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STUDY ON THE SEISMIC BEHAVIOUR OF ARCHAEOLOGICAL HERITAGE BUILDINGS: A WALL IN CHOKEPUKIO

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Abstract. *Archaeological buildings are like a constructed calendar of the history of civilizations, which present thus a major importance for the preservation of cultural, ethnographic and artistic values of past folks. Here, investigations are developed on the seismic behaviour of archaeological building remains, with application to the Chokepukio site in Cuzco, Peru. The study included on-site inspection, experimental tests and numerical modelling and analysis, performed to evaluate the seismic vulnerability of the remaining traces of Chokepukio.*

The study started by identifying the historical background of the site, after which a characterization of the architecture and a survey of the remaining structures were made. In-situ experimental investigation was carried out on a particular remaining stone masonry wall, which dates back to the 12th Century. Experimental tests were mainly based on operational modal analysis, which was used as a non-destructive technique to obtain the dynamic modal parameters of the structure (frequencies and mode shapes).

Different finite element (FE) models were considered regarding material and structural variables, searching for the one that represents better the studied wall. Within this process, each FE model was calibrated through a sensitivity manual analysis and an automatic optimization routine. Computed indicators for the final selected-updated wall model show high correspondence between experimental and numerical frequencies and mode shapes. The updating of the FE model allowed the identification of two different qualities of masonry, which matches with the field observation.

Regarding the seismic assessment, kinematic limit analysis was performed based on simple equilibrium of macro-blocks. Global rocking motions and a mechanism with partial rocking of the wall were considered. The results of analysis indicate safety of the global rocking mechanisms, but the partial mechanism results unsafe. It can be concluded that safety factors are sensible to a little change in the wall condition, thus this kind of analysis is suitable to support decision of seismic retrofitting with minimal intrusion.

1 INTRODUCTION

The archaeological remains in the Andean region of Cuzco, Peru, are an ancient example of sustainability in construction, since they were built by exploiting the high availability of local stone material. In effect, stone was the material used between Wari and Inca cultures to erect citadels as support to their civilizations. These constructions were mostly built of andesite stone, which is an extrusive igneous rock named after the Andes. However, the masonry pattern suffered an evolution through civilizations, which was mainly associated to new skills developed by builders. Referring to pre-Inca settlements in Cuzco region, constructions present a masonry pattern of irregular rubble stone arranged between earth layers, e.g. Piquillacta (VI to IX centuries) in Fig. 1a. Later, stone masonry evolutes for a system with more geometrically regular rubble stones and earthen-based joints, which is the case of Chokepukio (X–XV). In the Inca culture (XV–XVI) the stone construction assumes a more monumental style, where the stonework is made of large stone blocks perfectly fitted together without mortar, e.g. Sacsayhuaman in Fig. 1b.


a

b

Figure 1: Stonework of constructions in (a) Piquillacta and (b) Sacsayhuaman.

Despite the large number of patrimonial stone constructions, Peru is a country mostly identified with earth construction. The structural seismic assessment of these constructions has been poorly studied and disseminated, e.g. by Zavala et al. [1] for Caral citadel. On the other hand, Cuzco presents a fault system, see Fig. 2a, that is active and causes a high seismic hazard. Effects of ground settlements, probably due to seismic activity associated to fault motion, are observed for example in Machu Picchu, Fig. 2b. Furthermore, the structural behaviour of this kind of constructions is particularly complex, once the difficulty for characterizing the geometry, materials and damage state, for identifying the structural system, as well as for creating reliable numerical models.

The International Council on Monuments and Sites (ICOMOS) has published different strategies for studying historical constructions. These strategies evidence the need for deep knowledge on the aspects referred before, which can only be assessed by extensive experimental and diagnosis campaigns. In this context, non-destructive testing is an important tool since it allows the evaluation of constructions with minimum intrusion. Vibration tests, such as operational modal analysis (OMA), are a powerful inspection technique that allows the estimation of the structural dynamic parameters. On the other hand, structural evaluation requires adequate methods and tools for modelling and analysis, particularly regarding the seismic assessment. For this purpose, several approaches can be used regarding the nature and complexity of the construction, such as continuum finite element models, structural component methods or rigid block analysis [2].

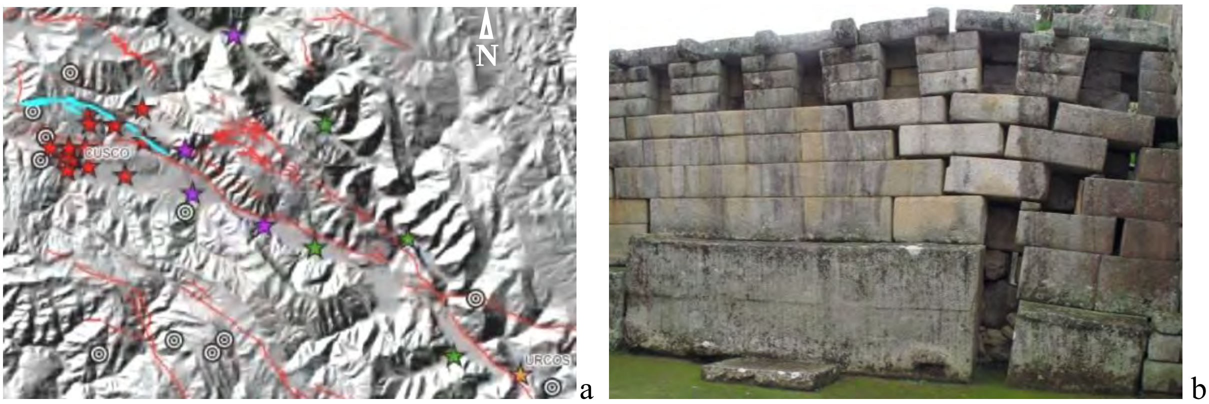


Figure 2: Seismicity of Cuzco: (a) neotectonic map and (b) effect of ground settlements to wall in Machu Picchu.

In complex structures, as is the case of archaeological building remains, the seismic assessment may require a multiple-view analysis approach, using different methods in mode to validate one against the others. The finite element (FE) method has been widely applied for seismic assessment of historical constructions, e.g. by Lourenço et al. [3] applied to masonry ruins. However, the development of a reliable FE model normally requires a calibration of the actual condition of the structure, namely regarding material elastic parameters, boundary conditions and existing damage. This calibration is usually made based on experimental in-situ vibration tests, trying to approximate the experimental mode shapes by the numerical simulation, through a progressive process of updating values for the model variables.

This paper presents investigations regarding the structural behaviour of archaeological heritage buildings, with application to the Chokepukio Archaeological Site in Cuzco, Peru. The study includes on-site inspection, experimental tests and numerical modelling and analysis, performed to evaluate the seismic vulnerability of the remaining traces of Chokepukio.

2 THE CHOKEPUKIO ARCHAEOLOGICAL SITE

The Chokepukio Archaeological Site is located 30 km from the city of Cuzco, in the Andes of Peru. A wide variety of remaining structures, built with stone masonry and mud mortar, were found in this archaeological site. Unfortunately, this complex is severely damaged, due to weathering, ground settlements and earthquakes, e.g. the 1950 Cusco earthquake. Chokepukio was occupied by Lucre and Killke cultures, which are considered ethnicities of transition between the Wari (650–1000 AD) and Inca (1425–1532 AD) [4].

Chokepukio presents a particular architecture of walls forming enclosures around open spaces, which are connected by streets and narrow passageways. In order to describe the local architecture, McEwan et al. [4] divided the site in three principal sectors, named from A to C, according to construction features and occupation period. Sector A presents the highest density of standing structures and its walls enclose substantial areas (2,600 m^2) with small rooms connected among them, see Fig. 3a. This sector genuinely belongs to the late Intermediate period of the Peruvian history (1000–1450 AD), previous to the Inca occupation, and is the zone studied in this work. Isometric schemes of typological structures are presented in Figure 3b.

Constructions found at Chokepukio were built using local andesite stone. The height of the constructions ranges from 8 to 10 m. The masonry is composed of irregular rubble stones with mud-mortar joints of thickness between 2.5 to 10 cm. Each wall of Chokepukio seems to be constructed in stages with growth in length and height, which aspect is evidenced by marked transitions among some of the stone courses. The mortar is a mixture of local soil, clay, straw and cactus resin. Walls were built with multi-leaf arrangement and are 2 m width in average.

In general, the walls present trapezoidal and rectangular niches at different heights. In some cases, the original earthen coatings are still visible on walls and niches. Nowadays, building remains, arranged in twelve groups of structures, have been found, which correspond to an urban settlement.

One particular feature at Chokepukio is that walls include transversal buttresses, to improve vertical stability, and probably also to provide earthquake resistance. However, most of buttresses are partially in ruins, making the walls more vulnerable to out-of-plane failure. In order to know the foundation conditions, local excavations were carried out in the east corner of Sector A. It was observed that the foundation is 3.0 m in depth (with respect to the lowest point of the door, see Fig. 4) and includes footings to increase the wall stability.

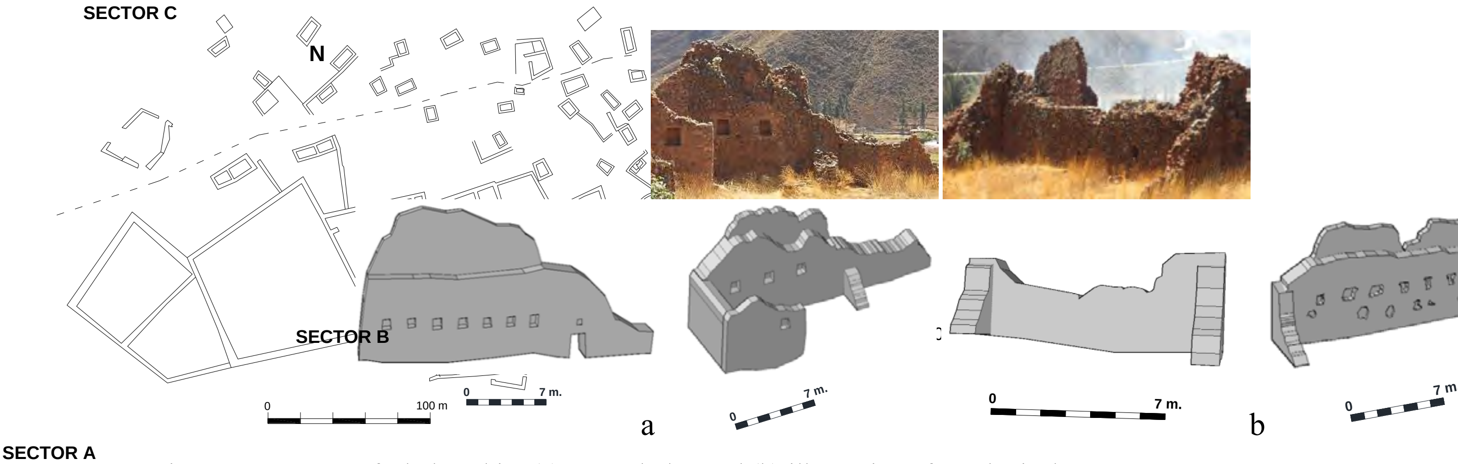


Figure 3: Sector A of Chokepukio: (a) general plan and (b) illustration of typological structures.

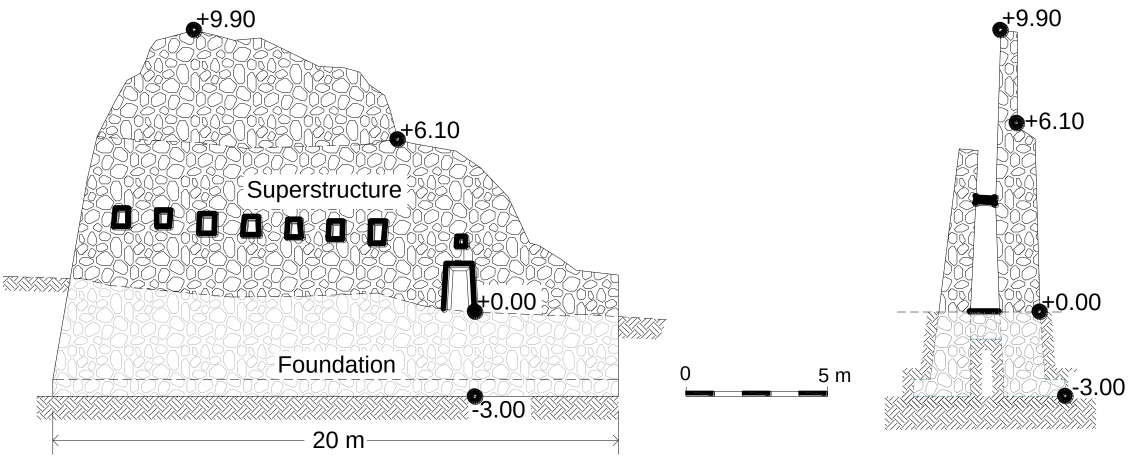


Figure 4: Front and transversal sections of the studied wall structure.

3 EXPERIMENTAL DIAGNOSIS TESTS

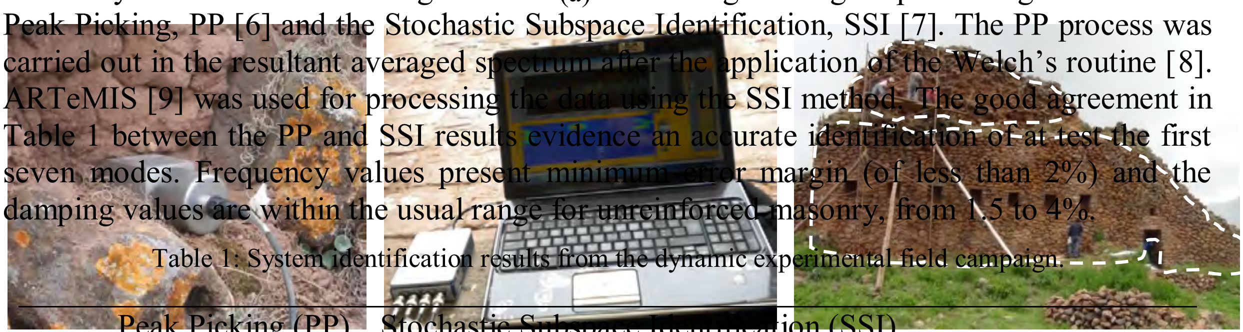
In-situ investigation was mainly based on operational modal analysis, OMA [5], which was used to obtain the dynamic modal parameters of the structure (frequencies and mode shapes). Ambient vibration tests were carried out on the structure presented in Figure 4, which is a couple of walls that are shored one against the other with timber struts. In this case, only the front wall was instrumented, which presents variable geometry (thickness varies from 1.2 to 1.8 m at the base and 0.4 to 0.6 m at the top), and average length and height of 20 m and 9 m, respectively. The wall presents two different stone patterns in height. The bottom part is built with large stones and thin mud-mortar joints, while the top part presents smaller stones and thick mud-mortar joints.

For the experimental tests, sixteen measurement points were set in the wall in order to acquire as much data as possible, see Fig. 5. For this purpose, seven setups were considered using two sensors at the wall top as reference nodes. In all setups, the sampling rate was set to 200 Hz and the acquisition time to 10 min. The transducers used were four piezoelectric accelerometers with a sensitivity of 10 *V/g* and a dynamic range of ± 0.5 *g* together with an USB-powered 24 bits resolution data acquisition system.

Figure 5: Front and transversal sections of the studied wall structure.

(a)

The system identification stage was carried out using two signal-processing methods: the Peak Picking, PP [6] and the Stochastic Subspace Identification, SSI [7]. The PP process was carried out in the resultant averaged spectrum after the application of the Welch's routine [8]. ARTeMIS [9] was used for processing the data using the SSI method. The good agreement in Table 1 between the PP and SSI results evidence an accurate identification of at test the first seven modes. Frequency values present minimum error margin (of less than 2%) and the damping values are within the usual range for unreinforced masonry, from 1.5 to 4%.



(b) (c) (d)

Table 1: System identification results from the dynamic experimental field campaign.

Mode	Peak Picking (PP)	Stochastic Subspace Identification (SSI)		Frequency error [%]
	Frequency [Hz]	Frequency [Hz]	Damping [ξ]	
1	1.99	2.00	2.9	0.22
2	3.16	3.16	4.0	0.23
3	4.30	4.37	3.1	1.60
4	5.12	5.10	2.2	0.41
5	6.33	6.33	2.6	0.31
6	6.72	6.81	2.4	1.03
7	9.18	9.23	3.4	0.11

4 NUMERICAL MODELLING AND UPDATING PROCESS

Numerical modelling is a compromise between allowed complexity and acceptable accuracy. In this study, three finite element (FE) models, which are presented in Figure 6, were developed using DIANA [10]. Initially, models were built considering the masonry as a homogeneous material with E-modulus of 800 MPa and specific weight of 27.5 kN/m^3, according to values in the literature, e.g. [11]. The models were assembled using eight-node isoparametric brick elements of type HX24L. The walls are considered as fully restrained at the base, as they have significantly deep foundations. In all models, the walls were considered with different thicknesses in height corresponding to section changes. In the same way, the thickness in length of the instrumented wall was considered variable from 1.20 m on the left side, to 1.50 m from the door to the right side.

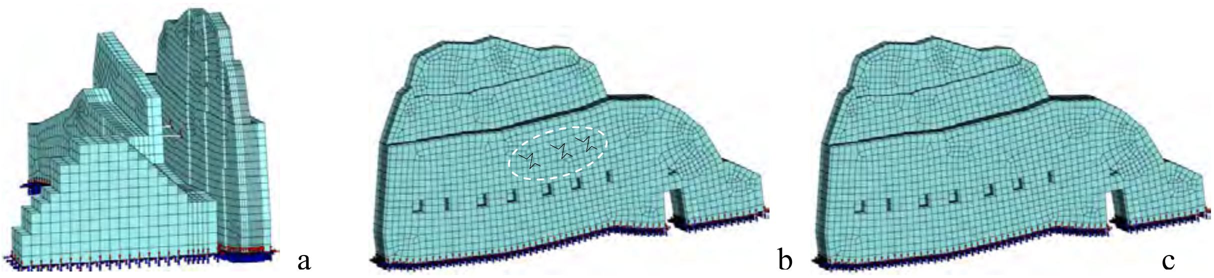


Figure 6: Developed FE models: (a) Model 1 considering interaction between walls, (b) Model 2 assuming the instrumented wall as decoupled and (c) Model 3 considering only the instrumented wall.

A first model was built taking into account the interaction between the instrumented wall and the other one located behind it, through the existing timber truss, as presented in Fig. 6a. The timber struts were considered as eucalyptus pieces with an E-modulus of 15,000 MPa, which were modelled as beam elements hinged at the connection with the walls. A second model was created assuming the instrumented wall as decoupled, and simulating the interaction from the timber struts as no-tension springs with an equivalent axial stiffness, see Fig. 6b. Finally, a third model was built assuming the two walls as completely disengaged (only the instrumented wall is considered, see Fig. 6c).

The numerical modal analysis and the subsequent model updating process were carried out considering only the first four natural frequencies and corresponding mode shapes of the instrumented wall, mainly due to the complexity of the higher modes. In order to compare experimental and numerical mode shapes and frequencies, the Modal Assurance Criterion, MAC [12] and the Frequency scales with MAC combination, FMAC [13] were used.

Figure 7 shows the FMAC graphs obtained for each model before the optimization process, which denote that neither the frequencies nor the mode shapes are accurately estimated by any numerical model. Moreover, frequency errors are high, evidencing poor accuracy. However, Model 3 presents less error in frequencies (max. frequency difference of 45%) while MAC values are better in Model 1 (lower MAC value of 0.57). Between these two models, Model 3 was preferred to apply a calibration by an optimization process, due to its simplicity and lower computational demand.

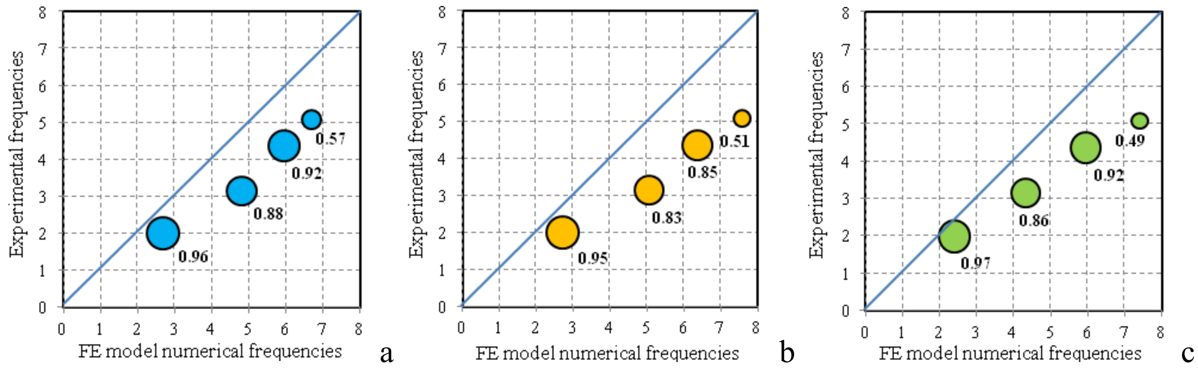


Figure 7: FMAC graphics for: (a) Model 1, (b) Model 2 and (c) Model 3 (frequency values in Hz).

The purpose of the optimization is to find the most appropriate values for unknown variables (which are set at the beginning of the process) in order to approximate the numerical frequencies and mode shapes to those experimental. The process is monitored through computation of the error or objective function defined by Ramos [14], which is minimized using a least square method. The optimization process was implemented using the 'lsqnonlin' algorithm available in MatLab [15].

The unknown variables were selected through sensitivity analysis, from an initial set that included geometrical aspects, boundary conditions and material properties. Moreover, the possibility of different materials for the bottom and top parts of the wall was considered. The conclusion was that the variables with more influence on changing frequencies were the E-modulus and the specific weight, while the mode shapes were mostly influenced by the geometry. The final values for the varying parameters evidence a clear difference of the E-modulus for the bottom and top parts of the wall, respectively of 580 MPa and 210 MPa. The E-modulus of the top part is lower because the masonry is most an earth-mortar/rubble stone mix. For the specific weight, a value close to 25 kN/m^3 was obtained for both parts of the wall.

The FE model was then updated by considering the optimised values for variables. The final FMAC relationship, which is presented in Figure 8a, denotes high correspondence between numerical and experimental frequencies and mode shapes: the maximum difference in frequencies is less than 8%, while the lowest MAC value is 0.86. For a visual comparison, Figure 8b-c presents the first four mode shapes obtained from the experimental and numerical approaches. These results evidence the utility of the optimization process and the reliability of the updated numerical model.

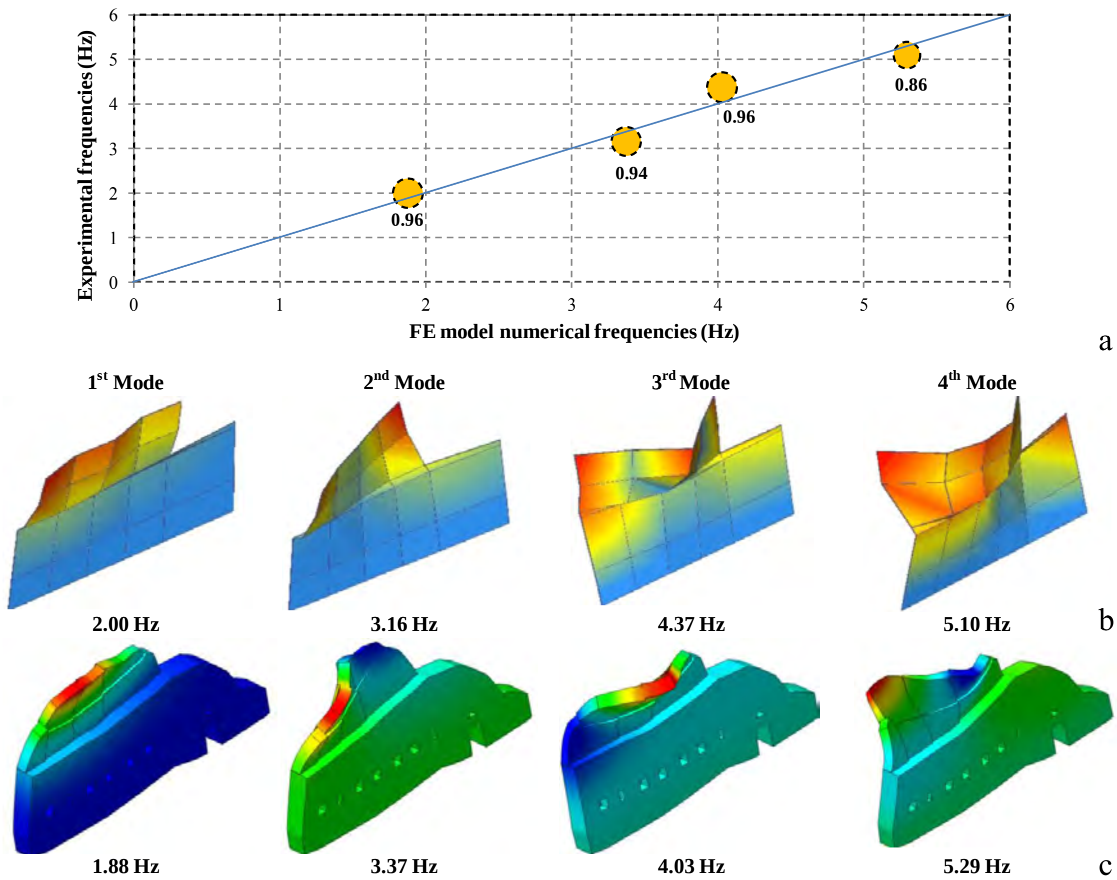


Figure 8: Experimental versus FEM mode shapes: (a) FMAC after optimization process, (b) experimental mode shapes and (c) FEM numerical mode shapes.

5 SEISMIC SAFETY VERIFICATION THROUGH LIMIT ANALYSIS

The plastic or limit analysis has been historically developed as an approach for evaluation of structures that present a behaviour determined by the formation of plastic hinges. For these structures, as soon as a sufficient number of hinges is formed, a collapse mechanism is activated with free-body movement that induces mostly inertial forces to the released parts. Then, a simplification is made by considering a static equilibrium approach [16]. For the case of masonry buildings, the common approach for limit analysis is based on macro-block discretization, by assuming collapse mechanisms for large structural assemblages.

Limit analysis has been included in seismic codes as a possible method for local failure assessment, e.g. in the Italian building code [17]. This procedure has been applied for seismic assessment of masonry structures complementarily to FEM-based approaches, e.g. [3]. Limit analysis is in general not sufficient for a full structural analysis under seismic loads, but it can be used to obtain a simple and quick estimation of collapse loads and failure mechanisms.

The key of limit analysis is the definition of potential collapse mechanisms, which can be a relatively difficult task depending of the structural typology. While for a masonry arch a mechanism requires the formation of at least four hinges, for a cantilever wall only one hinge is required. The hinge is expected to be formed, under the lower bound theorem and corresponding to the inferior limit condition [16], with the thrust line touching the section edge at the base of the wall. Hinges are also to be expected in sections with marked geometry change.

After the definition of the collapse mechanism, a kinematic approach is used to evaluate the load multiplier that activates the mechanism, α_0, which is the relationship between the horizontal load and self-weight applied to each body involved in the mechanism. Then, the solution for the equilibrium can be obtained through application of the principle of virtual work (PVW), which can be formulated (for a mechanism involving n bodies, m weight loads from dead bodies, and o external forces) according to:

$$\alpha_0 \left[\sum_{i=1}^{n} W_i \delta_{x,i} + \sum_{j=n+1}^{n+m} W_j \delta_{x,j} \right] - \sum_{i=1}^{n} W_i \delta_{y,i} - \sum_{h=1}^{o} F_h \delta_h = Int.\ Work \tag{1}$$

where W_k is the weight of the body k; $\delta_{x,k}$ and $\delta_{y,k}$ are virtual displacements of the body k relatively to its mass centroid, in x (horizontal) and y (vertical) directions respectively; F_h is an external force applied to a body and δ_h is its corresponding virtual displacement on the body.

The load multiplier α_0 activates the mechanism, after that a progressive motion occurs until reaching a maximum displacement state, corresponding to a zero value for the load multiplier (weight load vector is aligned with the hinge point), see Fig. 9a. The kinematic response considers the horizontal action that the structure is progressively able to stand with the evolution of the mechanism, until the annulment of the horizontal force itself, i.e. as long as the structure is not able anymore of stand horizontal actions. The relationship between the load multiplier and the displacement d_k of a control point k can be assumed linear according to:

$$\alpha = \alpha_0 \left[1 - {d_k}/{d_{k,0}} \right] \tag{2}$$

where $d_{k,0}$ is the displacement of the control point corresponding to a zero value for the load multiplier. This relation can be interpreted in Figure 9b as a linear capacity curve.

The load multiplier α_0 is directly computed from applying the PVW. However, the maximum displacement $d_{k,0}$ corresponding to a zero value of the load multiplier is determined with the evolution of the mechanism, which in fact provides a nonlinear relation in rotation. Then, complementarily to an acceleration-based approach (linear kinematic analysis), a displacement-based approach (nonlinear kinematic analysis) can be defined, e.g. as specified in [17].

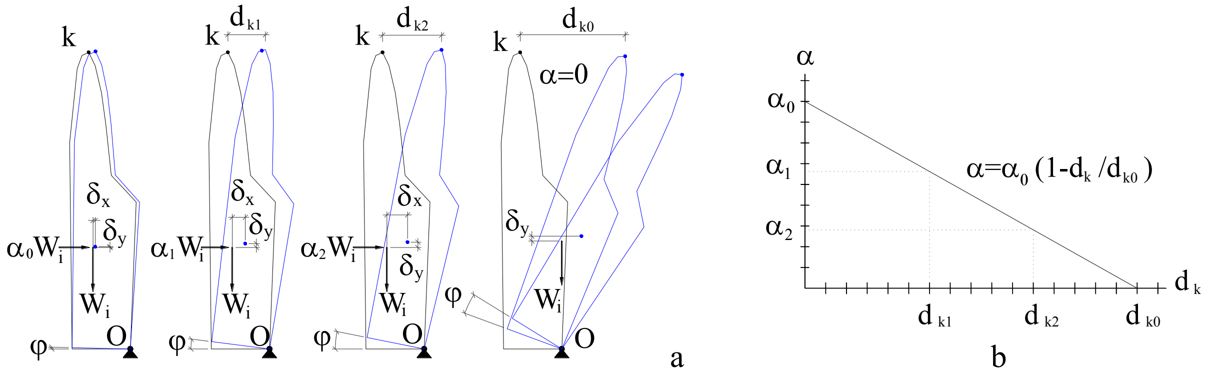


Figure 9: Evolution of mechanism: (a) motion sequence and (b) linear capacity curve.

For the safety verification, the capacity parameters need to be considered relatively to a single degree of freedom (SDOF) system, i.e. the capacity curve is transformed into a capacity spectrum, see [17]. For the case of a one-body mechanism, the spectral acceleration a^* is computed as the product of the collapse load multiplier, α_0, by the gravity acceleration, g. The spectral displacement d^* is computed by multiplying the real displacement of the control point k, d_k, by a modal participation factor, that in this case has a value of 1. Thus, a linear relation is also established for the capacity spectrum similar to that of Equation (3), where the collapse activation acceleration a_0^* is in function of the maximum spectral displacement d_0^*.

Even if according to the Italian building code [17], the seismic safety of buildings needs in general to be verified for damage and life safety limit states, in this work only the life safety limit state is considered as applicable to archaeological building remains. The safety verification is defined differently for two kinds of mechanisms: that developed at ground level and thus denoted as ‘global mechanisms’ (e.g. in Fig. 10a-b), and that activated at an upper level of the structure and denoted as ‘local mechanisms’ (e.g. in Fig. 10c). The two cases require, respectively, the verification of Expressions (3-4), where the right part of the expressions is to compute the spectral acceleration demand according to the Peruvian seismic code [18]:

$$a_0^* \geq \frac{a_g(P_{VR})S}{q} \equiv \frac{Z}{q}\cdot S\cdot g \qquad a_0^* \geq \frac{S_e(T_1)\psi(z)}{q} \equiv \frac{Z}{q}\cdot min\left(2.5\frac{T_p}{T_1}, 2.5\right)\cdot S\cdot g\cdot\psi(z) \qquad (3\text{-}4)$$

where, in the Italian building code, $a_g(P_{VR})$ is the reference peak ground acceleration at the site; S is the soil amplification factor; q is the behaviour factor (assumed with a value of 2); $S_e(T_1)$ is the elastic spectral acceleration evaluated for the fundamental vibration period of the structure, T_1; and $\psi(z)$ is the normalized first vibration mode of the structure as a function of the elevation z of the hinge. In the Peruvian seismic code, Z is the zoning coefficient and T_p is the period at the end of the plateau in the elastic acceleration response spectrum.

Beyond the acceleration-based verification, the Italian building code [17] also specifies safety verification in displacement. In this case, an ultimate spectral displacement d_u^* is defined correspondently to the life safety limit state as $0.4d_0^*$. Then, d_u^* is compared with the spectral displacement demand d_d^*, which is computed in function of a secant period T_s defined for the SDOF system as illustrated in Fig. 11a. Also here, cases of global and local mechanisms are considered, respectively requiring the verification of Expressions (5-6).

$$d_u^* \geq S_{De}(T_s) = \frac{T_s^2}{4\pi^2}S_e(T_s) \qquad d_u^* \geq S_{De}(T_1)\psi(z)\left[\frac{T_s}{T_1}\right]^2 \Big/ \sqrt{\left[1-\frac{T_s}{T_1}\right]^2 + 0.02\frac{T_s}{T_1}} \qquad (5\text{-}6)$$

where $S_{De}(T_s)$ is the elastic spectral displacement evaluated for the secant period T_s.

6 SEISMIC SAFETY OF THE CHOKEPUKIO WALL

The seismic assessment of archaeological building remains is a subject that has received no much attention. This is a matter of concern, particularly in countries presenting a large number of archaeological sites and high seismic hazard, as Peru. In this sense, this paper presents an articulation of simple and available tools, which allows the seismic assessment of building remains on a wide range of structural typologies.

Concerning the studied wall, three collapse mechanisms have been considered as presented in Figure 10, namely two global rocking motions and a partial rocking mechanism of the top part of the wall. The first global mechanism is purely a rigid-body motion, while the second mechanism is constrained by the timber struts that are shoring the wall. The activation of this last mechanism requires the reaching of the axial load strength of the wooden struts, which is considered as an external force to the wall of 185 kN. The third mechanism considers the rotation of the top part of the wall around a hinge located in the section with thickness change.

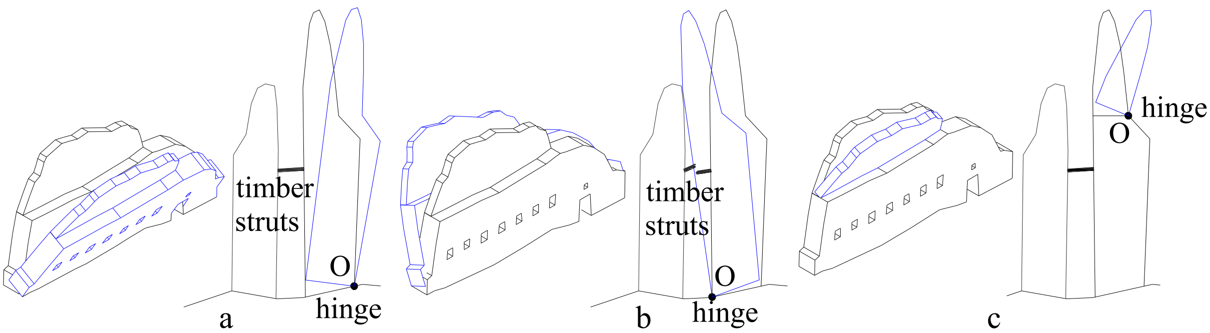


Figure 10: Considered mechanisms: (a) 1st global rocking, (b) 2nd global rocking and (c) partial rocking.

The limit analysis presents the advantage of requiring mostly a geometrical input, beyond the specific weight of the material. A tridimensional model of the wall corresponding to the calibrated geometry of the FE model was considered, and the optimized specific weight value was assumed. In this study, with only a body involved in the kinematisms, a virtual rotation is assumed around the edge hinge to apply the PVW as moment equilibrium (balance of the vertical and horizontal forces acting around the hinging point).

In this case, the seismic demand was defined according to the Peruvian seismic code [18], which establishes a reference peak ground acceleration at the site (Zg) of $0.3g$. Considering the local ground composed of graved deposits, the soil factor S was considered with a value of 1.2 (medium soil). The soil period T_p was in this case computed from the microtremor measurements at the ground level by computing the H/V spectrum [19], from which the predominant frequency was found in the range 2.0–2.5 Hz (0.4–0.5 s). Thus, an average value of 0.45 s was considered for the period T_p. The fundamental vibration period of the wall, T_1, was assumed as 0.53 s according to the modal analysis of the calibrated FE model.

The computed capacity spectra for the three considered mechanism are presented in Figure 11b, where it can be observed that the second global rocking mechanism requires the highest value of spectral acceleration ($0.27g$) for mechanism activation, mainly due to the constraint effect of the timber struts. On the other hand, the partial rocking mechanism presents the lowest activation acceleration. As shown in Table 2, the global rocking mechanisms are safe, both in terms of acceleration and displacement checks, while the partial rocking mechanism is unsafe and predictable to occur for a spectral acceleration of $0.17g$.

The partial mechanism results unsafe due to the considered amplification of the ground acceleration over the height of the wall, which implies a higher seismic demand. It is also noted as the check in terms of displacement provides in general higher safety factors in comparison to the force verification, which is mostly associated to the dissipation of the inertial horizontal load on the wall throughout the rocking motion.

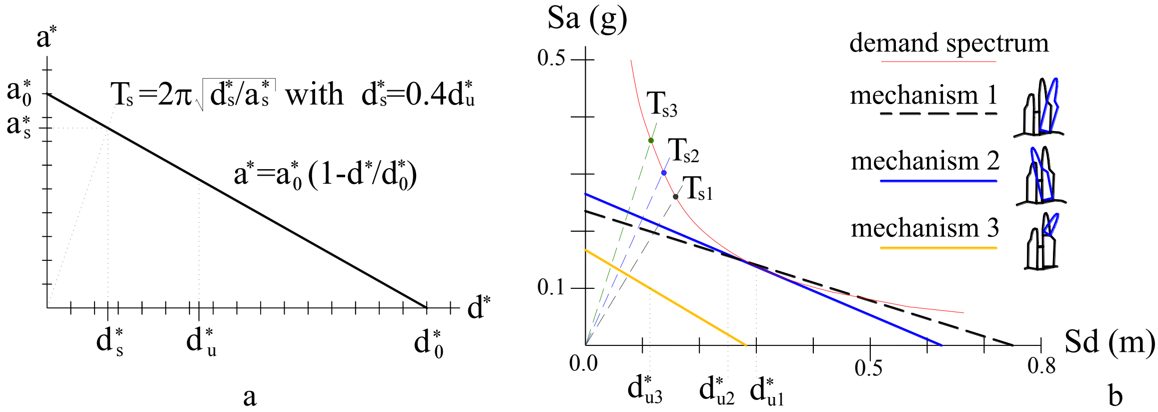


Figure 11: Capacity spectrum: (a) definition and (b) computation for the considered collapse mechanisms.

Table 2: Results of limit analysis and seismic safety verification.

Mechanism	α_0	Capacity a_0* (g)	Capacity d_u* (mm)	T_s (s)	Demand a_d* (g)	Demand d_d* (mm)	SF_a (g/g)	SF_d (mm/mm)	Check
1st global	0.24	0.24	300	1.53	0.18	155	1.33	1.93	safe
2nd global	0.27	0.27	250	1.33	0.18	132	1.50	1.89	safe
partial	0.17	0.17	113	1.13	0.20	123	0.85	0.92	unsafe

7 CONCLUSIONS

A multi-approach method for structural assessment of archeological remains is proposed, which includes on-site surveys, experimental tests and numerical-analytical computations. The application to a wall of Chokepukio allowed the development of a FE model with good structural representativeness against the ambient vibration dynamic tests, i.e. maximum difference in modal frequencies is less than 8% while the lowest MAC value is 0.86.

The principal frequencies of the wall were found in the range of 2 to 5 Hz, respectively varying from translational movements to oscillatory translational motions. The most significant variables to capture the modal response in frequencies were the E-modulus, which ranges from 210 to 580 MPa on the wall, and the specific weight (calibrated value close to 25 kN/m^3). The modal shapes are mostly determined by the geometrical aspects.

The seismic limit analysis to the studied wall denotes safety for global rocking mechanisms, i.e. collapse activation acceleration of 0.24g. However, the wall is unsafe when partial mechanisms are activated, i.e. collapse activation acceleration of 0.17g. Thus, most of the building remains in Chokepukio are probably vulnerable to seismic actions and require retrofitting. As it was observed that the timber struts, even if reacting with a small force, have strong influence in the safety, retrofit can be through local tying to avoid rocking mechanisms.

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