA 306 1998, EN 1996-1-1 2005, Lourenço et al. 2016). In specific, lower and upper bound values were assigned, moving from a conservative to a more realistic response. The range of material properties and assigned nonlinearity, specified with values of fracture energy Gf for tension and Gc for compression, are depicted in Table 1 (Lourenço 1996).

A 3D finite element (FE) model was built in Midas FX+ Version 3.3.0 for DIANA software, with no damage and full connectivity between parts (Figure 2c). Given the low tensile strength of the materials, cracks can easily arise upon increasing loading. The created FE mesh is composed of 321827 elements.
isoparametric tetrahedral linear elements, with a size from 15cm to 30cm and 4 to 7 elements per thickness (DIANA 2014).

Figure 1. Seismic vulnerability of earthen historic structures. Typical failure mechanisms (note the associated direction of seismic forces) (Lourenço et al. 2017).

According to the pushover nonlinear analyses, under a mass proportional lateral load, damage patterns are well correlated with the present damage. Strengthening is needed in multiple directions, since the overall capacity does not reach the peak ground acceleration for the Cusco region (0.25g), as shown in Figure 3 (NTE-0.30 2016). More information on the assessment of the current state of the church of
Kuño Tambo can be found in (Karanikoloudis et al. 2018).

Table 1. Mechanical properties of structural elements.

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Adobe masonry</th>
<th>Rubble stone masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength ( f_c ) (MPa)</td>
<td>0.45</td>
<td>0.60</td>
</tr>
<tr>
<td>Modulus of elasticity ( E ) (MPa)</td>
<td>100(^1) / 270(^2)</td>
<td>300(^1) / 1570(^2)</td>
</tr>
<tr>
<td>Poisson’s ratio ( \nu ) (-)</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Tensile strength ( f_t ) (MPa)</td>
<td>0.05</td>
<td>0.06</td>
</tr>
<tr>
<td>Fracture energy Mode I (tension) ( G_I ) (N/mm)</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Fracture energy (compression) ( G_c ) (N/mm)</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Specific weight ( \rho ) (kN/m(^3))</td>
<td>19(^3)</td>
<td>19(^4)</td>
</tr>
</tbody>
</table>

1 Literature review
2 Sonic tests (Lourenco et al. 2016)
4 Table 11.D.1 of OPCM 3431, 2005

Figure 2. Kuño Tambo church: (a) aerial view and wall paintings; (b) damage maps in each façade (Zanotti 2015); (c) 3D view of the FE model with base course foundation in brown and adobe masonry in grey (Karanikoloudis et al. 2018); (d) mode shapes from dynamic tests, of the first three modes, with interpolation (Lourenco et al. 2016).
3. PHILOSOPHY OF STRENGTHENING, DESIGN AND IMPLEMENTATION

Any proposed strengthening plan has to be aligned with conservation principles, namely minimal intervention, authenticity and reversibility (ICOMOS 2003). Key issues are also material compatibility, consolidation and sufficient care on connections and structural details. The extent of replacement, addition and consolidation in structural elements should be clarified from damage mapping and structural nonlinear analyses, and limited as possible, in order to preserve the historic fabric.

Overall objectives, in areas of high seismic hazard, are the improvement of lateral resistance and, ideally, the energy dissipation capacity. A transition is needed from local failures to a global structural performance, approximating the behavior under a flexible diaphragm (Lourenco et al. 2011). Compared to modern or even existing structures, historic structures are under a different process of strengthening, design and validation. The use of design and even characteristic values, in resistance and loading, is highly conservative and can often lead to excessive and even unnecessary strengthening measures.

Often, lower partial safety values for the Ultimate Limit State (ULS) are used. In the case of nonlinear structural analysis, the overall response should in any case, be based on average, unreduced values of strength (Tomaževič et al. 2007).

3.1 Traditional strengthening/retrofitting techniques

Traditional strengthening techniques are long proven and present in many historic earthen buildings, though often disregarded in current strengthening practices. Those traditional strengthening systems involve the combination of additional mass and stiffness elements, identified in two specific groups. First, solutions that can strengthen the walls by addition of buttresses. Second, by tying the walls and ensuring their connectivity, by means of timber elements (bond beams - wall plates, anchored ties and corner keys). Timber elements, introduced or reestablished in the existing structural system of masonry can contribute in enhancing substantially the capacity under lateral forces (Tomaževič et al. 2007, Vintzileou 2008). In order for the new system to perform, timber elements need to be confined in masonry and subjected to normal vertical stresses, from the overlapping masonry parts, roof loads and vertical anchoring systems, so that friction or shear action is available (Tomaževič 1999). Many traditional strengthening practices are already incorporated, as parts of guidelines and recommendations in seismic building codes and norms, such as in Peru, Nepal, India, New Zealand (Table 2).
Table 2. List of technical norms and national building codes accounted for on the design and evaluation process of earthen historic structures.

<table>
<thead>
<tr>
<th>Technical Norm</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>Norma E.030 (2016)</td>
<td>Technical norm for the design of earthquake</td>
</tr>
<tr>
<td>RNE E.10 (2006)</td>
<td>resistance of buildings</td>
</tr>
<tr>
<td>Junta del Acuerdo de</td>
<td>Technical norm for design of timber</td>
</tr>
<tr>
<td>Cartagena</td>
<td>elements for structural use</td>
</tr>
<tr>
<td>Norma E.080 (2006)</td>
<td>Guidelines for design of adobe structures</td>
</tr>
<tr>
<td>NBC 204 (1994)</td>
<td>Guidelines for earthquake resistant building</td>
</tr>
<tr>
<td>IS.13827 (1993)</td>
<td>construction in earthen buildings</td>
</tr>
<tr>
<td>NZS 4297:1998</td>
<td>Guidelines for improving earthquake resistance</td>
</tr>
<tr>
<td>NZS 4298:1998</td>
<td>of earthen buildings</td>
</tr>
<tr>
<td>NZS 4298:1998</td>
<td>Engineering design of earth buildings</td>
</tr>
<tr>
<td>NZS 4298:1998</td>
<td>Materials and workmanship for earth</td>
</tr>
<tr>
<td>NZS 4298:1998</td>
<td>buildings</td>
</tr>
</tbody>
</table>

3.2 The use of buttressing and timber frames. The strengthening scheme of Kuño Tambo

The addition or reestablishment of buttresses can efficiently address low out-of-plane capacity and large deflections, in earthen walls of large spans. The design verification of buttressing, in terms of size, number and covering span is easily done with simple analytical tools, such as limit analysis (NBS 204 1994, IS 13827 1993). Here, design guidelines, require a minimum thickness and length, equal to the thickness of the adjoining wall (NTC 2008). For a buttresses to work efficiently way, the connectivity with the adjoining wall has to be ensured, by means of horizontal timber elements, inserted at various elevations, along the interfaces with the existing walls. Interlocking can offer additional shear stiffness, along the interface.

For the church of Kuño Tambo, three new adobe buttresses, with rubble stone base, were designed for the south lateral wall of the nave. From applicable kinematic mechanisms under horizontal equivalent seismic forces, the maximum capacity, for the ULS, is 0.32g, significantly higher than the peak ground acceleration (PGA) demand for the Cusco region (0.25g) (NTE-0.30 2016). Horizontal timber keys are placed in each buttress in three elevations, at heights of 1.50 m, with the middle tie placed at mid height of the adobe masonry, in each buttress. The timber keys progress until half the thickness of the lateral walls and are configured as shown in Figure 4. Half-lapped pinned connections, with leather straps, are chosen from traditional local practices. As a new addition, the buttresses are partly wrapped with geogrid sheets, in order to enhance ductility and prevent disproportional collapse, under seismic loads.

Timber embedded elements can be placed on the top eaves of earthen walls and in the corner junctions, in order to enhance connectivity and deformability; i.e. bond beams and corner keys respectively and can either be diagonal or orthogonal. Both elements are inserted either from the exterior or interior parts of the walls, by removing courses, and are composed by two sets of timber beams, connected transversally with timber blockers. The insertion of corner keys should be done in horizontal planes, in various elevations, mostly involving the upper parts of the walls. According to preexisting crack formation, several processes are also applicable; i.e. partial replacement of material, stitching, repointing and grout injections, with mud-based grouts (Ortega et al. 2016, Silva et al. 2014). For the church of Kuño Tambo, a bond beam and a set of two orthogonal corner keys were inserted in each corner, with the configuration presented in Figure 5. Connections are also half-lapped and pinned, with leather straps. Lastly, a system of tie beams can be designed to offer lateral restraint, mostly to weak unrestrained earthen walls of large spans. The connection is established between parallel walls and can be effective in walls of different stiffness. In order for the ties to work in the event of an earthquake, both in tension and compression, an adequate double anchoring system is needed. More efficient are vertical timber
anchors, attached on the interior and exterior surfaces of the walls. Such a system is implemented for the nave of the church of Kuño Tambo, with eleven tie beams. In order to increase the anchoring and pullout capacity of the tie beams, the latter are connected to the overlying bond beams (Figure 6).

Figure 4. Kuño Tambo church: (a) under current conditions (top) and after strengthening (bottom); (b) horizontal timber keys in buttresses, in typical cross-section; (c) construction phases.

Figure 5. Kuño Tambo church: timber embedded elements in elevation and horizontal planes.

Figure 6. Kuño Tambo church: configuration of tie beams and bond beam, during the design and constructions phase.
3.3 Capacity tests at PUPC

In the design process of a strengthening scheme, for a historic earthen structure, one should have a clear understanding on the interaction and potential failure mechanism of an adobe - timber composite structure. The introduction of a single timber tie anchored in an adobe wall, activates the masonry subjected to a point load. An earthen wall, being the weakest link, is expected to fail first, given the assumption that the timber connections are strong enough to withstand failure. Here, the failure mechanism is the tensile failure of masonry in a plane of about 45°, surrounding the tie-anchor system, known as cone failure (Vinci 2014). In the case of embedded timber frames; i.e. corner keys, the cone covers a wider area. Results from pullout tests on timber ties and orthogonal timber keys, anchored in adobe masonry wallets, of various thicknesses, conducted at PUCP University, show the formation of a cone and justify the above reasoning (Torrealva et al. 2014).

As shown in Figure 7a, a single tie beam (Φ17.8 cm) is anchored externally with a vertical key 55 cm long (Φ10.2 cm) in adobe wallets of varying thickness, within the range 26-83 cm. The maximum axial force taken by the tie, given the cone formation in adobe masonry, is under linear regression with the thickness of the adobe masonry wallet Figure 7b. Yet, in reality, parallel earthen walls, connected with a system of timber ties will behave much differently. The ties will be activated mainly due to the relative stiffness between the lateral walls and cone failure will probably never occur. Instead, a combination of vertical and diagonal cracks at mid height will define failure.

In the case of pullout tests in adobe wallets with embedded orthogonal timber keys, the cone failure is extended around the timber frame. Here, main beam sections are 12.5x12.5cm and the thickness of the wall is 80 cm. As shown in Figure 8, two different testing assemblies replicate the orthogonal timber key, between two transversal adobe walls. In reality, one cone failure will be formed in the part with the weakest connection. Maximum pullout capacity was obtained within the range 27-37 kN.

Figure 7. Experimental results on pullout tests from PUCP: (a) tie beam and anchor of testing assembly and cone failure; (b) maximum pullout test results v. wall thickness and linear regression trend (Torrealva et al. 2014).

Figure 8. Experimental results on pullout tests from PUCP: orthogonal timber corner keys embedded in adobe specimens, testing assemblies and cone failure (Torrealva et al. 2014).
4. ASSESSMENT THROUGH COMPLEX NUMERICAL MODELS

For the Church of Kuño Tambo, the structural performance and seismic assessment of the strengthening scheme was validated through a macro-modeling numerical process. Here, all material properties are based on average, unreduced values (Tomažević et al. 2007). Nonlinearity was accounted for the adobe and base course stone masonry structural parts, with a total strain rotating crack model, incorporating the fracture energy concept (Table 1) (Lourenço 1998). The chosen timber species to be used for the strengthening elements are Eucalyptus Globulus, with strength values of timber class B (RNE E.10 2006). A square section of 20x20 cm$^2$ is chosen for the tie beams, while for the bond beam, horizontal and orthogonal timber keys, elements with 15x15 cm$^2$ section are chosen. The anchors are placed as wedges, in holes made at mid-section of the tie beams. Their upper section 7.6x15.2 cm$^2$ and the lower is 7.6x7.6 cm$^2$. Their length varies from 80 to 90cm. Here, the capacity and ductility of all timber connections is assumed not be compromised and is considered higher than the element’s capacity. Thus, ideal plasticity on the sectional resistance is assigned, with the Von Mises yield stress criterion, under uniaxial tension and compression. Regarding the external bracing of tie beams, the critical upper bound is set for compression, accounting the critical buckling load. In the nodes of intersection between timber elements, all connections are half-lapped and pinned with nails. In order to account for the sectional loss, but not in terms of stiffness, the strength in the entire configuration of the timber elements is reduced by half.

The created FE mesh is composed of 239298 10-noded tetrahedron elements, 14 two-node truss elements, representing the tie beams and 2078 three-noded beam elements, for the embedded timber frame. Due to the addition of mass and stiffness, through the new buttresses and timber frames, the structure presents a stiffer response, with a shift towards higher modes. Out-of-phase modes, representative of the current state are now replaced with global ones, an indication of an ‘integral behavior’. As shown in Figure 9a, the 1$^{st}$ out-of-plane mode for the lateral walls of the nave is 4.41 Hz (eigenvalue analysis), versus 1.59 Hz in the current state (dynamic tests).

Nonlinear pushover analyses, under mass proportional lateral loading, were performed in all principal directions. The overall performance under lateral loading is greatly improved and the redistribution of seismic loads between transverse and longitudinal walls is now working. The capacity chart in Figure 9b summarizes the obtained maximum lateral capacity values and the residual ones, near collapse, of around 20 cm lateral displacement. In all principal directions, the capacity reached is higher than the design requirement from the Peruvian seismic code, with a minimum safety margin of 1.5. For the specific regional seismic demand of 0.25g, in all directions, the structure behaves almost elastically, with no apparent damage. Note that lateral capacity values at peak and near collapse are compared for models at current state and strengthening, both with material properties updated by in-situ sonic tests. In the direction of the weakest capacity (N→S), the strengthening scheme presents a substantial higher capacity of 0.34g, increased by 70%, compared to that of the current state. The change is also evident on the failure mechanism (Figure 10). From the out-of-plane overturning of the entire south wall, both lateral walls are now deforming in out-of-plane bending, while activating the transversal walls. The tensile damage is also more widely spread, with vertical and diagonal cracks, and smaller crack widths.

![Figure 9](image_url)

(a) Mode shape configuration of the 1$^{st}$ translational mode, of the model with strengthening; (b) maximum lateral capacity in each direction, for models with and without strengthening.
Figure 10. The Church of Kuño Tambo: (a) load-displacement diagram, at the top of the south wall, for the model at current and strengthening state (note the seismic demand of 0.25g in red); (b) distribution of maximum principal tensile strains, for 20 cm of lateral displacement, for the model at current and strengthening state.

5. CONCLUSIONS

Given the low mechanical properties, the chronical lack of maintenance and the insufficient bracing elements, earthen historic structures will certainly perform inadequately in a future earthquake event, in the absence of corrective measures. Taking into account prescriptions from national building codes and local practices, a variety of possible traditional strengthening techniques and conservations tactics are applicable, for earthen historic structures.

Under the Seismic Retrofitting Project, of the Getty Conservation Institute, the strengthening scheme for the Church of Kuño Tambo has been developed by a collaborative team, involving engineers, architects and cultural heritage authorities. In order to provide integrity to the structure, external and internal bracing has been prepared, under the concept of an "integral behavior" and a flexible diaphragm. The implementation of new buttresses, together with an embedded system of timber strengthening elements, such as bond beams, anchors and corner keys, connected with tie beams, has improved the performance under lateral loading, increased energy dissipation and allowed the redistribution of seismic loads between transverse and longitudinal walls. The strength in corner junctions has increased substantially and transversal walls have been activated in the out-of-plane response. Thus, the structural behavior is altered, from out-of-plane rigid body motion of individual parts, to a combination of out-of-plane bending (longitudinal walls) and in plane bending and shear (transversal walls). Overall, the retrofitted structure, respects the performance criteria under the seismic local demand, with sufficient levels of repairable damage.

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7. REFERENCES


