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DEVELOPMENT OF FRAGILITY CURVES FOR CONFINED MASONRY BUILDINGS OF LIMA

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Abstract

The informal building's construction is associated to Lima's population growth, especially at the sub-urban areas. In Peru, confined masonry is the most common structural system used for buildings. It is fair to state that these buildings may have a poor structural performance during an earthquake of medium or high intensity. Therefore, it is important to know their structural behavior.

Fragility curves must be calculated in order to know the probability of exceeding a certain limit state due to a given ground motion and are obtained through the comparison between the structural capacity and the seismic demand. In this study the structural capacity of each masonry building is represented by a simplified capacity curve. For that, the Simplified Pushover-based Earthquake Loss Assessment (SP-BELA) methodology has been adapted to assess capacity curves for confined masonry buildings. Some field data gathered by previous studies at the PUCP was used to generate a database which contains statistical information about structural characteristics of the buildings. Then, a random population of buildings is generated using Montecarlo simulation and capacity curves are computed.

The seismic demand is represented by a set of ground motion records. Subduction records are selected from the database of the Pacific Earthquake Engineering Research Center in order to satisfy the seismological characteristics of Lima. The set of records is applied to each SDOF in order to calculate the displacement demand. From the comparison between the displacement demand and the displacement capacity, the probability damage matrix is calculated, and then the cumulative matrix. This last expresses the probability of exceedance of a given damage limit state for a given intensity level (fragility curves), which could help to compute vulnerability curves.

The results show the necessity to improve the seismic capacity of confined masonry buildings in Lima, this considering the 10% probability of exceedance in 50 years (a return period of 475 years), between 12% and 57% of collapses is expected in cases of 1-story and 2-story buildings, respectively.

Keywords: Fragility, Confined Masonry, Lima's buildings.

1. Introduction

Confined masonry (CM) has been the traditional structural system in the Peruvian coast for massive construction of buildings. The informal construction of buildings is associated to Lima's population growth, especially at the sub-urban areas. Most of these buildings are built without following adequate construction practices due to the low risk awareness of the population. It is fair to state that these buildings may have a poor structural performance during an earthquake of medium to high intensity. Around 84% of the total buildings in the metropolitan area of Lima are built with brick or cement blocks [1].

After Pisco's earthquake (2007) close to 76 000 buildings had to be rebuilt. Although, most of the collapsed constructions in Pisco were earthen structures, an important number of confined masonry buildings also collapsed. Constructions practices from Pisco are very similar to Lima's practices: 2 story buildings, long walls in one direction, lack of walls in the direction parallel to the street, use of non-adequate bricks, no technical supervision, etc. A typical confined masonry building is shown in Fig.1.

Fig. 1 – Typical CM building

Seismic vulnerability is a measure of how prone a building is to damage for a given severity of the ground shaking. Physical seismic vulnerability can be expressed through probability damage matrices and vulnerability curves. The first one describes the vulnerability as a discontinuous relationship, and the second as a continuous relationship.

The objective of this article is to derive fragility curves for confined masonry (CM) buildings of the metropolitan area of Lima, in order to assess their vulnerability in further studies. Fragility curves express the probability of exceeding a limit state for a given intensity motion level. In this study, a random population of CM buildings has been generated through Montecarlo simulation, starting from 120 buildings registered in previous studies carried out at the PUCP. Then, the SP-BELA method [2] has been adapted to calculate the simplified-pushover capacity curve for the SDOF system of each confined masonry buildings. To compare the capacity with the demand, the extending of the DBELA method proposed by Silva et al. [3] has been implemented. The demand considered for this study consists of a set of records of subduction motions records provided by the Pacific Earthquake Engineering Center Research (PEER).

2. Typology construction of confined masonry buildings in Lima

In order to characterize the buildings, a database was created based on existent surveys related to inspections made by previous researches [4]. The inspections were carried out in Lima's districts with high concentration of CM buildings. The recollected data has information about the important characteristics like number of stories, age, wall's dimensions, confinement dimensions, inter-story system, blueprints of wall distribution (Fig.2), among others. For this study, 120 surveys from buildings were processed. Typologies selected for this study are related to 1-story buildings and 2-story buildings.

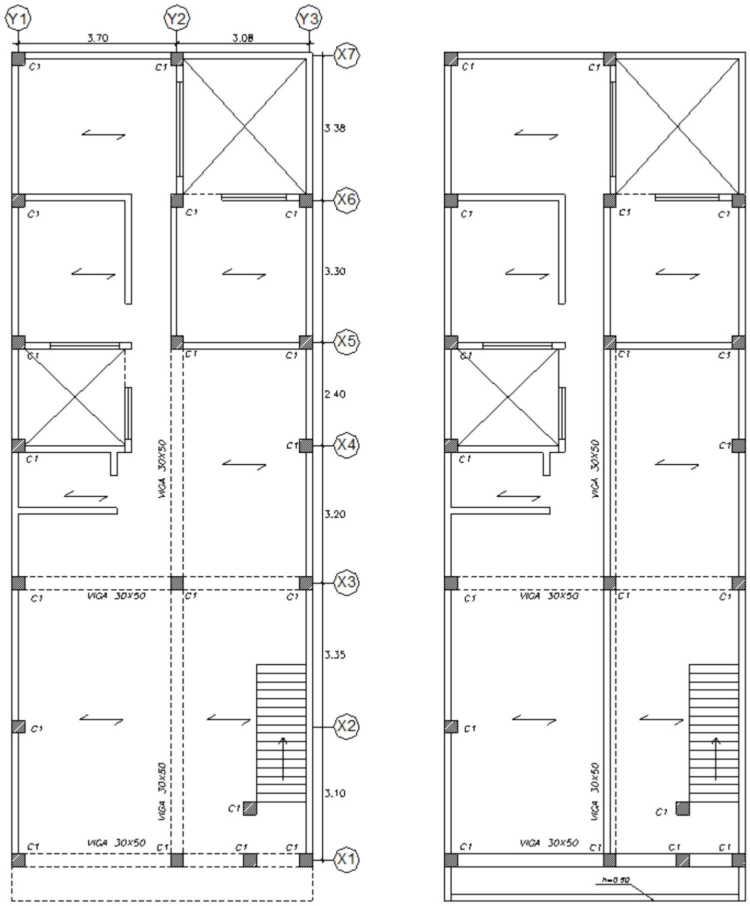

Fig. 2 - Plan view of one CM buildings. 1st floor (left) 2nd floor (right)

The majority of CM buildings' walls are built with non-solid partially industrialized bricks. These bricks are characterized to have openings of around 40 to 50% of the total area. Wall's width is 150 or 250mm. Inter-story system consists in a lightened slab of 200mm thick except in the last floors, in this case the roof consists on zinc sheets, wood, etc. From previous studies it is well known that these kinds of buildings have a large wall density in the perpendicular direction to the street, while in the parallel direction they have a poor wall density. Most of these buildings present horizontal and vertical irregularities.

Some variables have been defined in order to characterize the CM buildings. The mean values, standard deviations, and distribution laws of these variables are presented in Table 1 and 2. All of them were computed from the database described before. The purpose is to generate a random population of capacity curves of CM buildings based on the simplified procedure [2]. Montecarlo simulation has been used to generate all these random variables. It is important to state that the variables' values can be calculated depending on the number of floors. The x-x direction is defined as parallel to the street, and the y-y direction is defined as perpendicular to the street. During the review, masonry walls with an important percentage of openings were found, as were for doors and windows. Due to the strength reduction that it signifies, these walls are considered "Partially Confined" walls.

3. Simplified capacity curve for confined masonry

The main component of the SP-BELA method is the definition of the capacity curve for a given structural system using a simplified methodology. The capacity curve is defined in terms of the collapse multiplier and the lateral displacement, and both are defined for a given collapse mechanism.

3.1 Collapse mechanism

Borzi et al. [2] have originally developed SP-BELA method for reinforced concrete buildings. Then, Borzi et al. [5] presented the SP-BELA method for unreinforced masonry buildings (URMB) based on the collapse multiplier proposed by Benedetti and Petrini [6]. In this article, the SP-BELA for URMB has been adapted to be used for confined masonry buildings.

Masonry buildings have been largely studied by many authors. They agree that the concentrated damage is the most common collapse mechanism for masonry buildings as is shown in Fig.3. It means that the inelastic behavior occurs in one sole story. Some experimental studies show that this mechanism is a good representation of the behavior of CM buildings. In some cases the out of plane failure of walls is considered, but in the case of CM this is not necessary since walls are confined by the concrete piers.

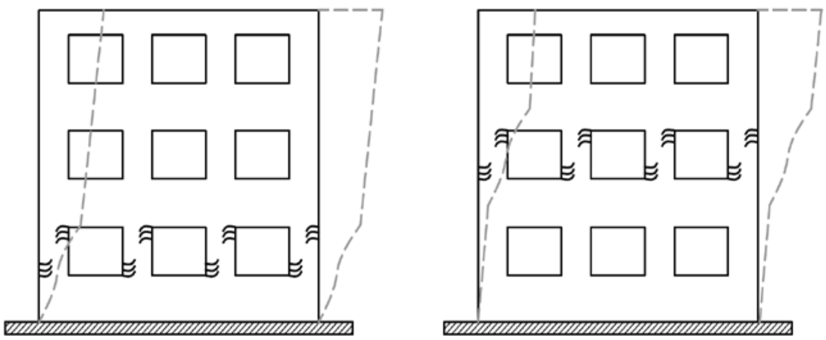

Fig. 3 - Collapse mechanism: Concentrated damage first floor (left), second floor (right) (adapted from [7])

3.2 Lateral load resistance

Lateral resistance of CM buildings is given mainly by the density and the resistance of the walls. The lateral force can be withstand by a combination of flexure, shear and rocking mechanisms [7]. The collapse multiplier will be used to quantify the lateral resistance of the buildings. The collapse multiplier is the relationship between the lateral resistance force and the weight of the building. In the case of CM buildings, it can be assumed that the lateral resistant force is given by the superposition of the lateral resistant force of their individual walls. For a given displacement, squat walls will have bigger shear force contribution than slender walls; therefore, for this case study, walls with length less than 1m were not considered.

Tomaževič et al. [8], Alcocer et al. [9], Riahi et al. [10] have studied lateral resistance of confined masonry walls. They agree in defining the capacity curve through 3 control points as in Fig.4. Number 1 corresponds to the cracking; number 2 to the maximum shear force; and number 3, to the ultimate lateral force.

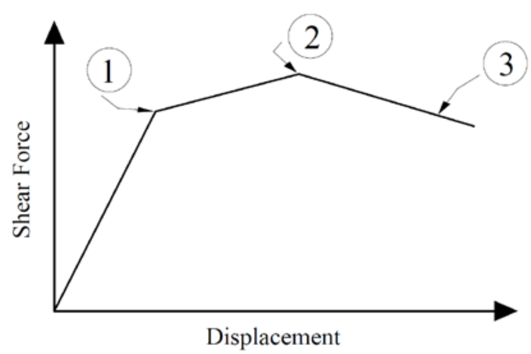

Fig. 4 - Control points to define lateral resistance force of CM walls

According to the analytical model proposed by Alcocer et al. [9], Eq. (1), Eq. (2) and Eq. (3) are proposed to calculate the collapse multiplier for one floor and one direction. These formulas consider that the shear stress and the vertical loads are uniformly distributed in all walls, that there is a rigid diaphragm, and that there is not torsional effect.

$$\lambda_{i-d-1} = \frac{1}{W_T \frac{\sum_{k=i}^{n} h_k W_k}{\sum_{j=1}^{n} h_j W_j}} A_{i-d} \left[0.5\tau + 0.3 * \frac{\sum_{k=1}^{n} W_k}{A_i * (1+\gamma_{AB})}\right] \tag{1}$$

$$\lambda_{i-d-2} = \lambda_{i-d-1} + \frac{1}{W_T \frac{\sum_{k=i}^{n} h_k W_k}{\sum_{j=1}^{n} h_j W_j}} * \beta * \left[1.26 * d_b^2 * \sqrt{f_c' * f_y}\right] * n_{i-d} \tag{2}$$

$$\lambda_{i-d-3} = 0.8 * \lambda_{i-d-2} \tag{3}$$

where $\lambda_{i-d-n}$is the collapse multiplier in the n-control point (1, 2 or 3) for the i-story in the d-direction, W_T is the total weight of the building, W_i is the weight of the i-story calculated as the weight per unit area and the total area of the building, h_i is the height of a i-story, A_{i-d} is the total shear wall area in the d-direction in the i-story, τ is the diagonal compression resistance of the masonry, γ_{AB} is the ratio between A_{i-d} and B_d with B_d being the maximum area between the area of shear walls in the loaded direction and in the orthogonal direction, β is the efficiency factor related to the confinement columns, n_{i-d} is the total number of columns in the loaded direction and in the i-story, d_b is the diameter of the reinforcement bars, f_c' is the compression resistance of the concrete, f_y is the yielding stress of the reinforcement steel. Random values used in the previous equations are shown in Table 1 and Table 2.

Table 1 – Random variables for 1-story buildings

Variable	**Description**	**Units**	**Mean**	**Standard deviation**	**Distribution**
A_{i-d}	Shear area confined wall x-x	m^2	1.35	0.89	Lognormal
	Shear area confined wall y-y	m^2	4.57	2.32	Lognormal
	Shear area partially confined wall x-x	m^2	1.16	0.86	Lognormal
	Shear area partially confined wall y-y	m^2	0.97	0.96	Lognormal
h_i	Height (i-interstory)	m	2.58	0.12	Normal
n_{i-d}	Number of confinement columns x-x	NA	5	2	Gamma
	Number of confinement columns y-y	NA	12	4	Gamma
τ_i	Diagonal compression masonry walls (1 and 2-stories)	MPa	0.47	0.05	Normal
A_T	Area (1 and 2-stories)	m^2	103.55	43.24	Lognormal
V_L	Vertical load (1 and 2-stories)	kN/m^2	7.1	0.7	Normal

Table 2 - Random Variables for 2-story Buildings

Story	Variable	Description	Units	Mean	Standard deviation	Distribution
1	A_{i-d}	Shear area confined wall x-x	m2	1.71	0.85	Lognormal
		Shear area confined wall y-y	m2	5.77	2.85	Lognormal
		Shear area partially confined wall x-x	m2	1.55	1.11	Lognormal
		Shear area partially confined wall y-y	m2	0.89	0.3	Lognormal
	n_{i-d}	Number of confinement columns x-x	NA	5	2	Gamma
		Number of confinement columns y-y	NA	12	4	Gamma
	h_1	1st story Height	m	2.58	0.12	Normal
2	A_{i-d}	Shear area confined wall x-x	m2	1.77	1.14	Lognormal
		Shear area confined wall y-y	m2	4.70	2.30	Lognormal
		Shear area partially confined wall x-x	m2	1.37	0.92	Lognormal
		Shear area partially confined wall y-y	m2	0.71	0.35	Lognormal
	n_{i-d}	Number of confinement columns x-x	NA	5	2	Gamma
		Number of confinement columns y-y	NA	12	4	Gamma
	h_2	2nd story Height	m	2.56	0.12	Normal

For the calculation of the shear wall area, the partially confined walls were considered affected by 0.45 times the total area. This reduction factor was proposed after the comparison of the shear capacity of walls with large openings and completely confined walls presented in Yañez et al. [11], who studied the behavior of confined masonry shear walls with large openings. The lower collapse multiplier of each direction and of all stories determines one that is used for the complete structure.

3.3 Displacement capacity

The SP-BELA method proposes the definition of a single-degree-of-freedom (SDOF) system, which is equivalent to the multi-degree-of-freedom (MDOF) system in terms of mass, stiffness and displacement capacity. The SDOF is defined in terms of the collapse multiplier and the displacement.

Starting from the collapse mechanism for CM buildings proposed in 3.1, a deformed shape will be assumed for each control point in order to define the equivalent SDOF system of the building as shown in Fig.5:

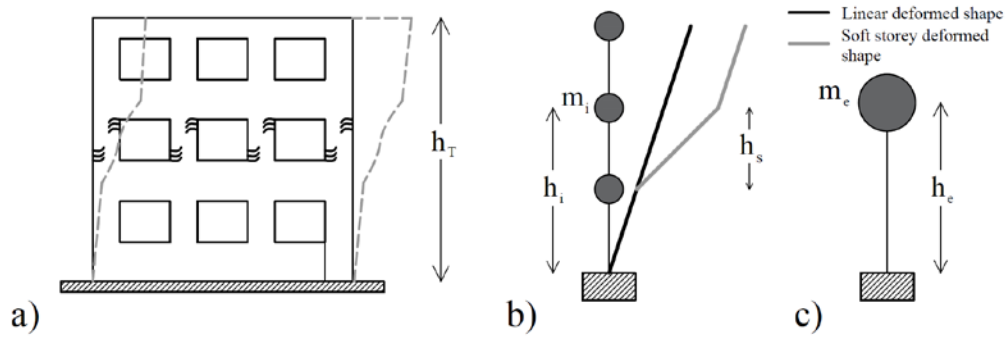

Fig. 5 – a) Concentrated damage mechanism collapse, b) deformed shape assumed for the MDOF, c) equivalent SDOF (Adapted from [2])

A deformed linear shape can describe correctly the elastic behavior of the structure. The control point 1 of the building represents the elastic behavior, then, the displacement capacity of the SDOF for the assumed shape is given by the following equation:

$$\Delta_y = k_1 h_T \delta_y \tag{4}$$

where h_T is the total height of the MDOF system, k_1 is the relationship between the total height of the MDOF and the height of the SDOF system in the elastic range, and δ_y is the maximum drift for the elastic behavior.

To calculate the displacement capacity in the non-elastic control points, it is necessary to add the concentrated non-elastic displacement of the weakest story as detailed in Eq. (5):

$$\Delta_i = k_1 h_T \delta_y + k_2 (\delta_{CPn} - \delta_y) h_s \tag{5}$$

where k_2 is the relationship between the total height of the MDOF and the height of the SDOF system in the inelastic behavior; h_s is the height of the weakest story (defined as the story that has the lowest collapse multiplier), δ_{CPn} is the drift for the n-control point.

The drifts for each limit state were adapted from Astroza and Schmidt [12], according to the definition of the limit states proposed herein. These values are shown in the Table 3. The first control point defines the elastic limit state, the second defines the maximum resistance strength of the CM wall, and the third defines the last resistance strength of the CM wall before collapse.

Table 3 – Definition of drifts (percentage) for each control point.

Control Point	**Drift %**
1	0.14
2	0.62
3	0.86

Restrepo-Velez [13] proposed the values for k_1 and k_2 that are shown in Table 4 according to the number of stories.

Table 4 – Values for $\boldsymbol{k_1}$ and $\boldsymbol{k_2}$ factors according to the number of floors

Number of Floors	$\boldsymbol{k_1}$	$\boldsymbol{k_2}$
1	0.790	0.967
2	0.718	0.950

The damping value used for the dynamic analysis has been taken from Priestley et al. [14].

$$\zeta = 0.05 + c/\pi \cdot (\mu - 1)/\mu \tag{6}$$

where ζ is the equivalent viscous damping coefficient of the SDOF system and c is a coefficient proposed by Naveed et al. [15], which was obtained through nonlinear regression of experimental data results of masonry piers, and μ is the ductility for a given limit state as the ratio between the displacement at one given limit state, and the displacement at the yielding point.

3.4 Definition of the limit states

The first two limit states (LS) for CM buildings have been defined according to the proposal of Lagomarsino and Giovinazzi [16], who proposes the slight and moderate damage at 0.7 and 1.5 times the yielding displacement respectively. The extensive damage state and collapse have been defined according to the drifts reached in the cyclic test of CM walls for the maximum strength and the collapse displacement; these values are shown in Table 5.

Table 5 – Damage States associated to Damage Threshold

Damage State	**Damage Threshold**
Slight	$0.7\Delta_1$
Moderate	$1.5\ \Delta_1$
Extensive	Δ_2
Collapse	Δ_3

where Δ_1 is the pseudo-displacement at the yielding point, Δ_2 is the displacement for the extensive damage state, that is associated to the maximum strength. Δ_3 is related to collapse. These limit states can be associated to the control points defined in 3.2 and 3.3, as shown in Fig.6.

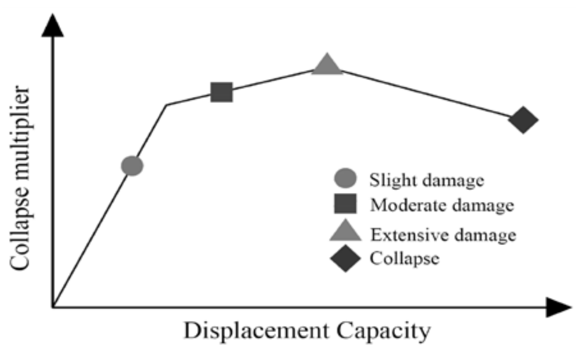

Fig. 6 - Damage States associated to Damage Threshold in the capacity curve

Using Montecarlo simulation, a sample of 100 capacity curves were generated corresponding to buildings of 1-story and 2-stories (Fig.7) according to the formulas proposed in 3.2 and the properties described in Table 1 and Table 2. For the derivation of the capacity curves, 1000 capacity curves were generated for each building typology.

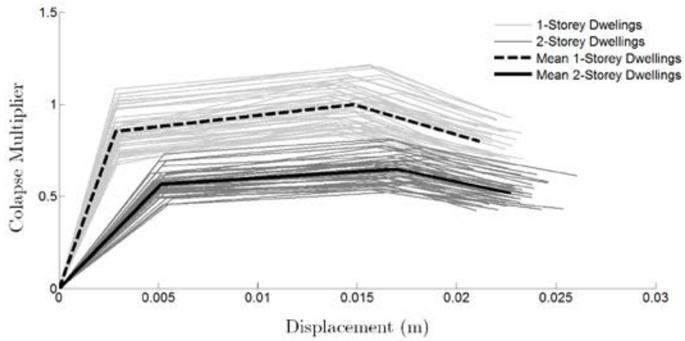

Fig. 7 - Random Population of CM buildings

4. Derivation of fragility curves for confined masonry buildings in Lima

Once the synthetic random population of buildings was generated, the demand should be defined and then compared. In this step, the modified DBELA method proposed in Silva et al. [3] has been used. One important advantage is to consider the effect of the record-to-record variability of the seismic input to get more realistic results.

The selection of the ground motion records is a key parameter to establish the demand. This database should comprise a variety of records in order to consider local seismic hazard properties as magnitude and peak ground acceleration range, common fault failure mechanism, frequency content, duration and epicentral distance.

For a given SDOF and a given record, the displacement demand is calculated as the maximum displacement obtained by the dynamic analysis. A Probability Damage Matrix (PDM) should be assembled based on the number of buildings that achieve a given limit state. These operations will be repeated for each record. From the PDM is possible to get the Cumulative Probability Damage Matrix (CPDM) in terms of percentage, and associating each record to one intensity measure level it is possible to fit a fragility curve set from a point cloud.

A second process is to find the best period of pseudo-acceleration to represent the intensity motion level in the fragility curves. For this purpose, the correlation factor between the cumulative percentages of buildings that exceeds a given limit state in the PDM, and so, obtaining the displacement demand of each ground motion record in a given period. This process is carried out for a set of periods, and for each limit state. Then, the best period is obtained by looking for the closest correlation factor to one (Fig.8). Once the scale is well defined, the demand should be expressed in terms of pseudo-acceleration at the period with the best correlation coefficient. The best correlation coefficient is 0.90 for a period of 0.14 s for 1-story buildings and 0.92 for a period of 0.22 s for 2-story buildings. The next step is to create a point cloud, which expresses the demand in pseudo-acceleration, and the cumulative probability of exceeding a damage level. Furthermore, the points for each damage level are fitted to a log-normal cumulative probability function, which has been establish in many studies as a good approach to represent fragility functions (Fig.9).

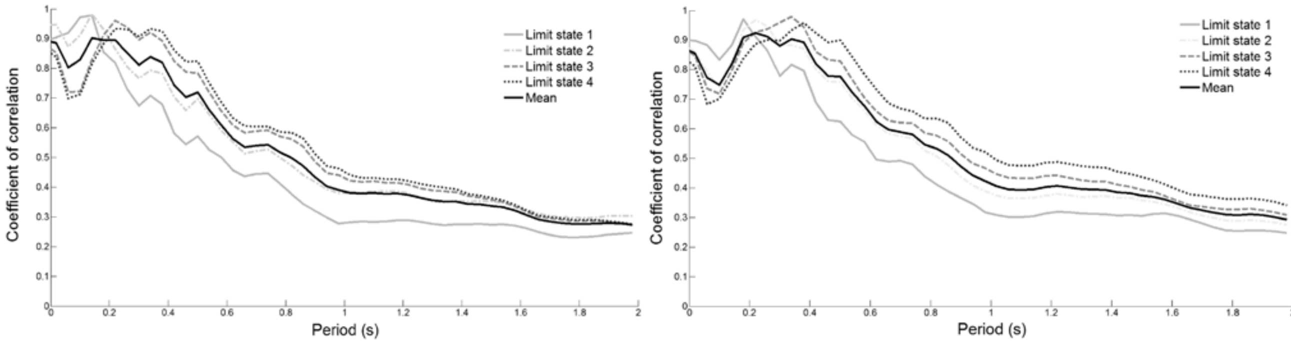

Fig. 8 - Correlation factors for each limit state along the periods 1-story buildings (left), 2-story buildings (right)

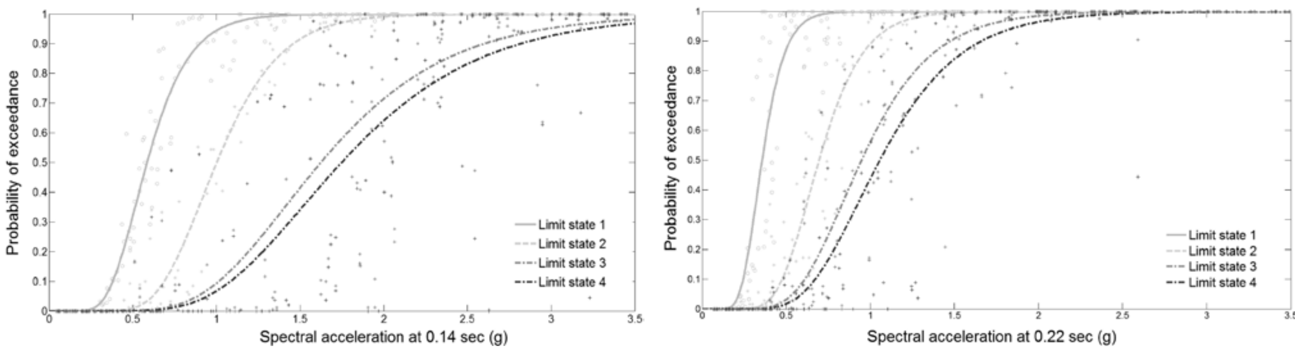

Fig. 9 – Fragility curves for CM buildings

In order to compare the results obtained for 1-story and 2-story buildings, the curves that represent the collapse (Limit state 4) for both types of buildings were plotted in terms of PGA in Fig.10. The correlation factor for the limit state 4 in 1-story building when plotting in PGA was 0.85, and 0.82 for the 2-story buildings. Collapses expected for 1-story buildings are less than the ones expected for 2-story buildings.

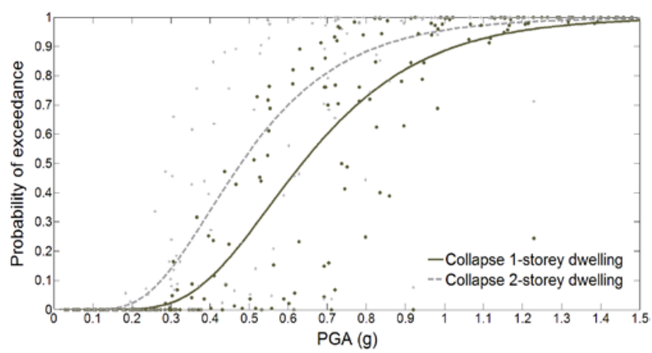

Fig. 10 – Limit state 4 (collapse) for 1, and 2-story buildings

5. Conclusions

A mechanics based procedure has been proposed for the computation of simplified pushover curves for CM buildings based on the work presented by Borzi et al. [2,4]. In addition, the probabilistic framework herein, and then extended by Silva et al. [3], has been used to calculate fragility curves for CM buildings.

The procedure relies on a probabilistic framework, in this way it is possible to consider material and geometric uncertainties. At the same time, it is possible to assess fragility curves for other building population by the inclusion of their characteristics. This method includes the record-to-record variability of the input according to the seismic characteristics of the studied area. Hence, this method provides a quick, accurate and realistic procedure for the assessment of fragility curves.

According to the Seismic Resistant Peruvian Code, a pseudo acceleration of 1.125g (0.45g times 2.5 amplification factor) is expected in coastal areas such as Lima. 1 and 2-story CM buildings may expect to suffer that acceleration since their period of vibration is less than the corner period (T_c) of the Peruvian spectral acceleration. Therefore, according to Fig.9, around 12% and 57% of collapses is expected for 1-story and 2-story buildings, respectively.

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7. Copyrights

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