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A LOGIC-TREE APPROACH FOR THE SEISMIC DIAGNOSIS OF HISTORIC BUILDINGS: APPLICATION TO ADOBE BUILDINGS IN PERU

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ABSTRACT

The presence paper proposes an approach for the seismic diagnosis of historic buildings based on the evaluation of several variables that influence seismic response in a logic sequence. The variables are related both with the global response of the structure and local vulnerabilities. Different sources of information are used in the assessment, such as historical data, on-site observation, numerical, analytical and experimental results. The diagnosis is organised in a logic-tree, presenting alternative hypotheses of seismic response by means of different paths. A level of confidence is associated to each path. The presentation of the diagnosis in this way allows an expedite comparison of seismic responses between different macroelements, constructions or building types. This approach is applied to the seismic diagnosis of an adobe church in Peru. Past earthquakes, such as the Peru Earthquake of the 15th of August of 2007, have shown that historic adobe constructions are vulnerable to earthquakes, especially if the quality of the construction has been jeopardised by lack of regular maintenance.

INTRODUCTION

Earthen construction in Peru built during the Hispanic Viceroyalty, dating from the 16th century to the early 19th century, made use of traditional and locally available materials, such as earth and timber. These constructions are primarily composed of adobe walls, rubble stone masonry base courses, timber framings, canes, mud and lime plaster. Prominent examples of colonial earthen architecture include the churches built throughout the Andean Cordillera, such as the Church of Kuño Tambo shown in Fig.1. Peru has a long reported history of strong and deadly earthquakes, which caused severe damage to heritage constructions. The damage caused by the 2001 Arequipa Earthquake (June 23), for instance, was evidence that historic earthen constructions are vulnerable to earthquakes, as it caused severe damage and even the collapse of several colonial churches and houses. Aguilar and Farfan (2002) report that 36 of the 246 historic houses of the city of Arequipa were severely affected by this earthquake. The recent 2007 Pisco earthquake (August 15) also caused the partial and total collapse of many historic adobe and timber buildings, as reported by Cancino et al. (2009).

The assessment of the seismic response of existing constructions can be based on the verification of the damage state of the structures for a given seismic action. Several damage states and corresponding compliance criteria are proposed for this purpose in standards (e.g. DPCM 9/2, 2011; ASCE/SEI 41-06, 2007; EN 1998-3, 2005). Demand parameters, such as drift and shear capacity, are

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normally used to verify whether a structure exceeds a given damage state. However, damage thresholds described in terms of drift are difficult to define, since there is little data of the progression of damage and deformation for existing structures (D'Ayala, 2005), especially for historic adobe constructions such as the Peruvian colonial churches. Nevertheless, some reference values of in-plane and out-of-plane drift for masonry constructions can be taken from literature (e.g. Varum et al., 2014; D'Ayala, 2013; Costa et al., 2012; NZSEE, 2011; ASCE/SEI 41-06, 2007; EN 1998-3, 2005; Calvi, 1999). Tolles et al. (2002), for instance, associate the states of none, slight, moderate, extensive and complete damage to specific damage location and severity in adobe buildings. However, the damage state of the buildings is described in a qualitative way and therefore difficult to relate with quantitative structural performance indicators such as drift.

Figure 1. Church of Kuño Tambo built in the 17th century in the province of Acomayo, Cusco, is representative of high lands colonial churches: (Left) South façade; (Right) interior of the nave.

The present research is part of the 'Earthen Architecture Initiative – Seismic Retrofitting Project in Peru' (SRP), a collaboration project of The Getty Conservation Institute (GCI), University College London (UCL), the Pontificia Universidad Católica del Perú (PUCP), and the Ministerio de Cultura del Perú (MCP). The project aims to research, design and validate seismic retrofitting techniques for historic colonial buildings in Peru. The project was promoted after the 2007 Pisco earthquake and case studies were selected based on their representativeness within Latin-American countries and seismic vulnerability as observed after past earthquakes. One of the SRP case studies is shown in Fig.1 – the Church of Kuño Tambo (KT). For further details on the construction system of this building see Cancino et al. (2012).

A logic-tree approach for the seismic diagnosis of historic buildings is proposed in the present paper. It has been applied to the diagnosis of the SRP case studies, both in the case where timber frames and vaults are the most critical components and when thick masonry walls govern the lateral response. The application of this methodology to the seismic diagnosis of KT is then presented.

METHODOLOGY

The approach for seismic diagnosis of historic constructions proposed here is based on the decomposition of the structures into several structural components or sub-structures (macroelements). A macroelement is a portion of the structure that can be considered independent from the rest, since in the event of an earthquake it will show a distinct seismic response, while interacting with other macroelements (Novelli and D'Ayala, 2011). These macroelements are then assessed according to a number of variables governing the seismic response of the structures. These variables are: i) the resilience of the structure in its undamaged and undeteriorated state, which provides a measure of the quality of the original structural concept; ii) the interaction between macroelements; iii) the type of connections within each macroelement; iv) the quality of the masonry fabric; and v) deterioration, which influences all the other variables. The approach is summarized in the tree diagram of Fig.2, where the diamond-shaped text boxes contain the variables and the rectangular text boxes contain the parameters evaluated for each variable.

The relevance of each variable is described by means of a *R* factor, which varies from 1 to the total number of variables. In the case of the diagram of Fig.2, 'Resilience' has relevance equal to 1,

since it is on the top of the assessment, while the 'Fabric' variable has relevance equal to 5. Neverthless, there is an important component of looping, since the diagnosis of less relevant variables normally influences the diagnosis of previously assessed variables.

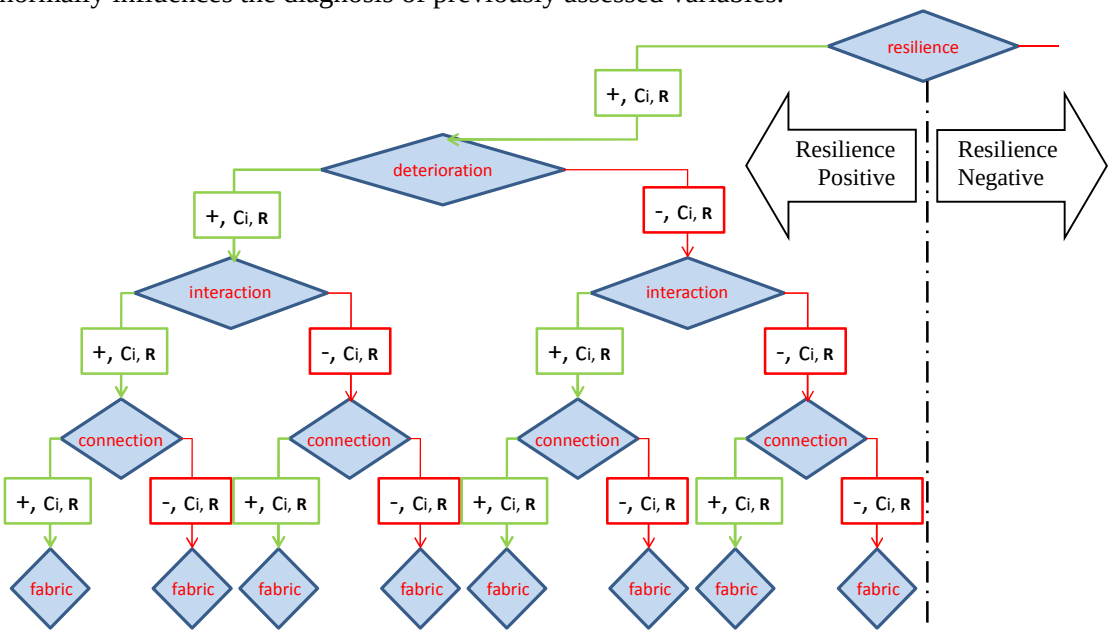

Figure 2. Tree diagram for the diagnosis of historic constructions

The assessment of the constructions or macroelements regarding each variable requires the evaluation of the state in terms of positive (marked as '+' and in green in Fig.2) or negative (marked as '–' and in red in Fig.2) and the attribution of a confidence factor to the judgment made. The confidence factor, *C*, associated to each possible path of Fig.2 is assumed as an average value of the confidence associated to the diagnosis of all variables, C_i. Hence, *C* dictates what the relevant paths to follow are. For instance, if two alternative paths have 50% of confidence, i.e. both scenarios are equally likely, then it is necessary to follow both paths during the planning of future interventions. On the other hand, if the confidence factor is higher than 50% for one path, it might be sufficient to only follow this path. Furthermore, alternative paths can be defined for different macroelements belonging to the same case study.

The variables are evaluated by means of local and global criteria. Local criteria take into account the existence of concentration of stresses and local fragilities, while global criteria evaluate the global response of the structure or macroelements in terms, for example, of drift and capacity. The 'resilience' variable is evaluated by means of global criteria. Variables such as interaction, connections and fabric are evaluated by means of local criteria. Different criteria may need to be formulated for different case studies.

GLOBAL CRITERIA

Global criteria are used to assess the resilience of the constructions to earthquakes. The following 3 criteria are applied:

G1. Regularity in elevation;
G2. Regularity in plan;
G3. In-plane and out-of-plane drift.

Criteria G1 and G2 are based upon the principle that a regular structure in elevation and plan has less propensity for local vulnerabilities. These are common guiding principles of national codes, such as the Peruvian (E.030, 2003) and European codes (EN 1998-1, 2004). However, the criteria for verification of regularity of these codes are not directly applicable to all building types, especially to historic constructions such as churches. Criterion G3 is a measure of the deformation capacity of the

structure and it evaluates the performance of macroelements regarding failure mechanisms associated to large displacements rather than degradation of strength.

LOCAL CRITERIA

Local criteria are used to assess the interaction conditions of the various macroelements, the performance of the connections and the quality of the masonry fabric. The following six criteria are applied:

L1. Maximum stresses and strains at interfaces;
L2. Occurrence of cracking;
L3. Homogeneity of the fabric;
L4. Shape ratios of units;
L5. Overlapping of units;
L6. Thickness and filling of the joints and quality of the mortar.

Criteria L1 and L2 are used to check the interaction conditions of the macroelements and the quality of the connections within the macroelements. The response of the macroelements in terms of stress distribution is directly taken from the results of pushover analysis performed with the finite element models of the case studies. Occurrence of cracking is investigated by checking whether the level of stress within the elements exceeds the assumed strength of the constituent materials. The other criteria are used to verify the quality of the fabric, which is investigated on the basis of geometric characteristics and observation, without considering the strength of the material, which is already taken into account by other criteria.

THE CHURCH OF KUÑO TAMBO (KT)

KT was built in the 17th century in the province of Acomayo, Cusco (see Fig.1 and Fig.3). The walls and buttresses are made of adobe with a rubble stone masonry base course. The thickness of the masonry walls range from 0.65m (WIII.3 of Fig.3) to 1.90m (WI.1 of Fig.3). The height of the base course ranges from 1.2m to 1.5m and the height of the adobe walls from 4m to 8.5m, as measured from the top of the base course up to the top of the gables. The height of the walls is not constant due to the fact that they follow the topography of the place. The adobe units are set in mud mortar. The base course, with thickness similar to the thickness of the walls, is made of stones of variable dimensions also set in mud mortar. The roof's structure of the church is a traditional system of roof-trusses known as 'par y nudillo' (rafter and collar tie). The longitudinal walls of the nave and the lateral walls of the baptistery are restrained by means of wooden tie-beams of approximately 0.20m diameter. Some of these tie-beams are fixed to the walls by means of timber anchors.

The diagnosis is conducted following the approach of Fig.2 and bringing together information from various sources, namely: i) results of the global numerical model of the church; ii) outputs of the testing campaign performed by PUCP on materials and structural components relevant to KT (Torrealva and Vicente, 2013); iii) authors' interpretation of results of tests from other sources; iv) on-site observation and survey, namely the SRP field campaigns that took place in the period 2010-2012; and v) relevant literature.

Pushover analyses are performed with the global numerical model of the church shown in Fig.3 (left). This model corresponds to an interpretation of the present state of the building. Two additional buttresses still standing on the East side of the church, next to buttresses B12 and B10. However, these buttresses are completely detached from the nave and therefore they are not included in the model. Similarly, KT had 4 buttresses on the West side, from which two buttresses are known to have been connected to the nave by interlocking of adobe units. Fig. 3 (right) shows the state of KT if these two collapsed buttresses of the West side are reinstated (buttresses B4 and B6). The roof structure is not modelled since it is difficult to simulate the complex relative rotation and sliding that the typical connections of the roof undergo. Furthermore, the failure of those connections relates to localized damage of the roof rather than failure of the walls, since it is assumed that the tie-beams are not

connected to the roof on the basis of on-site observation. A maximum of 0.3g acceleration was considered in the pushover analyses performed both in the X and Y positive and negative directions, which is the maximum ground acceleration with a probability of 10% to be exceeded in 50 years (return period of 475 years) for the zone where KT is located, according to the Peruvian seismic code (E.030, 2003). More details of the numerical models can be found in Fonseca Ferreira and D'Ayala (2012).

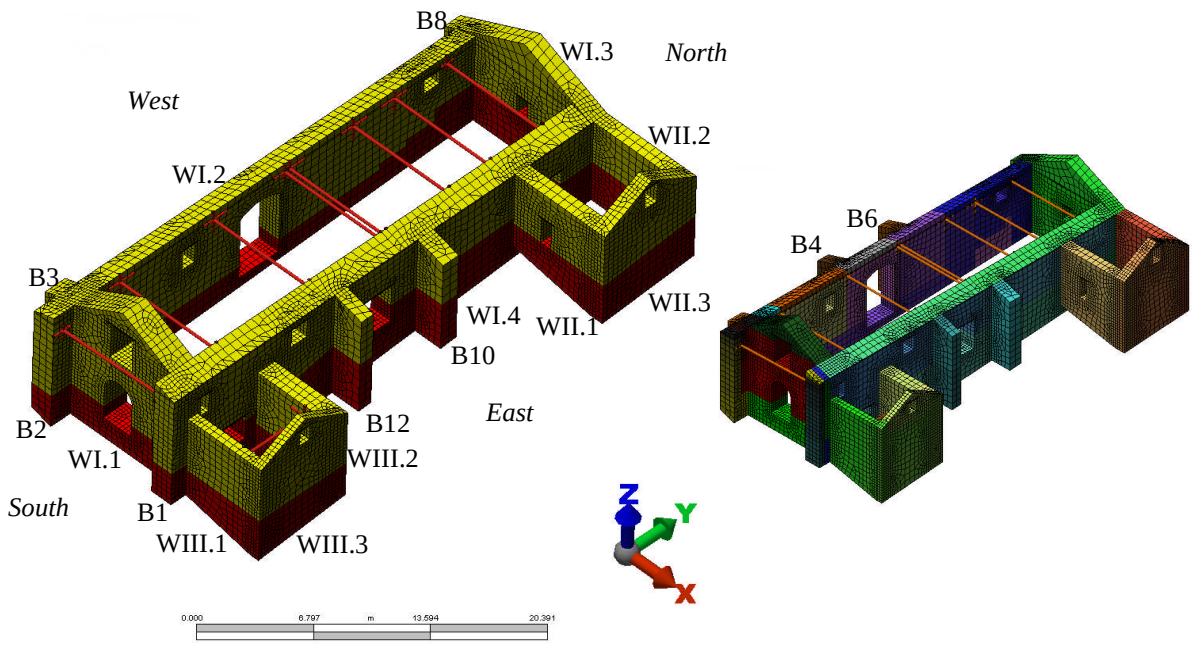

Figure 3. Global model of KT: (Left) present state; (Right) state after buttresses B4 and B6 are reinstated

KT is assumed as a system composed of the following macroelements for the purpose of the final diagnosis: 1) adobe façade of the nave (WI.1); 2) adobe back wall of the nave (WI.3); 3) lateral adobe walls of the nave (WI.2 and WI.4); 4) Rubble stone base course; 5) adobe buttresses (B1, B2, B3, B8, B10, B12); 6) adobe walls of the sacristy (WII.1 to WII.3) / baptistery (WIII.1 to WIII.3); 7) wooden tie-beams and anchors; 8) roof wooden structure (Fig.1 Right).

RESILIENCE OF KT

The vertical resisting system, which is composed of walls and buttresses, does not have significant misalignments, except in the case of WI.2 and WI.4. WI.2 is the most critical case due to the absence of the collapsed buttresses According to measurements made on-site, the permanent lateral deformation of these walls is 0.15m approximately, which corresponds to around 10% of the thickness of the wall.

The existence of a high base course and of an adobe wall on the top of it causes a variation of stiffness in elevation within the resisting system, which can be seen as an irregularity in elevation, even though the thickness of the base course is equal to the thickness of the adobe wall. Furthermore, the height of the walls change, since the building follows the natural topography of the place. The 'height of the base course/total height of the wall' ratio also varies among walls and even within a given wall. The base course has a minimum height of 2m, in order to protect the adobe walls from raising water. The existence of the base course also creates a plane of discontinuity at the interface adobe/rubble stone, which can lead to the sliding of the adobe wall on the top of the base course. This interaction is further evaluated within the 'Interaction' variable.

As far as regularity in plan is concerned, the presence of the sacristy and baptistery creates asymmetry with respect to the longitudinal direction of the church (Y-direction of Fig.3), even if the collapsed buttresses B4 and B6 are reinstated. As far as the transversal direction is concerned (X-direction of Fig.3), the church is also unsymmetric due to the progressive decrease of the height of

the walls from South to North, as the church is located on a hill. This makes the building asymmetric in terms of both mass and stiffness with respect to two orthogonal directions.

The in-plane and out-of-plane drift of the walls of KT are compared with reference values for unreinforced masonry walls available in literature. Table 1 shows reference drift ranges for in-plane and out-of-plane behaviour for several damage states (DS) obtained by experimental campaigns and by the FaMIVE procedure for masonry buildings in Nepal, Italy and Turkey, according to D'Ayala (2013). In the case of KT, the hypothesis of a good connection of the walls by interlocking of units at intersections and by the presence of tie-beams is investigated. Under this hypothesis, failure modes involving more than one wall are likely to develop (combined behaviour). This type of failure modes have greater stiffness than the in-plane and out-of-plane modes and hence it is characterised by lower values of drift (D'Ayala, 2013). Table 2 shows drift ranges for combined behaviour obtained from the FaMIVE procedure for buildings in Nepal, Italy and Turkey (D'Ayala, 2013). Finally, the limits of in-plane and out-of-plane drift of the walls of KT according to the provisions of Eurocode 8 (EN 1998-3, 2005) are shown in Table 3.

Table 1. Drift ranges for in-plane and out-of-plane response of masonry walls (D'Ayala, 2013)

Source	In-plane drift (%)			Out-of-plane drift (%)		
	DS of Damage Limitation	DS of Significant Damage	DS of Near Collapse	DS of Damage Limitation	DS of Significant Damage	DS of Near Collapse
Experimental	0.18-0.23	0.65-0.90	1.23-1.92	0.33	0.88	2.3
FaMIVE procedure	0.023-0.226	0.069-0.679	0.790-1.579	0.263-0.691	0.84-1.580	1.266-1.969

Table 2. Drift ranges for combined response of masonry walls (D'Ayala, 2013)

Source	Drift for combined behaviour (%)		
	DS of Damage Limitation	DS of Significant Damage	DS of Near Collapse
FaMIVE procedure	0.030-0.168	0.099-0.582	0.198-1.401

Table 1. Drift limits for KT according to the provisions of Eurocode 8 (EN 1998-3, 2005)

Walls of KT	In-plane drift (%)			Out-of-plane drift (%)		
	DS of Damage Limitation	DS of Significant Damage	DS of Near Collapse	DS of Damage Limitation	DS of Significant Damage	DS of Near Collapse
WI.1		0.4	0.53		0.36	0.48
WI.2		0.4	0.53		0.25	0.33
WI.3		0.4	0.53		0.37	0.49
WI.4		0.4	0.53		0.25	0.34
WII.1		0.4	0.53		0.28	0.37
WII.2		0.4	0.53		0.24	0.33
WII.3		0.4	0.53		0.45	0.60
WIII.1		0.4	0.53		0.44	0.58
WIII.2		0.4	0.53		0.23	0.31
WIII.3		0.4	0.53		0.74	0.98

Considering the current state of KT, as shown in Fig.3 (Left) and assuming a good connection among walls, the capacity curves of the walls of KT for in-plane response are shown in Fig.4. The most vulnerable walls are WIII.2 and WII.1 with values of drift that can exceed the DS of significant damage if the reference drift ranges from FaMIVE are considered. These walls are subjected to in-plane inertial forces transmitted by the out-of-plane movement of the longitudinal walls of the nave. Walls I.1, I.2, I.4, II.2 and III.1 can also exceed the DS of significant damage if the lowest thresholds from FaMIVE are considered. A similar situation occurs for walls I.3, II.3 and III.3 as far as the DS of

damage limitation is concerned. The most vulnerable walls have a nonlinear response from approximately 0.055% drift onwards, which indicates that an important reduction in stiffness initiates for an acceleration between 0.2g and 0.25g. A limit state of damage limitation is characterised by negligible permanent drifts, where the structural elements retain their strength and stiffness properties (see e.g. EN 1998-3, 2005). Hence, 0.055% drift might be a lower bound for the DS of damage limitation as far as the wals of KT are concerned. The excedance of this limit state is especially important in the case of KT, since the church has valuable interior wall paintings.

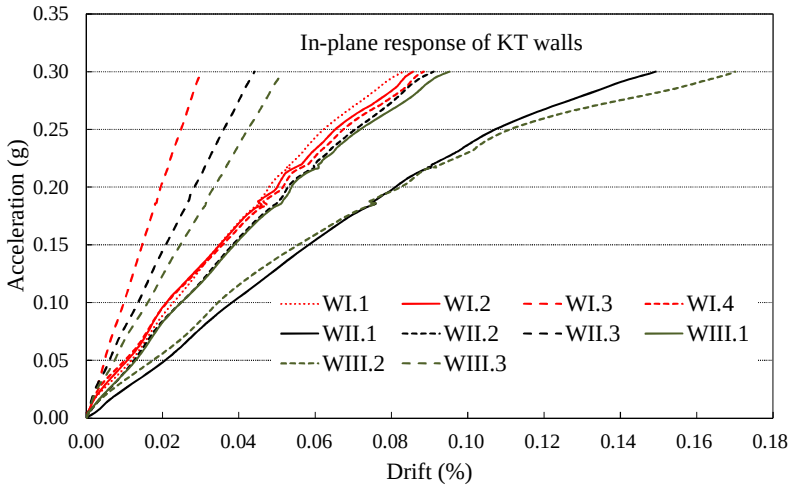

Figure 4. Capacity curves of KT for in-plane response described in terms of acceleration and drift

Fig.5 shows the capacity curves for out-of-plane response of the walls of KT. It can be seen that walls I.2 and I.4 exceed the DS of near collapse if the provisions of Eurocode 8 are considered. If the drift range from FaMIVE for combined response is taken into account, all walls of KT can exceed the DS of significant damage. Walls I.2 and I.4 present an important nonlinear plastic response from 0.24% and 0.20% drift onwards, respectively, which corresponds to an acceleration of 0.2g in both cases.

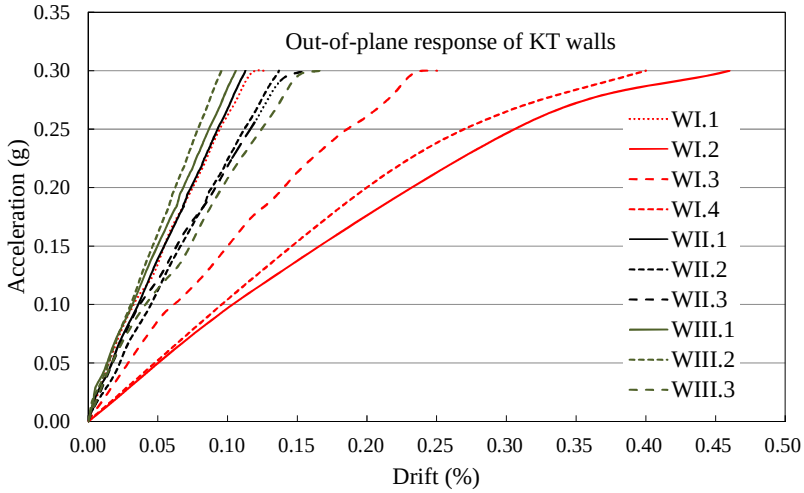

Figure 5. Capacity curves of KT for out-of-plane response described in terms of acceleration and drift

The existing lateral restraints of walls I.2 and I.4 appear to be insufficient to avoid severe damage and even the collapse of these walls. Hence, it is important to check whether the lateral response of walls I.2 and I.4 is considerable different when buttresses B4 and B6 (see Fig.3 Right) are reinstated. The out-of-plane drift of these walls is therefore shown in Fig.6 for the following hypotheses: i) tie-beams modelled and not modelled; and ii) buttresses B4 and B6 modelled and not modelled. Hypothesis i) informs on the structural relevance of the tie-beams. It can be seen that if buttresses B4 and B6 are reinstated and the tie-beams are effective, wall I.2 might present significant

damage; however it will not collapse, as per the provisions of Eurocode 8. Similarly, wall I.4 might not present significant damage. It is interesting to notice that the response of wall I.4 when the tie-beams are effective and buttresses B4 and B6 are reinstated is similar to its response when the tie-beams are removed.

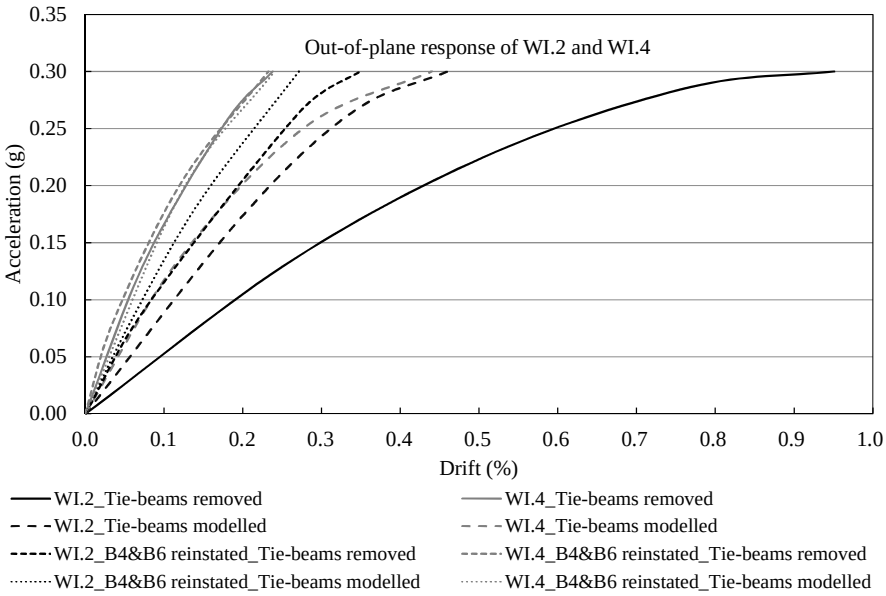

Figure 6. Out-of-plane capacity curves of WI.2 and WI.4 for various hypotheses

The overall resilience of KT is considered good with 85% of confidence, since the original concept leads to a structural performance that complies with the requirements in terms of safety, provided that the collapsed structural elements are reinstated and the macroelements interact well with each other. However, the original concept might not prevent the existence of significant damage in wall I.2. The confidence associated to this diagnosis is modest, since the reference drift ranges used to assess the damage state of the walls were not formulated, calculated or measured to the specific case of adobe walls.

INTERACTION OF MACROELEMENTS

The macroelements have structural links that govern the phenomena of interaction between them. This interaction usually occurs at an interface, which can be considered poor or good depending on local conditions. The interactions analysed for KT are the following:

1. Interaction between the walls of the nave;
2. Interaction between the walls of the baptistery/sacristy;
3. Interaction between the walls of the nave and the walls of the baptistery/sacristy;
4. Interaction between the walls and the buttresses;
5. Interaction of the base course with the adobe walls;
6. Interaction of the wooden tie-beams with the walls;
7. Interaction of the roof's structure with the walls.

These interactions are evaluated by means of on-site observation of the interfaces between macroelements and by evaluation of the stress state at those interfaces by means of the numerical results obtained with the model of the present state of KT (Fig.3 Left). The latter criterion is an indicator of the existence of excessive concentration of stresses at the interfaces. This is done by computing the percentage of finite elements that fail at the interfaces using the Drucker-Prager failure criterion (Drucker-Prager, 1952). This criterion is also the material model used to simulate the response of the masonry walls and buttresses of KT. Furthermore, the maximum level of tension stress is compared with the tension strength of the material in order to evaluate the percentage of finite elements that fail in tension rather than in shear. The assumed tension strength of adobe and rubble

stone masonry are 30.75kPa and 48.75kPa, respectively. These values relate to 75% of the corresponding cohesion, which is generally accepted as a good representation of the masonry tension strength. The values of cohesion were experimentally obtained by PUCP by means of shear-compression tests performed with triplets (see Torrealva and Vicente, 2013). The greatest concentration of stresses of KT model occurs between walls I.4 and II.1 and between walls I.4 and III.2, where around 70% of the interface fails in tension. In summary, severe cracking occurs at:

- The interaction of the nave and baptistery with the East wall of the nave (WI.4);
- The interaction of the North wall (WI.3) with the walls of the nave (WI.2 and WI.4);
- The interaction of the nave (WI.4) with buttresses B10 and B12. Even if the percentage of failed elements at these interfaces is not as great as in the case of the above interfaces; the buttresses are unable to restrain the out-of-plane movement of the walls if cracks develop between them and the walls.

As far as the interaction of the wooden tie-beams with the walls is concerned, verification is done under the hypothesis of absence of wooden anchors, as on-site observation provided evidence of existence of only three tie-beams anchored to the walls at present. No anchors were observed at the connection of the other six tie-beams with the walls. On-site measurements show that the tie-beams not anchored to the walls are simply embedded 0.6m into the walls. The capacity of this type of connection is therefore governed by a Coulomb-type law; i.e. the capacity relies on the friction between timber and adobe and the normal force provided by the weight of the adobe wall and tie-beam plus the vertical overload provided by the roof. However, the tie-beams are located almost at the top of the adobe walls, so a relatively modest vertical load is present. Considering a coefficient of friction of 0.3 (NZSEE, 2011), the capacity of the connection is equal to 4.8kN. For an acceleration of 0.3g, all the tie-beams would slide, as the ties are subjected to a pull-out load that ranges from 8 to 50kN.

The wall-plates of the nave are not continuous and therefore they do not work as an element that distributes the load evenly among the adobe walls. This configuration of the wall-plates enables the load provided from the roof weight to be distributed along the top of the walls; however, it does not promote a box-like response of the nave. As far as the roof's structure of the sacristy and baptistery are concerned, there are no wall-plates and therefore the rafters are sitting directly on the walls. This interaction creates excessive concentration of stresses around the rafters.

Interlocking of adobe units between the sacristy and the nave and between the walls of the nave and buttresses B10 and B12 was observed on-site, as shown in Fig.7. This might indicate a good level of connection between these macroelements. On the other hand, on-site observation also allows concluding that the baptistery does not have the same level of interlocking, which might suggest a poor interaction with the nave.

Interlocking between WI.4 and WII.1 Interlocking between WI.4 and B12

Figure 7. Interaction between KT macroelements

It is therefore considered that all these interactions are poor with 80% of confidence and therefore require further improvements, except the interaction of the base course with the adobe walls. The latter interaction constitutes a discontinuity in the vertical resisting system and therefore cracking of the walls might preferably occur at this discontinuity. However, it might also have a damping effect

on the dynamic response of the church, provided that the walls are kept stable by orthogonal walls, buttresses and tie-beams.

CONNECTIONS WITHIN MACROELEMENTS

The following connections are evaluated:

1. Connections within the timber roof;
2. Connection of the wooden tie-beams with the timber anchors.

The rafters of the nave are simply sitting on wall-plates with a modest scarf. Each pair of rafters are connected together by means of leather straps and, in some cases, wrought iron nails. The same type of connection is used to connect the collar beam with the rafters. These joints are very flexible and the elements slender, so the roof structure has a modest in-plane and out-of-plane stiffness. As far as connection 2 is concerned, there are at least 3 different types of timber anchors in KT, as shown in Fig.8. However, the connection of the key with the tie is rather poor and the key is slender and unable to avoid excessive stress concentrations, especially when the anchor is done with ony 1 vertical key. This variable is therefore considered negative with 90% of confidence.

WI.2 WI.2 WIII.2

Figure 8. Layout of KT wooden anchors

QUALITY OF THE MASONRY FABRIC

A typical adobe brick measures 0.70m long x 0.35m wide x 0.20m high (Cancino *et al.*, 2012). The shape ratios of the units are 1:2 width/length and 1:3.5 height/length. The adobe units are set in an English bond pattern; even though the pattern is not clearly identifiable in some areas of the walls due to the deterioration of the adobe wall. The masonry is characterised by an overlapping length of 0.35m, which is ½ of an adobe unit, on face and through thickness of the wall. Head and bed joints are completely filled with fairly homogeneous mud mortar mixted with straw. The thickness of both head and bed joints varies within a range of 0.015m-0.03m. These are characteristics of a good masonry quality. However, the mud plaster detached almost completely from most of the adobe walls and buttresses, except from WI.1 which seems to have been done with a better plaster. The adobe fabric is therefore considered positive with 60% of confidence. The rubble stone masonry of the base course is made of heterogeneous stones with joints of various thicknesses – the range of variability goes from 0.02m to 0.06m. The base course does not have plaster. The quality of the stone masonry fabric is therefore considered poor with 90% of confidence.

LOGIC-TREE DIAGNOSIS OF KT

The diagnosis of KT is illustrated in Fig.9. ‘Resilience’ is the most relevant variable, since the structure still sound; even though interaction between macroelements and connections are poor and

maintenance is neglected. 'Deterioration' is the second most relevant variable due to the collapse of buttresses B4 and B6 and generalised material decay due to inappropriate water drainage and use of poor masonry plasters. The collapse of buttresses B4 and B6, which are believed to have been well connected with WI.2 by interlocking of units, substantially increased the seismic vulnerability of WI.2. Material decay also contributes to a poor interaction of macroelements by decreasing the strength of the material at the interfaces. Furthermore, the capacity of the connection of the tie-beams with the adobe walls is further compromised by the presence of moisture at the top of the walls.

The most important structural vulnerabilities of KT are related to poor interaction conditions between the macroelements, if the collapsed elements are reinstated. The connections within the roof structure and between the tie-beams and the anchors are also very poor. However, the quality of the connections might not be as critical as the interactions between macroelements, provided that these interactions promote a box-like response. This explains the fact that a higher relevance is attributed to the 'Interactions' variable than to the 'Connections' variable. Two alternative paths are obtained. Path 1 regards to the base course and path 2 to the other macroelements.

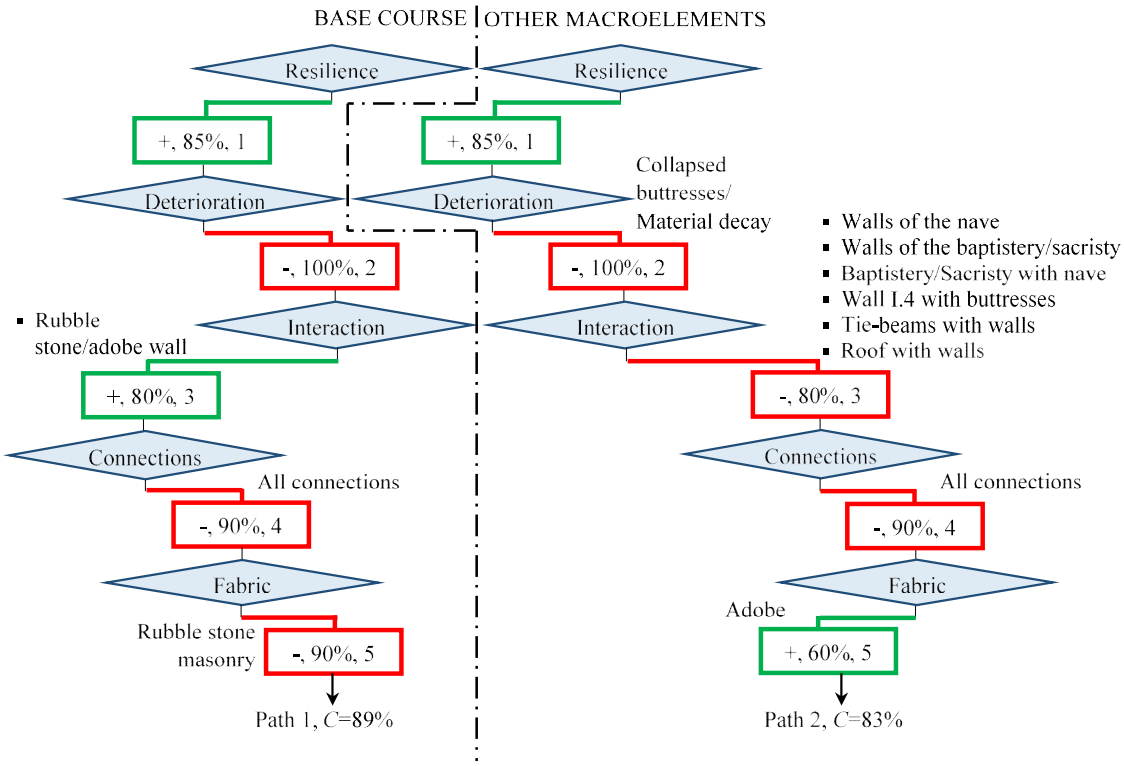

Figure 9. Logic-tree diagnosis of KT

CONCLUSIONS

An approach for the seismic diagnosis of historic buildings is proposed. It is based on a logic sequence of analyses of variables that influence the seismic response of the constructions. The diagnosis is summarised in a logic-tree format where the vulnerabilities of the structures are clearly and quickly identified. Alternative paths characterised by a qualitative confidence level, which is attributed by the analyst as a percentage of his/her confidence on the judgment made, are defined in the diagnosis tree. The most plausible paths, i.e. the paths with highest levels of confidence, can therefore be taken into account in the planning and designing of future interventions if necessary. The general framework of the approach proposed here can be applied to all building types. The variables and the global and local criteria can then be easily adapted to specific case studies.

This approach was then applied to the seismic diagnosis of a Peruvian adobe church (Kuño Tambo, KT). Although KT is rather simple from a structural viewpoint and composed of a few number of macroelements, variables such as 'Interaction between macroelements' and 'Connections'

require a considerable amount of information to be evaluated with a good level of confidence. Different sources of information, such as numerical modelling, experimental work and on-site survey, were therefore taken into account. However, on-site survey was limited to the extent of which it did not imply loss of historic materials. The diagnosis of KT shows that the building has a good resilience, i.e. the structural concept of thick adobe walls restained by buttresses, transversal walls and tie-beams is good in terms of seismic response. However, the interactions between macroelements are poor in most cases and damage/deterioration has further increased the vulnerability of KT. Hence, the macroelements do not cooperate with each other to promote the originally sought box-like response.

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