



Assessment of Combined In-Plane and Out-of-Plane Fragility Functions for Adobe Masonry Buildings in the Peruvian Andes

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Specialty section:

This article was submitted to
Earthquake Engineering,
a section of the journal
Frontiers in Built Environment

Received: 29 January 2020

Accepted: 31 March 2020

Published: 08 May 2020

Citation:

Sumerente G, Lovon H, Tarque N
and Chácará C (2020) Assessment
of Combined In-Plane
and Out-of-Plane Fragility Functions
for Adobe Masonry Buildings
in the Peruvian Andes.
Front. Built Environ. 6:52.
doi: 10.3389/fbuil.2020.00052

In the city of Cusco and in other parts of Peruvian Andes, adobe masonry is the primary construction material. Adobe continues to be used for the construction of housing because of its low cost, its thermal properties, the use of unskilled labor, and the local traditions of the Peruvian highlands. The Peruvian National Statistics Office estimates that 67% of rural housing in Cusco is made of adobe masonry. Besides, previous seismic events and laboratory tests demonstrated that adobe dwellings without reinforcement are prone to collapse during an earthquake. Therefore, seismic vulnerability assessment of this type of dwellings is necessary aiming at developing proper contingency and mitigation risk policies. Then, fragility curves constitute a key tool when conducting seismic loss assessment because they provide information regarding the probability of exceeding a certain damage limit state as a function of a given engineering demand parameter. This work aims at developing fragility curves, combining in-plane and out-of-plane loading conditions, for typical adobe buildings located in the city of Cusco. Initially, a set of one- and two-story adobe houses were studied to determine the geometrical characteristics of representative local building typologies. Subsequently, 1,000 artificial buildings were generated by means of Monte Carlo simulation based on the information gathered. The structural capacity of each artificial building was represented by simplified bilinear and trilinear capacity curves for in-plane and out-of-plane mechanisms, respectively. In order to represent the characteristics associated with subduction processes, a set of ground motion records was established. The damage state of each building was assessed for each seismic record, and this information was collected into a Probability Damage Matrix (DPM). Finally, fragility curves were fitted for each damage state of the cumulative DPM. Preliminary results show that one- and two-story adobe dwellings have a probability of collapse of 30 and 60%, respectively, when considering a peak ground acceleration of 0.30 g, which corresponds to the expected acceleration related to a return period of 475 years over a soil type 2 according to the Peruvian Standards.

Keywords: adobe, seismic vulnerability, capacity curves, out-of-plane failure, in-plane failure

INTRODUCTION

Fragility assessment constitutes an important component when determining the seismic risk of a region because it is capable of measuring how prone buildings are to suffer damage due to a given intensity of ground motion (Borzi et al., 2008). Seismic vulnerability assessment is usually focused on structures characterized by the presence of rigid diaphragms whose global response is ruled by in-plane mechanisms. Nevertheless, this is not the case of adobe masonry buildings because their structural performance is given by the combination of in-plane and out-of-plane mechanisms due to the lack of a box-type behavior. This particular response has been observed in previous earthquakes, as well as in laboratory tests (Ahmad et al., 2011; Tarque et al., 2012).

The present investigation focuses on the seismic vulnerability assessment of one- and two-story adobe dwellings located in the city of Cusco, Peru. For this purpose, Probability Damage Matrices (DPMs), together with fragility curves, are developed for these structures considering two types of failure mechanisms: in-plane and out-of-plane. For this, it is necessary to determine capacity curves for each mechanism and for each building. On the one hand, the capacity for the in-plane mechanism can be adequately represented by a collapse multiplier developed by Benedetti and Petrini (1984). Such multiplier has been used in different methodologies such as the VULNUS method proposed by Bernardini et al. (1990) for masonry buildings, the FaMIVE method by D'Ayala and Speranza (2002) for historical constructions, in Cosenza et al. (2005) for reinforced concrete constructions. In addition, the simplified pushover-based earthquake loss assessment (SP-BELA) method has also been applied for masonry structures as reported by Borzi et al. (2008). There are other investigations that use a similar methodology, such as Crowley et al. (2004); Vicente et al. (2011), Neves et al. (2012); Formisano et al. (2017), Villar-Vega et al. (2017); Lovon et al. (2018), Chieffo and Formisano (2019), and Chieffo et al. (2019). On the other hand, the capacity associated with out-of-plane mechanisms has been evaluated by means of a trilinear displacement versus force curve. This curve was proposed by Doherty et al. (2002) and further evaluated by Griffith et al. (2003), for the evaluation of rocking of masonry walls without reinforcement. It is important to notice that historic buildings are not evaluated in the present work; these kinds of structures need other considerations in their analysis, as are described in Briceño et al. (2019) and Fuentes et al. (2019).

In this research, the seismic vulnerability of unreinforced adobe masonry dwellings has been evaluated considering the combination of in-plane and out-of-plane collapse mechanisms. For this purpose, a random sample of adobe buildings was generated using the Monte Carlo simulation. The generation of this synthetic database is based on a detailed survey carried out to define geometrical characteristics of typological adobe buildings. Furthermore, the capacity of each building was calculated by the definition of simplified bilinear and trilinear capacity curves, respectively. The analysis combines both capacity curves, which were compared with the seismic demand, computed from records that represent the seismicity of the studied area. Based on the performance of each synthetic buildings. Probability Damage

Matrices were developed, and subsequently, analytical fragility curves were properly derived.

METHODOLOGY

Crowley et al. (2004) classify the methodologies for vulnerability assessment in three categories: empirical, hybrid, and analytical. In the empirical method, the assessment is based in the damage observed from past earthquakes, making use of the DPM and the vulnerability functions based on expert judgment. In the hybrid method, the DPM and vulnerability functions combine postearthquake damage statistics with simulations. The analytical method convolves the ground motion demand and capacity of the stock of buildings; the capacities are modeled analytically. Considering the variation in capacity and demand, the probability of being or exceeding limit states (LSs) are defined. This work used the analytical method to obtain the fragility curves.

On one side, seismic vulnerability assessment is often oriented to evaluate the uncertainty in the response of a given structural class. This procedure may require the analysis of a large number of buildings, which constitutes an arduous and long-lasting task. It is because of that fast analysis methodologies were adopted. On the other side, the geometrical and mechanical properties of the building class should be properly represented. In this way, field surveys play an important role. A field survey was carried out in some zones of Cusco city to gather information of one- and two-story buildings. This survey allowed the statistical characterization of adobe buildings, namely, mean value, standard deviation, and probability density function. A synthetic sample was generated using Monte Carlo simulation.

The in-plane mechanism was evaluated using the SP-BELA methodology; this uses the collapse multiplier and the LSs. On the other hand, the out-of-plane behavior was represented by a displacement–lateral force curve assessed according to Doherty et al. (2002). The displacement method (Silva et al., 2013) is adopted to define if a dwelling will survive or not to a given input motion. The general idea is to compare the capacity displacement with the spectral displacement demands, obtained from a group of seismic records. Special attention is done to the out-of-plane actions because it is the first mechanism to analyze. If the building does not collapse under out-of-plane, then the in-plane capacity is evaluated for each LS. This procedure allows to have combined fragility curves considering both failure mechanisms.

TPOLOGY OF ADOBE BUILDINGS

The characterization of the building stock in Cusco was carried out through fill surveys. Those were prepared in such a way to collect important information as number of stories, dimension of walls and openings, roof system, and wall distribution, among others. The surveys were developed for one- and two-story buildings because the adobe dwellings located in such area are usually built up to two levels (**Figure 1**).

Aiming at assessing the seismic vulnerability of this type of structures, 13 and 22 adobe dwellings were surveyed for one-

and two-stories, respectively. Based on these surveys, it was possible to determine the statistical characteristics of the main geometrical parameters that influence their seismic behavior. It is worth to note that walls, including the presence of door or window openings, with a length less than 1 m were not considered in this investigation because they may not present a significant contribution to the shear strength of the structure. In addition, it was observed that some two-story buildings were characterized by the presence of *quincha* walls for the division of large areas (Figure 2). These walls are composed of a timber frame filled with intertwined cane and mud, and they presented a maximum thickness of 0.15 m. For this investigation, the *quincha* walls were considered as an additional mass to the buildings, but its contribution to the global strength of the structure was neglected. In the two-story dwellings, the timber diaphragm was composed of wooden floor joist distributed along the shortest direction, on these rest a tongue-and-groove floor. This type of floor system does not increase significantly the lateral stiffness of the structure because it constitutes as a flexible diaphragm. All the walls parallel to the facade were considered in the Y-axis, and the perpendicular walls in the X-axis.

After the adobe dwellings were surveyed, the collected information was processed, allowing the statistical

characterization of adobe buildings. For instance, the mean value of the wall thickness was 0.5 m, whereas the average height for buildings was 2.5 and 4.9 m for one- and two-story buildings, respectively. It was also evidenced that the constructed area for one-story dwellings presented a mean value of 38.8 m². In the case of two-story dwellings, this value increased to 50.50 m². A summary of the parameters defined for this investigation, together with their corresponding mean value (μ) and standard deviation (σ), is reported in Table 1.

STRUCTURAL CAPACITY ASSESSMENT

The behavior of masonry structures characterized by a flexible diaphragm is governed by in-plane and out-of-plane failure mechanisms. However, Borzi et al. (2008) reported that previous earthquakes have demonstrated that there is no interaction between these mechanisms because one can occur despite the other, and therefore, it can be assumed that these responses are uncoupled. In this sense, the assessment of the seismic vulnerability of the adobe dwelling required the definition of independent capacity curves for each mechanism, together with the corresponding damage LSs.

In-Plane Failure Mechanism

The structural capacity of a building can be properly represented through a pushover failure mechanism, which relates the base shear force due to the distribution of lateral forces and the displacement at a control point (Borzi et al., 2008). From a probabilistic point of view, an accurate estimation of the structural capacity of a building is not usually required; hence, simplified approaches, such as the SP-BELA method, are frequently used to determine the capacity curve of structures. The application of these simplified approaches provides results with a reasonable accuracy when compared to those obtained by means of more sophisticated methodologies and with a significant reduction of the computational effort for the estimation of capacity curves. A bilinear pushover curve represents the response of adobe masonry structure subjected a cyclic test (Tarque et al., 2012; Montesinos, 2016).

The objective of structural analysis is to determine the level of damage of a structure, which is usually measured in terms of the lateral displacement or interstorey drift. In order to carry out the evaluation, limits are established; thus, the damage assessment



FIGURE 1 | Typical surveyed buildings. (A) One- and (B) two-story adobe dwellings.

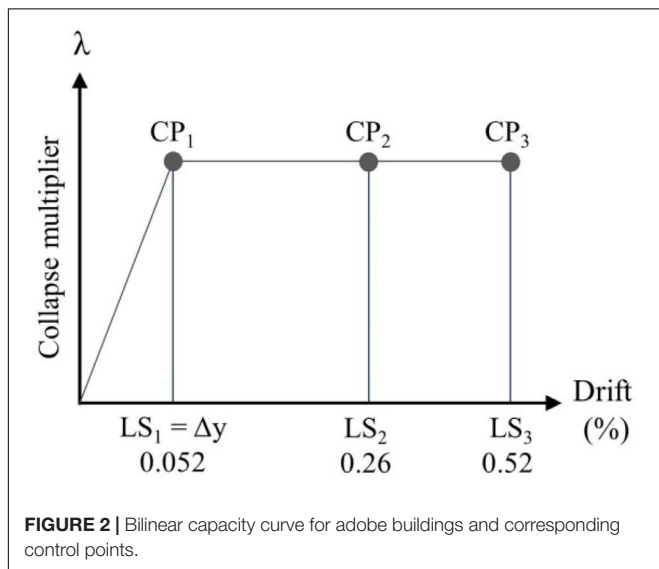


FIGURE 2 | Bilinear capacity curve for adobe buildings and corresponding control points.

TABLE 1 | Identification of uncertain parameters and statistical characteristics.

Parameter	Symbol	Unit	One-story		Two-story	
			μ	σ	μ	σ
Wall density in X direction	ρ_x	—	0.17	0.07	0.16	0.04
Wall density in Y direction	ρ_y	—	0.17	0.10	0.12	0.02
1st story height	h_1	m	2.47	0.24	4.92	0.47
2nd story height	h_i	m	—	—	2.46	0.23
Area	A_T	m ²	38.83	26.60	50.50	26.85
Wall thickness	t	m	0.46	0.05	0.49	0.04

can be carried out in a discrete manner. When a structure is subjected to a lateral force, it develops displacements that define its strength and level of damage. Three possible drifts are considered as points of control (CP), which are related to the LSs and are defined as follows: light damage, moderate damage and collapse (Figure 3). Based on Borzi et al. (2008), after the earthquake occurred, in the light damage (LS₁), the building can be used without repair or reinforcement. In the significant damage (LS₂), the building can be used whenever reinforcement has been done, which means a high cost in the repair. In the collapse (LS₃), the building is not able to resist lateral forces and even loads of gravity, resulting unsafe for the inhabitants.

The limiting conditions for adobe constructions have been identified by experimental tests; the damage has been related to a drift capacity. The LS values obtained for adobe walls from incremental displacement tests carried out by Tarque et al. (2012) are indicated in Figure 3. Control points identify important stages in the in-plane behavior of masonry buildings: CP₁ is considered the limit elastic where the building is nearly undamaged and cracks up to 0.3 mm thick; in CP₂, the building is extensively damaged but repairable with horizontal cracks beginning and new cracks up to 1.6 mm thick, and in CP₃, the collapse is expected with cracks larger than 5 mm thick.

The representation of the capacity curve is given by a lateral force versus displacement graph. The SP-BELA method by Borzi et al. (2008) proposes the use of an equivalent system, thus reducing the system of multiple degrees of freedom (MDOFs) to a system of a single degree of freedom (SDOF) equivalent in terms of mass, stiffness, and displacement capacity. To

define the displacement capacity of the equivalent structure, it is necessary to assume a deformation for each LS. Figure 4, which was built according to Borzi et al. (2008), illustrates these considerations for a two-story building. In MDOF, m_i is the mass considered in a story height h_i , the soft mechanism is predicted within a height h_p and h_T is the total height of the building; h_e and m_e is the equivalent height and the equivalent mass, respectively, in SDOF.

When the deformation of the structure is elastic, it is considered a linear form. The first control point (CP₁) is determined by the following expression:

$$\Delta_y = k_1 \cdot h_T \cdot \delta_y \tag{1}$$

where k_1 is the elastic coefficient, h_T is the total building height, and δ_y is the drift of the elastic behavior (Restrepo-Vélez and Magenes, 2004). The product of the last two values provides the height of the equivalent building (h_e). The development of a plastic deformation occurs at a certain level, and it is assumed that the damage is concentrated in a height (h_p). Then, the soft floor mechanism is predicted (Borzi et al., 2008). Control points (Δ_i) for the structure in the inelastic range is expressed as follows:

$$\Delta_i = k_1 \cdot h_T \cdot \delta_y + k_2 \cdot h_p \cdot (\delta_{CP_i} - \delta_y) \tag{2}$$

where k_2 is the relation between the total height of the MDOF system and the height of the equivalent SDOF system, h_p is the height of the weakest level and where the inelastic behavior of the structure begins, and δ_{CP_i} is the drift of the inelastic behavior in a certain control point. In order to define which floor of the construction will be the first to enter into an inelastic range, it

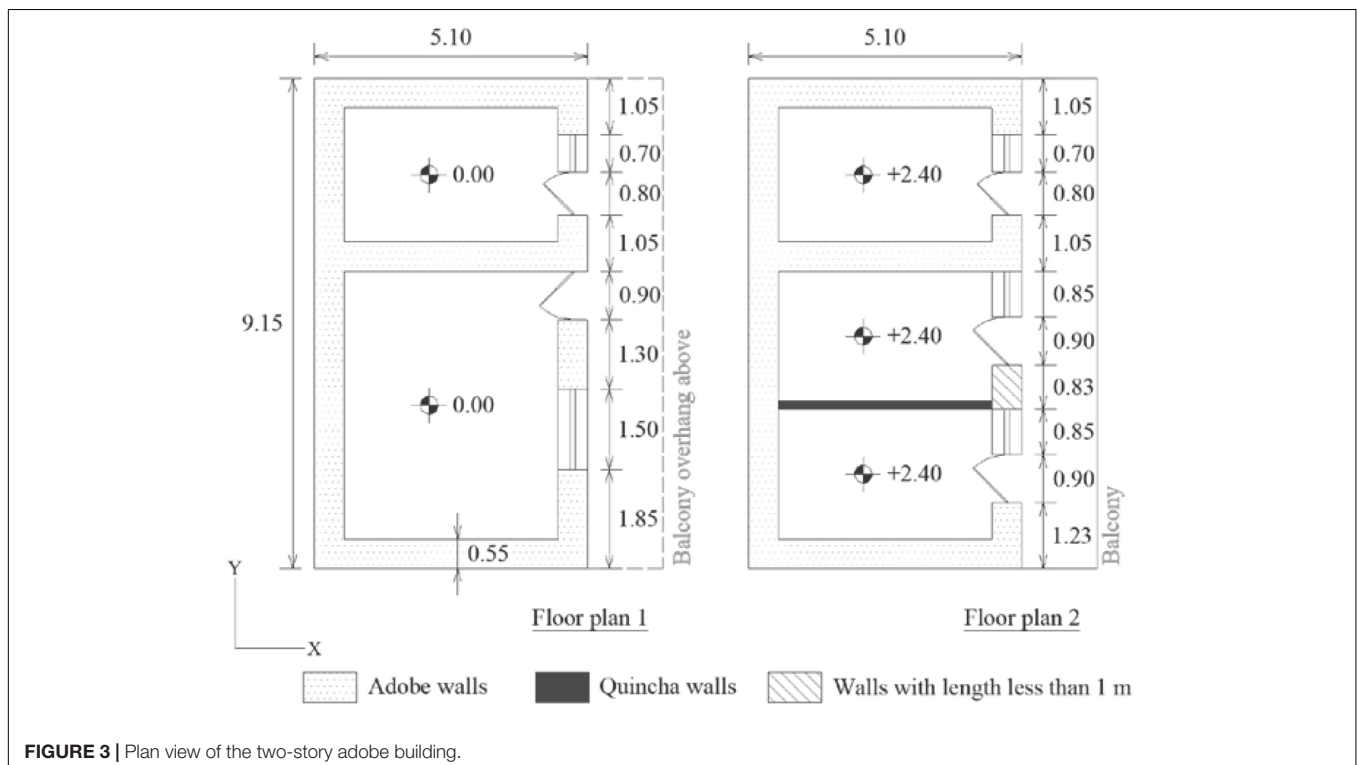


FIGURE 3 | Plan view of the two-story adobe building.

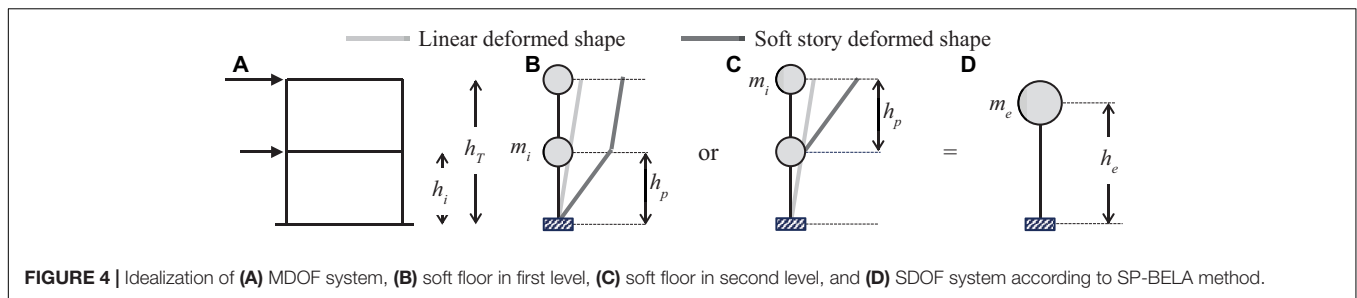


FIGURE 4 | Idealization of (A) MDOF system, (B) soft floor in first level, (C) soft floor in second level, and (D) SDOF system according to SP-BELA method.

is necessary to determine the collapse multiplier λ_i in X and Y directions. For this purpose, the expression provided by Benedetti and Petrini (1984) and reported in Eq. (3) is used.

$$\lambda_i = \frac{1}{W_T \cdot \frac{\sum_{k=i}^n h_k \cdot W_k}{\sum_{j=1}^n h_j \cdot W_j}} \cdot A_i \cdot \tau_{ki} \cdot \left[1 + \frac{\sum_{k=i}^n W_k}{1.5 \cdot A_i \cdot \tau_{ki} \cdot (1 + \gamma_{AB})} \right]^{1/2} \quad (3)$$

where W_T is the total weight of the building, W_i is the weight of the i th floor where the soft floor is predicted, τ_{ki} is the diagonal compression resistance of the adobe at the i th floor, A_i is the total shear wall area in the direction of application of load in the i th level, n is the number of stories, and γ_{AB} is the relation between A_i and C_i , with C_i being the maximum area between the wall area in the loaded direction (A_i) and the orthogonal direction (B_i). For instance, if the analysis is in direction X, the values of A_i and B_i are the product between the building surface (A_T) and the wall percentages (ρ_X, ρ_Y) such that $A_i = \rho_X \cdot A_T$ and $B_i = \rho_Y \cdot A_T$, respectively; moreover, $C_i = \max\{A_i, B_i\}$ and with a consideration that i indicates the level where soft story was predicted, the multiplier λ_i is calculated using Eq. (3); the same process is made in direction Y. However, for this direction, $A_i = \rho_Y \cdot A_T$ and $B_i = \rho_X \cdot A_T$, and as a result, a new λ_i is obtained and compared; the minimum value will be the collapse multiplier.

Out-of-Plane Failure Mechanism

The response of adobe buildings, when subjected to lateral loading, is also characterized by out-of-plane collapse mechanisms. These mechanisms are mainly due to the poor connection between orthogonal walls or the inadequate interaction between walls and roof system. During the occurrence of these mechanisms, the restitutive force is given by the self-weight of the wall. To describe these mechanisms, a cantilever wall and a simply supported wall, which span vertically between supports at ceiling or floor levels, can be considered; the configurations assumed for rocking to assess one- and two-story adobe buildings are shown in Figures 5A,B, respectively.

In a similar way to the in-plane failure mechanism, the capacity of a wall can be represented by means of a force-displacement curve of an equivalent SDOF system. This capacity curve describes the behavior caused by the seismic input into the structure. Doherty et al. (2002) idealized a non-linear SDOF model in order to analytically obtain the displacement profile of a rocking wall. In this model, walls without reinforcement could behave as rigid bodies, which are balanced at pivot points,

which allowed to determine the capacity curve for out-of-plane mechanism with a bilinear force-displacement relation. For a parapet wall (Figure 5A), the pivot point is in the base, and for a simply supported wall (Figure 5B), it is assumed that the pivot points are in the base and in the wooden floor. In this approach, the restorative force is produced by the self-weight. Results in experimental F-d relationship assume a curvilinear profile, and it is observed that it varies significantly to the theoretical bilinear model; the resistance to rocking is reduced from the bilinear model to a force plateau in a trilinear model. For these reasons, the curvilinear profile can be idealized by a trilinear model as shown in Figure 6.

The bilinear curve can be defined by F_o and K_o . The following expressions were proposed by Doherty et al. (2002) based on the boundary conditions in Figure 5.

$$F_o = M_e \cdot g \cdot \frac{t}{h} \quad (4)$$

$$F_o = 4 \cdot M_e \cdot g \cdot \frac{t}{h} \quad (5)$$

where F_o is the force, and g is the gravity. In buildings with uniformly distributed mass, the effective mass M_e for the boundary condition used corresponds to 75% of the total mass M of the wall. K_o is the stiffness of the slope for the wall associated with F- Δ , and it is defined as $1.5M_e(gt/h)$ and $6M_e(gt/h)$ for the boundary conditions (Figure 5), respectively.

It is observed that the relation between the experimental non-linear curve deviates from the bilinear one (Figure 6). This is

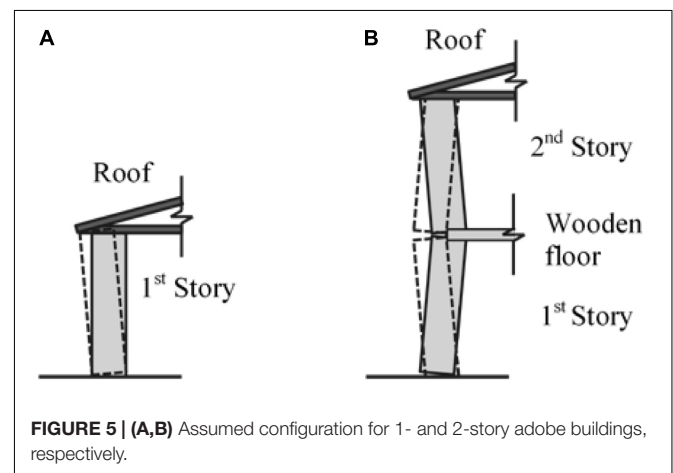


FIGURE 5 | (A,B) Assumed configuration for 1- and 2-story adobe buildings, respectively.

due to the non-linear deformations occurring in the mortar joint. The experimental curve can be modeled by a trilinear curve, which is defined by three parameters: Δ_1, Δ_2 , and Δ_u . Δ_u is the ultimate displacement, which means the point of static instability; if the displacement is greater than Δ_u , the wall will collapse; Δ_u is considered to be $2/3t$. The amplitude of the force plateau is controlled by the displacement Δ_2 , and the force plateau is $(\Delta_2 - \Delta_u)K_0$. The relations Δ_1 and Δ_2 are related to the properties of the materials and the state of degradation of the mortar joints. Doherty et al. (2002) suggest nominal values for the ratios Δ_1/Δ_u and Δ_2/Δ_u for walls in “new,” “moderately,” and “severely degraded,” these ratios and degradation are the results of 14 simply supported walls in quasi-static and dynamic test.

SEISMIC FRAGILITY ASSESSMENT

After the generation of the initial database of the geometrical characteristics of adobe dwellings representative of the city of Cusco, a group of synthetic buildings was created by means of Monte Carlo simulations. The assessment of the seismic vulnerability of the synthetic building required determining the capacity curves associated with in-plane and out-of-plane loading conditions. For this purpose, it was necessary to define additional uncertain parameters related to mechanical properties of the material and additional variables required for the estimation of in-plane and out-of-plane capacity curves. Using a group of seismic records, it was possible to obtain the seismic demand. Finally, fragility curves were computed by comparing such capacity curves to the demand. A flowchart of the methodology used for the derivation of fragility curves is illustrated in Figure 7.

Determination of Capacity Curves

The derivation of fragility curves for the seismic vulnerability of adobe dwellings in the city of Cusco required the estimation of their structural performance. In this sense, the in-plane and out-of-plane responses of this type of buildings were evaluated by means of bilinear and trilinear capacity curves, respectively. In addition, these capacity curves required the definition of uncertain parameters, which are associated with

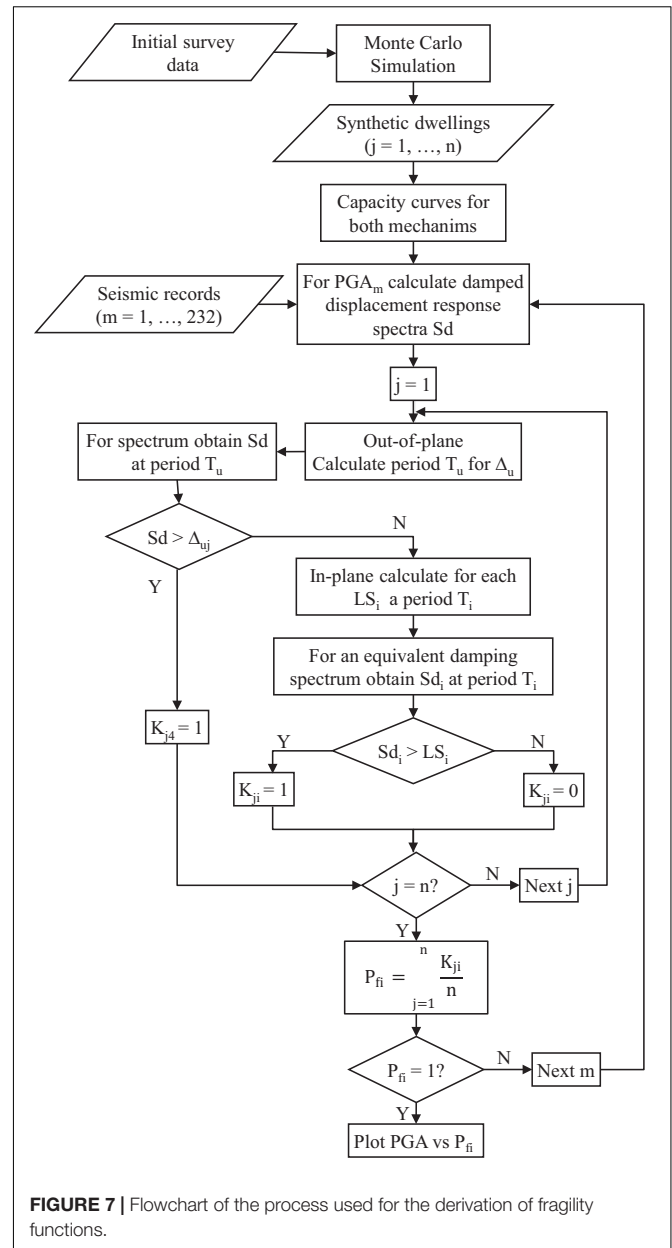


FIGURE 7 | Flowchart of the process used for the derivation of fragility functions.

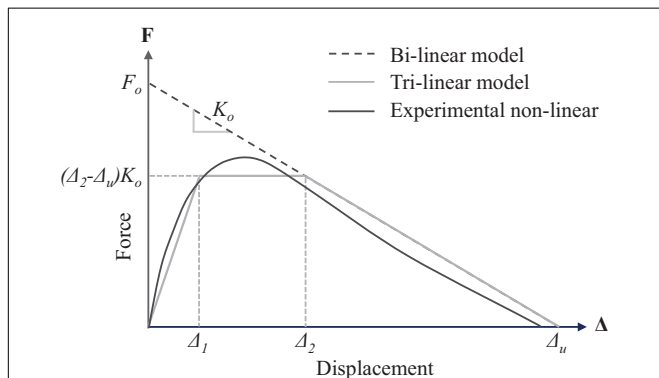


FIGURE 6 | Force-displacement relationship for out-of-plane. Adapted from Doherty (2000) with permission from K. Doherty.

geometrical and mechanical properties of these buildings. During the building survey, all geometrical variables that were involved in the analysis, together with their corresponding statistical characteristics, were properly identified. It is worth noting that all the statistical characteristics of some uncertain parameters such as mechanical properties of adobe, death load, and live load in floor and roof were not obtained from the building survey. Therefore, it was necessary to define these properties based on values reported in literature. For instance, the diagonal compressive strength of adobe masonry τ_{ki} was obtained from the investigation conducted by Tarque et al. (2012). The remaining parameters were defined, taking into consideration the Peruvian code E-020 (NTC E.020, 2006). These additional uncertain parameters, as well as their corresponding

TABLE 2 | Parameters used for the estimation of the capacity curves of the synthetic buildings.

Variable	Description	Units	μ	σ
γ_a	Specific weight of adobe	kN/m ³	16	2.5
τ_{ki}	Shear resistance of masonry adobe walls	N/m ²	4000	500
γ_w	Specific weight of wood	kN/m ³	10	0.5
γ_t	Specific weight of tiles	kN/m ³	16	0.5
LL_f	Live load in floor	N/m ²	2000	200
LL_r	Live load in roof	N/m ²	1000	200
DL_f^*	Dead load in floor	N/m ²	550	100

* μ represents the value for a wooden floor joist with logs (diameter of 100 mm) distributed every 0.50 m and the tongue-and-groove floor over this.

statistical characteristics, are summarized in **Table 2**. The probability density function defined for all uncertain parameters corresponded to a normal function because it was assumed that it fitted better to the data obtained from the surveys.

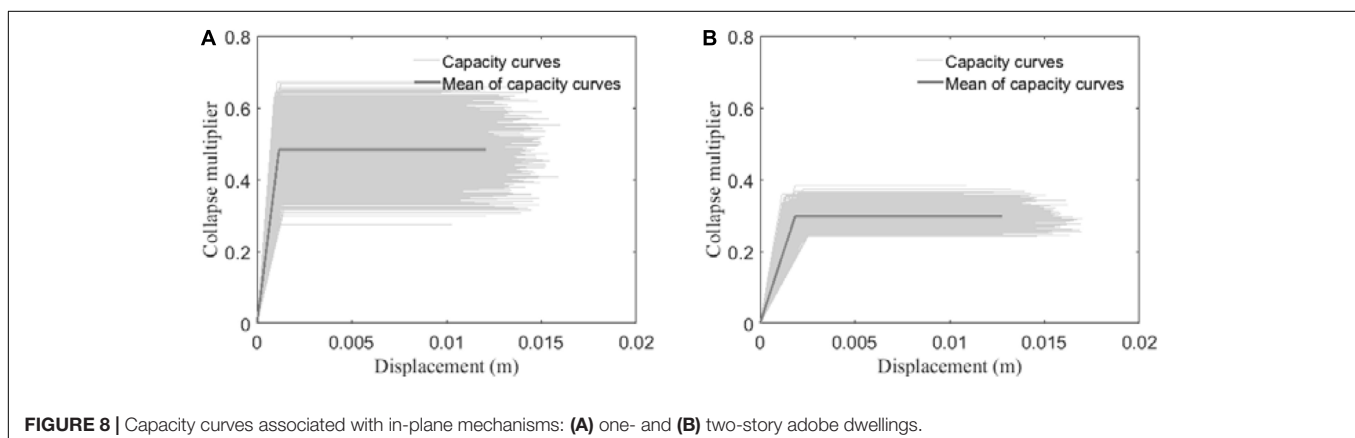
Monte Carlo simulations were used to create 1,000 synthetic dwellings, keeping the initial statistical properties and considering that these statistical properties were truncated at 80% for more reliable results. The variables k_1 and k_2 presented a deterministic behavior, and they were calculated using the expressions reported by Restrepo-Vélez (2003). For one-story adobe dwellings, the average value for k_1 is 0.79, and the value of k_2 is 0.97. In the case of two-story dwellings, the values of k_1 and k_2 presented values of 0.72 and 0.95, respectively. **Figure 8** shows the capacity curves for the 1,000 synthetic buildings. The collapse multipliers of one-story dwellings range between 0.3 and 0.7. **Figure 8A** also shows the elastic and the collapse limits with average values of 1.2 and 12.2 mm, respectively. Considering that the average height is 2.47 m, the drifts for elastic and collapse limits correspond to 0.049 and 0.49%, respectively. Similarly, in **Figure 8B**, the collapse multipliers associated with two-story buildings range between 0.25 and 0.4, whereas the mean of elastic and collapse limits are 1.9 and 12.8 mm, and with an average height of 2.46 m for two-story, the drifts are 0.077 and 0.52%, respectively.

The seismic vulnerability assessment also required the definition of capacity curves associated with the out-of-plane

response of the synthetic adobe dwellings. In this process, considering the boundary conditions (**Figures 5A,B**) and using Eqs. (4) and (5), it was possible to obtain a bilinear relation. For this purpose, it was necessary to obtain the displacement capacity that is a function of the wall thickness t . In the present work, the ratios are considered severely degraded because of the fact that the units of adobe are joined by mud mortar. In addition, in some walls, there is no protection at the base (pivot point), and they are exposed to the rain effects that degraded the effective width. Another consideration is that the main age of adobe dwelling surveys is 28 years, which means that there is an important degradation of the mortar joints. The values for the previous consideration suggested by Doherty et al. (2012) were 0.20 for Δ_1/Δ_u and 0.50 for Δ_2/Δ_u , respectively. Capacity curves are represented with a seismic coefficient, which is the ratio of the horizontal force (F_o) and the wall weight. **Figure 9A** shows the value of seismic coefficient that ranges between 0.05 and 0.1 for one-story dwellings, whereas the seismic coefficient for two-story varies between 0.13 and 0.17 (**Figure 9B**). Both cases present results according to what it was expected: the seismic coefficients for two-story buildings are greater than those related to one-story dwellings; the model in Doherty et al. (2002) indicates that for a supported wall F_o is greater than for a parapet wall.

Derivation of Fragility Curves

In this stage, with the data of synthetic buildings and their respective capacity curves, it is important to evaluate the performance of these buildings using the seismic demand, which is a representation of the earthquake ground motion (seismic input). In this process, the effect of the record-to-record variability of the seismic input was used in order to get realistic results (Lovon et al., 2018). The city of Cusco is located in an area where, due to presence of geological faults, the main seismogenic source produces cortical earthquakes. The shallowness of this type of earthquakes produces damage that resembles seismic events occurring because of a subduction process (Tavera, 2017). Then, because of the lack of records for the main source for Cusco, a database of 232 ground motions, which were recorded from subduction mechanism by the Pacific Earthquake Engineering Research Center (PEER), was selected for this investigation (for further details see

**FIGURE 8** | Capacity curves associated with in-plane mechanisms: (A) one- and (B) two-story adobe dwellings.

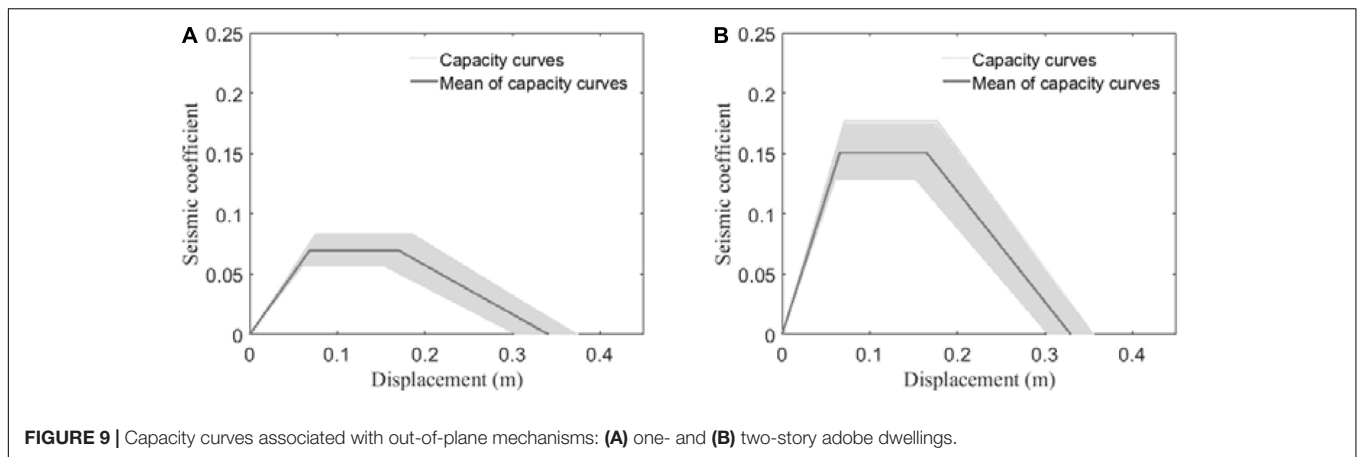


FIGURE 9 | Capacity curves associated with out-of-plane mechanisms: **(A)** one- and **(B)** two-story adobe dwellings.

Lovon et al., 2018). This database contains a great variety of local seismic hazard characteristics such as magnitude, range of accelerations, frequency content, and duration that may represent an earthquake at the Peruvian Coast and could also affect Cusco. **Figure 10** shows the pseudospectral accelerations for all the records that have magnitudes between 5 and 9 Mw. The soil factor is considered for rock soil because of origin of the records.

Based on the method proposed by Silva et al. (2013), it was necessary to calculate the displacement demand for each LS. First, the demand for in-plane mechanism was calculated using the equivalent viscous damping ζ_{LSi} for each LS. This equivalent damping was obtained using the expression reported in Eq. (6) as proposed by Calvi (1999):

$$\zeta_{LSi} = a \cdot \left(1 - \frac{1}{(\mu_{LSi})^b} \right) + \zeta_{el} \quad (6)$$

where ζ_{el} is the damping for the elastic response that is equal to 5%; μ_{LSi} is the displacement ductility for the i th LS, whereas

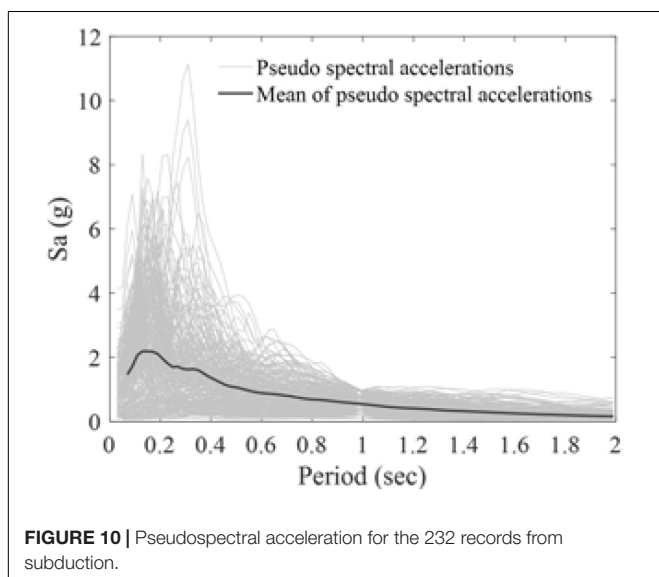


FIGURE 10 | Pseudospectral acceleration for the 232 records from subduction.

a and b are two coefficients [both depending on the hysteretic properties of the structure (Tarque et al., 2012)] with values of 25 and 0.5, respectively. For the derivation of the fragility curves, it was necessary to establish the damage thresholds based on the LS. **Table 3** shows the damage states proposed by Tarque et al. (2012). On the other hand, according to Griffith et al. (2003), the evaluation of the out-of-plane response does not require changing the 5% damped elastic displacement response spectrum, and the unique threshold is collapsed.

Restrepo-Vélez and Magenes (2004) proposed Eq. (7) for in-plane in order to obtain the period (T) for each LS_i , being λ the minimum collapse multiplier calculated with Eq. (3):

$$T_i = 2\pi \cdot \sqrt{\frac{LS_i}{\lambda \cdot g}} \quad (7)$$

In out-of-plane, it is necessary to use a secant stiffness (K_{ef}) in order to represent a substitute structure of an MDOF system. The K_{ef} is obtained from force-displacement curve corresponding to the point of maximum displacement (Δ_2), which is described in Doherty et al. (2002) as the point with more consistent notion. The K_{ef} for the equivalent SDOF system is obtained by the following:

$$K_{ef} = K_o \cdot \left(\frac{\Delta_2 - \Delta_u}{\Delta_2} \right) \quad (8)$$

where K_o is the average secant stiffness (**Figure 6**). Hence, the period for out-of-plane is obtained by the following expression:

$$T = 2\pi \cdot \sqrt{\frac{M_e}{K_{ef}}} \quad (9)$$

TABLE 3 | Damage thresholds associated with limit state.

Limit state	Damage threshold
LS1—light damage	0.75 * CP ₁
LS2—moderate damage	1.9 * CP ₁
LS3—extensive damage	CP ₂
LS4—collapse	CP ₃

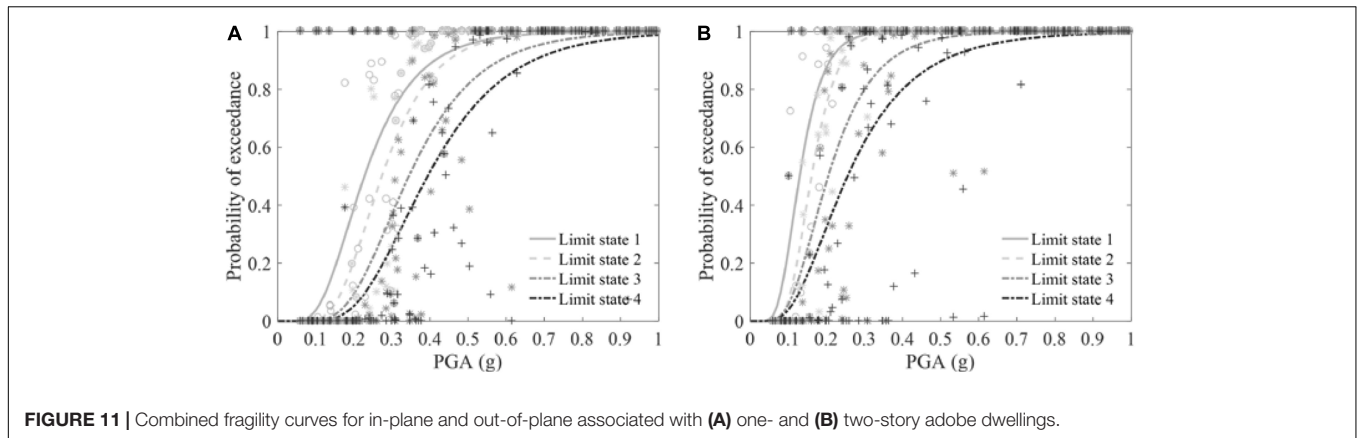


FIGURE 11 | Combined fragility curves for in-plane and out-of-plane associated with (A) one- and (B) two-story adobe dwellings.

Initially, the capacity curves for both mechanisms were obtained for all the synthetic dwellings. Each dwelling was evaluated for a sequence of records in order to obtain its seismic performance; thus, when the seismic demand exceeded a certain LSs, it was progressively recorded in the DPM until a record exceeded an LS4. The process started by evaluating initially the out-of-plane mechanism. It was possible to recognize the performance of the dwelling by comparing the demand and the displacement. In case the demand exceeded Δ_u , then the structure collapsed; otherwise, the dwelling was capable of withstanding the load, and the analysis continued with the assessment of the in-plane mechanism in which the demand was compared to the capacity for the different LSs. Subsequently, the DPM was assembled progressively after all synthetic buildings were subjected to all the records. From the DPM, it was possible to obtain the Cumulative Probability Damage Matrix (CDPM), which is interpreted as a point cloud that indicates the percentage of exceeding an LS. Finally, the point cloud was adjusted to an accumulated log-normal function and fragility curves obtained (Figure 11). According to Ericksen et al. (1954), the 1950's Cusco earthquake had a peak ground acceleration (PGA) of 0.3 g. Then, the probability of exceeding the collapse state for this earthquake is 30 and 60% for 1- and 2-story buildings, respectively, according to the computed fragility functions.

CONCLUSION

The objective of the research was to derive fragility curves for one- and two-story adobe buildings in Cusco, using the SP-BELA method proposed by Borzi et al. (2008). One of the modifications applied to such methodology allowed the development of combined in-plane and out-of-plane fragility curves. The importance of these functions relies on the combination of both failure mechanisms, which is usually addressed using logic trees, and in the current work, it is implemented by analyzing both mechanism in the sample of buildings. The probabilistic work carried out allows covering the uncertainty of the material and the geometric characteristics of the constructions.

The adobe buildings around the city of Cusco need to improve their seismic performance by means of external reinforcement.

For a PGA value of 0.3 g, approximately 30 and 60% of collapse is expected in one- and two-story buildings, respectively. The 0.3 g is the maximum horizontal acceleration expected to occur in Cusco over soil type 2 (V_s between 180 and 500 m/s) according to the Peruvian Seismic Code E-030 (NTC E.030, 2018) (return period of 475 years). This assessment may not be applicable to dwellings composed of mixed materials as stone and adobe, basically located at the historic center, and they may need another approach.

For a further study, it is necessary to assess fragility curves with records of Cusco. The database of PEER for subduction was used to evaluate the seismic hazard in the area in study because of a lack of records.

Based on the obtained fragility curves, it was possible to generate vulnerability curves that may help decision makers to create efficient contingency plans. This work can also be extended to all the adobe dwellings that maintain a similar characterization in the Peruvian Andes.

DATA AVAILABILITY STATEMENT

The datasets generated for this study are available on request to the corresponding author.

AUTHOR CONTRIBUTIONS

GS performed the field evaluation with the data interpretation. HL and NT followed the analyses interpretation and the conceptual evaluation of the fragility curves, also studied by GS and corrected by CC. Finally, CC contributed also to the results interpretation.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to all the persons who were involved in this work, especially to the persons who helped and made possible to obtain the survey data.

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Conflict of Interest: The authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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