17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 27th to October 2nd, 2021

FRAGILITY CURVES FOR CONFINED MASONRY IN TYPICAL CONSTRUCTION IN PERU..

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Abstract
The purpose of this work was elaborate the fragility curves for confined masonry buildings and estimate probabilities of leave associated with different levels of performance for typical construction in Peru. The structural typology proposed consist of a 5-level building and modeled by OpenSees computer program. The fundamental period of the structure have been calculate according to empirical models, however, the seismic weight of the structure, the stiffness and the seismic shear force are estimated. After, once the structure defined, the incremental dynamic analysis is carried out by scaling the accelerograms to measure the behavior of the structure against seismic loads. A total of 23 pairs of seismic records were used, each with 100 scaling values, making a total of 4600 nonlinear history time analysis with pseudo-accelerations from 0.02g to 4.00g. With the results, the probabilities of exceedance for 3 levels of performance, as, Immediate Occupancy (OI), Life Protection (PV) and Collapse Safety (SC) were calculated. The results showed that for pseudo accelerations of 0.50g the probability of exceeding the level of performance of (OI) was 100%, for the (PV) was 30% and 2% for the (SC). The results obtained are very interesting because, though, the research has focused for a typical construction in Peru; the study to can be extended to different buildings.

Keywords: fragility curves, probability of collapse, incremental dynamic analysis, confined masonry.

1. Introduction

The analysis and design of earthquake resistant structures in seismic countries such as Peru are constantly in progress. Each time new studies, publications and research papers provide valuable contributions for their practical application. Even so, the seismic behavior of structures is a very complex issue that often leads engineers to make simplifications based on mathematical models that somehow represent reality. Those of masonry walls confined in buildings is very common in Peru, given the high rigidity they give to the structural system allowing the building to comply with the requirements of earthquake resistant design established in the standards. The seismic activity in Peru has its origin in the process of interaction between the Nazca plate and the South American plate, between these two rocky massifs there is a subduction effect. This process is responsible for the vast majority of seismic events in Peru. The last major earthquake originating in the plate convergence process, occurred on August 15, 2007 with a magnitude of 7.0ML (Richter scale) and 7.9Mw (Moment scale), called "the Pisco earthquake" due to which its epicenter was located 60 km west of this city [1].

The use of appropriate fragility curves for structures is an essential and basic tool for estimating earthquake loss [2]. The fragility curves show the probability that a limit state is exceeded, always associated with the structural response of the building. To determine these curves it is necessary to define performance levels using for example the mezzanine displacement parameter (Drif) as a measure of damage. Thus, Carrillo and Alcocer (2012) propose acceptance limits for the structural response of low-rise walls. Its limits are based on performance indicators, crack widths and damage rates [3]. In general, to define a fragility curve fits a lognormal cumulative probability distribution function [4], where the probability of exceeding a damage level is described based on the seismic intensity characterized. On the other hand, an incremental dynamic analysis (IDA) is a method of parametric analysis that allows you to estimate the performance of a structure. This methodology consists in scaling an accelerogram to measure the behavior of the structure against seismic loads. [5].

For this study we used a series of seismic records from Peru applied to a structural model of a 5-level building as shown in the Fig. **1**. The choice of seismic records will be based on their magnitude and depth. The results of experimental trials of confined masonry walls conducted by Pari and Manchego (2017) [6] were used to determine the constitutive constitutive floor relationship. In this work an experimental campaign was developed that consisted of building nine walls on a natural scale (of a level) that were subsequently tested under cyclic lateral load in their plane. The cyclic tests were carried out in the structures laboratory of the Pontifical Catholic University of Peru (PUCP), following the guidelines of FEMA 461. Three (3) walls with vertical load equivalent to 3 levels and six (6) walls without load were tested vertical. Of the latter, three (3) walls were tested to a limit of reparability in order to repair and retest them. As a preliminary part of the cyclic tests, control tests were carried out in clay units and in the mortar. itself, it was made of masonry prisms that allowed to characterize the mechanical properties of the masonry [6].

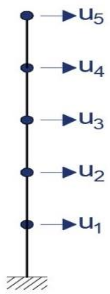

Fig. 1 – Modelo Estructural simplificado estudiado

Some recent works in Latin America where fragility curves have been proposed are, for example, the work of Haindl (2014) [7], where the seismic performance of a two-story reinforced concrete wall house, representative of the construction in Chile, through fragility curves. On the other hand Velásquez (2006) develops a methodology that allows to reasonably predict earthquake losses in Peruvian buildings. The methodology has a probabilistic approach and is based on fragility curves [8]. The importance of this type of studies is based on the need to estimate the damage to structures and safeguard the lives of people. Also this study can be extended to any type of structural system since they are based on fundamental concepts..

2. Nonlinear Model in Opensees

The open system for seismic engineering simulation OPENSEES (Open System for Earthquake Engineering Simulation) is a computer platform for the development of simulation applications of the behavior of structural and geotechnical systems, subject to seismic events. OpenSees uses finite element-based methods, therefore the first step for modeling is to subdivide the system into elements and nodes, in order to define the action of loads, and nodal restrictions [9]. In this work the stiffness of the walls is modeled using uniaxial spring-type elements. Uniaxial elements are modeled using elements of unit length and with the hysteretic material (Hysteretic Material), existing in OpenSees.

3. Structural Model

3.1 Structure description

The structure considered consists of a 5-level building based on confined masonry walls located in zone 4 of the seismic map of Peru. The type of use of the building is of the housing type. The dimensions of the structure are 6 meters wide and 20 meters long with an area of 120 square meters and a typical floor height of 2.5 meters. Usually, the last floor of the building is the one with the lowest mass but for this work it has been considered that all levels of the structure have the same seismic mass in order to simplify the calculations. Fig. 2 schematically shows the structural system that is evaluated.

Fig. 2 – Typical 5-level building of confined masonry

3.2 Structural Model

The structure model is composed of spring-type elements. The mass is concentrated in the nodes and the base is considered to be embedded in the ground. Each spring type element has a constitutive relationship that represents the behavior of a confined masonry building. The 5 degrees of freedom considered are in the horizontal direction "x" as shown in the Fig. 1.

3.3 Seismic Weight

The seismic mass considered is 1000 Kg/m^2 which implicitly includes its own weight, dead load and usage overload. The seismic weight of the structure is shown in the Table 1.

Table 1 – Structure weight

Level	Weight (kN)	Accumulated weight (kN)
5	1177.20	1177.20
4	1177.20	2354.40
3	1177.20	3531.60
2	1177.20	4708.80
1	1177.20	5886.00

3.4 Structure Period

The fundamental period of the structure was determined from the empirical expression established by the Peruvian standard for earthquake resistant design E0.30 [10]. The regulations establish that the fundamental period of vibration for each direction is estimated with the following expression: $T = h_n/C_T$ [10]. Where h_n is the total height of the building $h_n = 12.50\ m$ y C_T is a coefficient equal to 60 for masonry buildings. Consequently, the period (T) of the 5-level building under study is 0.208 seconds.

3.5 Rigidity of the Structure

The dynamic properties of the structure such as periods, natural modes of vibration are relevant parameters in the seismic analysis of linear and nonlinear systems. These parameters depend on their stiffness and mass properties, and are obtained by modal analysis. Modal analysis can be performed in two ways: i) using eigenvectors and vectors and ii) using Ritz vectors. In this investigation, the modes are determined using the problem of own values and vectors. The mass matrix is a diagonal matrix with a story mass of 120 ton. The stiffness matrix of the structure is determined by the stiffness of each story (k_i) according to Eq. 1.

$$K = \begin{bmatrix} k_1 + k_2 & -k_2 & 0 & 0 & 0 \\ -k_2 & k_2 + k_3 & -k_3 & 0 & 0 \\ 0 & -k_3 & k_3 + k_4 & -k_4 & 0 \\ 0 & 0 & -k_4 & k_4 + k_5 & -k_5 \\ 0 & 0 & 0 & -k_5 & k_5 \end{bmatrix} \tag{1}$$

Since the period of the structure is 0.208 seconds and assuming a uniform stiffness in all floors, the stiffness of the floor necessary to meet this requirement can be determined by solving the problem of own values given by the Eq. 2.

$$[K - \omega_n{}^2 M]\emptyset_n = 0 \tag{2}$$

Where ω_n represents the modal frequency, K is the stiffness matrix, M is the mass matrix in units of Kilograms (Kg) and $\emptyset_n$ represents the modes of vibration of the structure. Solving the problem of own values, it is determined that the stiffness of the floor to have a period of 0.208 seconds is 1351 kN/mm.

3.6 Basal shear force

The location of the building was considered to be in the department of Ica, Peru, where the most destructive seismic event of the last 20 years occurred with a magnitude of 7.9 (M_w) on the moment scale (Pisco earthquake). The building being located in the city of Ica belongs to seismic zone 4 and corresponds to a value of $Z = 0.45$. The structure for being destined for housing and being contemplated as a building of common use, corresponds to a value of $U = 1$. It is considered a soil type S3 or soft soil, characterized by the presence of medium or fine sand or sandy gravel (Soil amplification factor (S): 1.10, Soil period $T_P = 1.0\ s$ y $T_L = 1.6\ s$). The structure is based on confined masonry walls, therefore it corresponds to a value of $R = 3$ as a reduction factor without considering irregularities. The seismic amplification factor is $C = 2.5$. The design seismic shear stress is obtained according to E.030 and is given by Eq. 3.

$$V = \frac{Z * U * C * S}{R} * P \tag{3}$$

Where P is the seismic weight of the structure.

The shear forcé is $V = 2318\ kN$. On the other hand, in Peru the load combinations are specified by the standard of reinforced concrete E.060. The ultimate shear forcé (Vu) is obtained with Eq. 4.

$$Vu = 1.4 * Ve \tag{4}$$

Vu is the ultimate force or design force and Ve for this case is the shear force obtained according to E.030. Consequently, the ultimate shear forcé is equivalent to $Vu = 3245\ kN$. To obtain the nominal force Vn, the Peruvian standard E.060, establishes Eq. 5.

$$\phi Vn \geq Vu \tag{5}$$

Where ϕ s the reduction factor equal to 0.85 obtained from the E.060 standard and where it is obtained that the value of the nominal force is $Vn = 13817\ kN$.

3.7 Effective Rigidity

The Standard establishes that the design of the walls covers its entire range of behavior, from the elastic stage to its probable incursion into the inelastic range, providing sufficient ductility and control of the degradation of resistance and stiffness [11]. To consider probable incursions of the structure in the inelastic range, we worked with an effective stiffness at 70% of its initial properties. Considering the stiffness reduction factor of 0.70 and the story stiffness of 1351 kN/m, the effective stiffness is 945.70 KN/mm. The periods of the structure calculated with the effective stiffness are shown in Table 2. The effective masses are also included ($M_n^* = {L_{nx}}^2/M_n$) and modal participation factor ($\Gamma_n = L_{nx}/M_n$).

Table 2 – Structure periods using effective stiffness

Modes	**Periods**	$\boldsymbol{\Gamma_n}$	$\boldsymbol{M_n^*(Kg)}$	$\boldsymbol{M_n^*}$**(%)**
1	0.249	731.71	526829.27	88.58
2	0.085	-219.51	47414.63	7.97
3	0.054	121.95	14634.15	2.46
4	0.042	73.17	5268.29	0.89
5	0.037	-24.39	585.37	0.10

3.8 Damping of the structure

For the damping of the structure, the Rayleigh model was considered. According to this model, the damping matrix is obtained as a linear superposition of the mass and stiffness matrices of the structure, according to Eq. 6.

$$C = a_0 * M + a_1 * K \tag{6}$$

C is the damping matrix, a_0 and a_1 are calculated considering the periods of the structure. It was considered at a damping ratio of 2% para los dos primeros modos. for the first two modes. Consequently, the values of $0.75248\ \ 1/seg.$ and $0.00040\ seg.$ respectively. he damping obtained for each mode is shown in

Table 3.

Table 3 – Modal structure damping

Modes	Periods $(seg.)$	Frequencies $(rad./seg.)$	Damping $(\xi\%)$
1	0.249	25.23	2.00
2	0.085	73.92	2.00
3	0.054	116.36	2.67
4	0.042	149.60	3.27
5	0.037	169.82	3.65

3.9 Hysteresis of the material

To define the hysteresis curve of each floor (trilinear curve) it is necessary to define three limit states that define the three notable points of the curve. These three limit states are defined as the cracking force Vcr, the maximum force Vn y and the rupture force Vr. The associated displacements are the cracking displacement δcr, the máximum displacement δn and the rupture displacement δr respectively. For the value of the maximum shear force Vn, the nominal resistance $Vn = 3817\ kN$ is considered. The values of the cracking limit state $(Vcr, \delta cr)$, rupture $(Vr, \delta r)$ and the maximum displacement δn, were determined based on the results of the experimental tests carried out by Pari and Manchego (2017) [6]. His work was aimed at identifying damage states associated with the structural performance of confined masonry walls so that they can be used for the construction of fragility functions [6]. Nine (9) walls were built on a natural scale (2.60 x 0.13 x 2.40 m). The walls were tested under cyclic lateral loads in their plane with controlled lateral displacement following the guidelines of FEMA 461 [6]. From the results of the study, the curve of Fig. 3**¡Error! No se encuentra el origen de la referencia.** was obtained, which shows the simplified envelope curve. Table 4 shows the values of each notable point of the curve of **¡Error! No se encuentra el origen de la referencia.**.

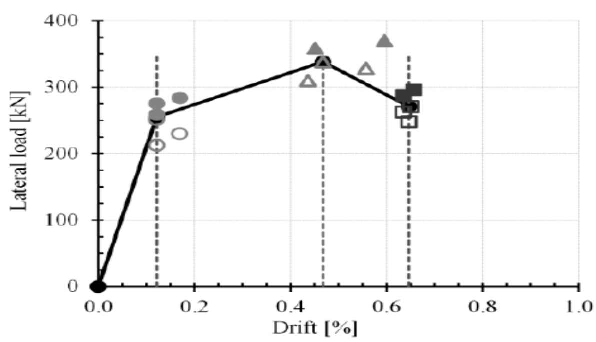

Fig. 3 – Idealized Envelope Curve - Pari y Manchego (2017) [6]

Table 4 – Force and displacement of envelope curve - Pari y Manchego (2017) [6]

Wall	V_{cr} (kN)	V_{max} (kN)	V_u (kN)	δ_{cr} (%)	δ_{max} (%)	δ_u (%)
MQ	255.30	338.40	270.70	0.12	0.47	0.65

From Table 4 it can be determined that the cracking force represents 75% $(F_1 = 0.75)$ of the maximum force. On the other hand, the rupture force represents 80% $(F_2 = 0.80)$ of the maximum strength of the experimental tests. From the results of the test described above, the parameters were determined to simulate the hysterical behavior of each floor of the building in this work. Thus, the cracking force (V_{cr}) is estimated as the product of the maximum nominal force (V_n) and the factor F_1, obtaining a value of $V_{cr} = 2880.00\ kN$. The value of the rupture force V_r, is estimated as the product of the maximum nominal force and the factor F_2, obtaining a value of $V_r = 3054\ kN$. The cracking displacement (δ_{cr}) is determined as the ratio of the cracking force and the effective stiffness, resulting in a displacement of 2880.00/945.70 = 3.05

mm, equivalent to a drift of 0.122%, drift very similar to the drift 0.12% of the MQ wall in Table 4. The maximum displacement (δ_n) represents 0.47% of the story height equal to 11.75 mm and the rupture displacement (δ_r) represents 0.80% equal to 16.25 mm. Fig. 4 shows the graphical representation of the force - strain curve to be used considering the points of the cracking, maximum and breakage limit states. Table 5 shows the results that will represent the behavior of confined masonry walls..

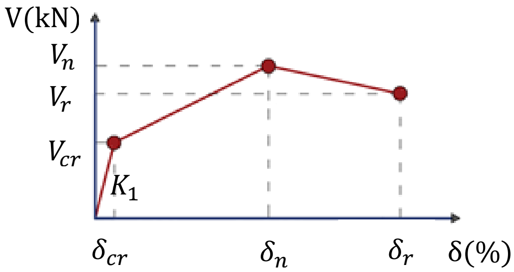

Fig. 4 – Proposed force – deformation curve.

Table 5 – Values of the force - displacement ratio for masonry walls.

Descriptión	**V_{cr} (kN)**	**V_n (kN)**	**V_r (kN)**	**δ_{cr} (%)**	**δ_n (%)**	**δ_n (%)**	**K_1 (kN/mm)**
Element: Link - Wall	2880.00	3817.00	3054.00	0.122 (3.02mm)	0.470 (11.75mm)	0.650 (16.25mm)	975.70

4. Incremental Dynamic Analysis

4.1 Escalation of Seismic Records

The scaling of the records in the incremental dynamic analysis is intended to expose the structure to various levels of semic intensity. The seismic intensity is represented by the maximum acceleration of the register or by the spectral acceleration corresponding to the fundamental period of the structure. If maximum register acceleration (PGA) is used as an escalation parameter, it is equivalent to using a period T equal to zero for the spectral acceleration applied to the structure. In this work, the seismic records are scaled by a factor "F" in such a way to achieve a pseudo acceleration of 0.2g for the period T equal to 0.249 seconds. Then, the records are progressively scaled until all time history analyzes estimate the collapse of the structure, collapse being understood as the moment when any of the floors considered in the direction of analysis present drifts equal to or greater than 0.65%. The nonlinear dynamic response of the structural model is evaluated in the horizontal direction "x" using a set of 23 pairs of horizontal seismic records selected according to their depth. The depth of the seismic events considered ranges between 0 and 30 kilometers, considered surface earthquakes. The magnitude of the momento (M_w) of the seismic records ranges between 5.0 and 8.1. The PGA range of seismic records varies between 0.54 and 268.24 cm/seg^2. La Table 6 shows the seismic records used in this study, as well as their magnitude, depth, PGA and the factor "*F*". The selection of this set of seismic records was based on using interplate type records. Seismic events were obtained from the website http://www.red-acelerografica-peru.com/. Fig. 5 shows the response spectra $(\xi = 2\%)$ of the seismic records initially scaled to a pseudo acceleration of 0.2g in the period T equal to 0.249 seconds, these are graphs that contain the points of the elastic response of a system of degree of freedom, when subjected to the action of an earthquake, registered or artificially generated [12].

Table 6 – Seismic events

Event / Magnitude / Depth (Km)			Component	PGA	Event / Magnitude / Depth (Km)			Component	PGA
Earthquake 17-10-1966	8.1 M_w	24.0	EW [1]	180.560	Earthquake 07-03-2019	5.4 M_L	18.0	EW [25]	3.136
			NS [2]	268.236				NS [26]	3.874
Earthquake 02-04-2014	7.4 M_w	20.0	EW [3]	32.589	Earthquake 24-01-2019	5.3 M_L	30.0	EW [27]	2.327
			NS [4]	17.089				NS [28]	2.990
Earthquake 31-03-2019	6.3 M_L	7.0	EW [5]	0.537	Earthquake 15-01-2019	5.3 M_L	24.0	EW [29]	65.718
			NS [6]	0.550				NS [30]	92.599
Earthquake 08-01-2019	6.3 M_L	15.0	EW [7]	0.819	Earthquake 09-04-2018	5.3 M_L	26.0	EW [31]	1.831
			NS [8]	0.972				NS [32]	1.604
Earthquake 05-06-2017	6.2 M_L	18.0	EW [9]	36.049	Earthquake 11-02-2018	5.3 M_L	20.0	EW [33]	1.163
			NS [10]	34.897				NS [34]	1.500
Earthquake 13-08-2017	6.1 M_L	15.0	EW [11]	6.987	Earthquake 05-10-2018	5.2 M_L	19.0	EW [35]	1.034
			NS [12]	12.882				NS [36]	0.962
Earthquake 18-07-2017	5.8 M_L	28.0	EW [13]	9.931	Earthquake 24-05-2018	5.1 M_L	25.0	EW [37]	2.464
			NS [14]	7.283				NS [38]	2.838
Earthquake 08-05-2019	5.6 M_L	17.0	EW [15]	17.149	Earthquake 30-05-2019	5.0 M_L	30.0	EW [39]	0.709
			NS [16]	17.565				NS [40]	0.778
Earthquake 14-02-2019	5.6 M_L	20.0	EW [17]	13.418	Earthquake 28-05-2019	5.0 M_L	24.0	EW [41]	1.270
			NS [18]	20.425				NS [42]	1.262
Earthquake 29-11-2017	5.6 M_L	25.0	EW [19]	13.068	Earthquake 03-01-2019	5.0 M_L	19.0	EW [43]	6.830
			NS [20]	14.413				NS [44]	8.430
Earthquake 15-09-2018	5.5 M_L	16.0	EW [21]	27.807	Earthquake 03-10-1974	6.2 M_L	21.2	EW [45]	191.889
			NS [22]	48.699				NS [46]	207.355
Earthquake 05-04-2018	5.5 M_L	30.0	EW [23]	2.015	-	-	-	-	-
			NS [24]	2.364				-	-

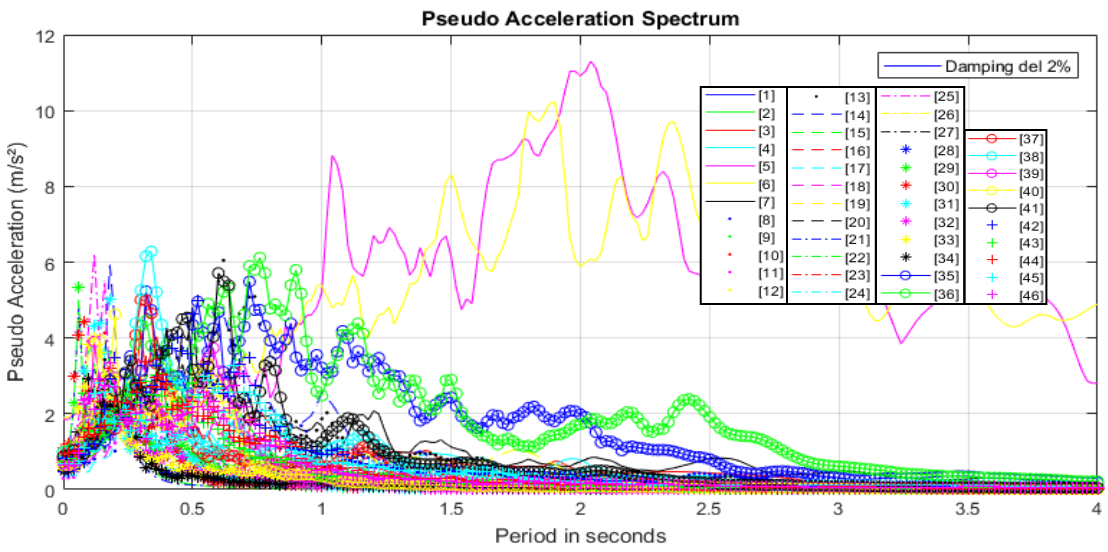

Fig. 5 – Espectro de Pseudo Aceleraciones

4.2 IDA curves

To generate the incremental dynamic analysis (IDA) curves, the 23 pairs of seismic records (46 Records) detailed in Table 6 were considered. The scaling factors vary from a value of 0.2 to a value of 20.0 times the value of the pseudo initial acceleration (0.2g), which is equivalent to 100 scaling values corresponding to pseudo-acceleration values from 0.04g to 4.00g. In total, 4600 nonlinear history time analyzes were performed, of which the floor displacement responses were obtained. The incremental dynamic analysis curves (IDA Curves) are constructed from the maximum displacements of the structure for each record and for each level of scaling. Fig. 6 and Fig. 7 show the IDA graphs corresponding to the displacements between floor of level 1 and level 2 respectively depending on the pseudo acceleration value.

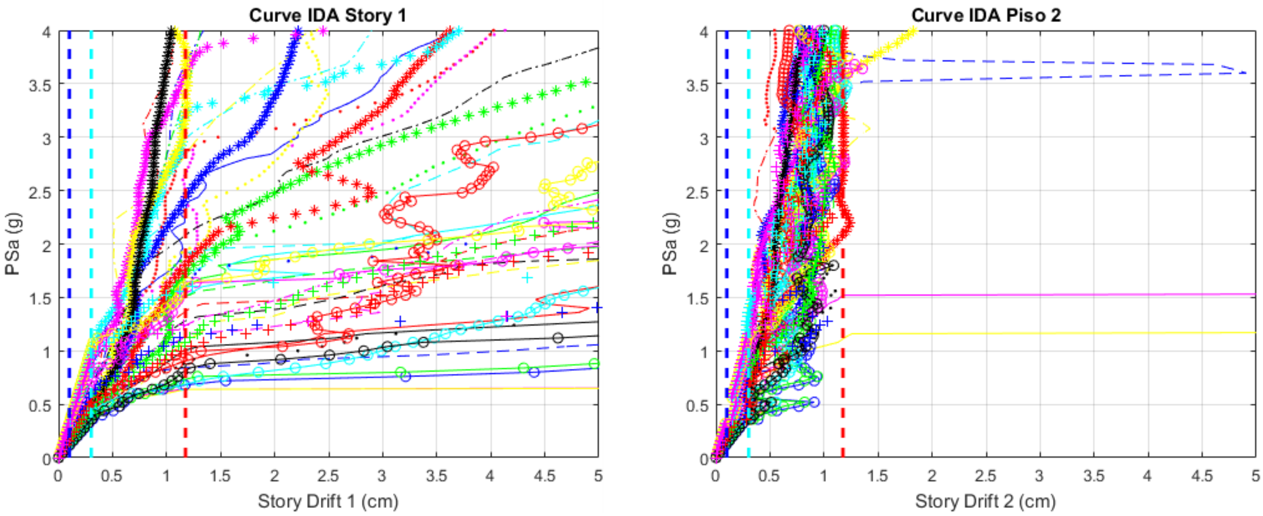

Fig. 6 – IDA Curve – Story 1 and 2

In Fig. 6 and Fig. 7, segmented vertical lines are shown representing the travel limits associated with the levels of immediate occupancy performance OI (Blue line), life prevention PV (Cyan line) and safety of SC collapse (red line). The results of the incremental dynamic analysis show that the incursion in the inelastic range of story 1 and 2 is more noticeable than in story 3, 4 and 5. Structural instability of the building is also observed for different levels of intensity with greater notoriety in levels 1 and 2.

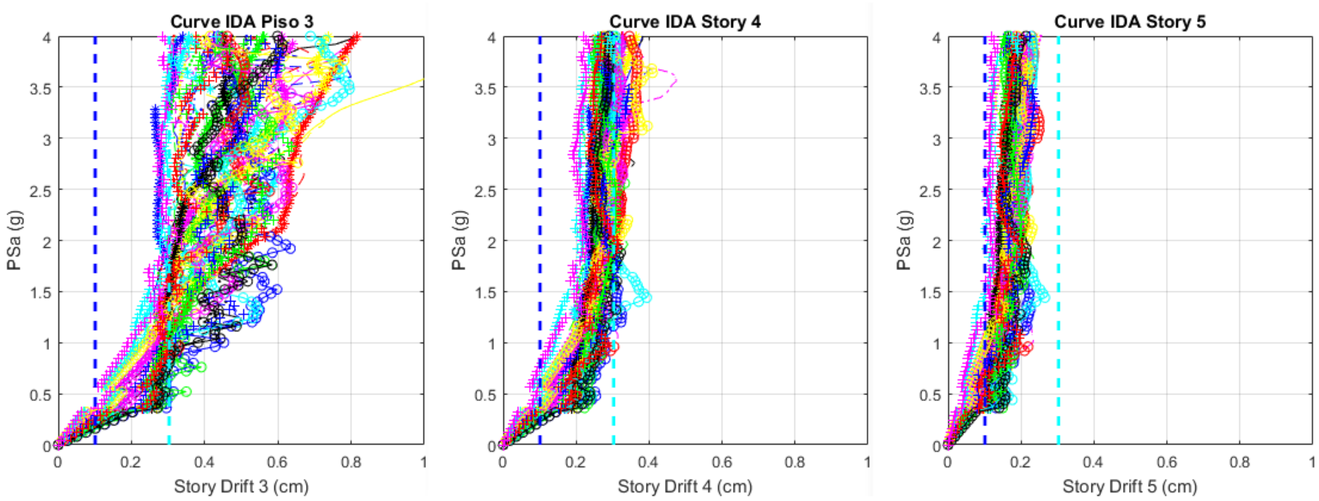

Fig. 7 – IDA Curve – Story 3, 4 and 5

5. Fragility Curves

5.1 Performance Levels

The performance level of a structure is associated with some damage limit state. We understand by Damage to the deterioration suffered by the structures subjected to external forces, in turn, this deterioration is related to the response of the structure, displacements, efforts, etc. Therefore the performance levels represent a boundary condition based on the possible damage to the structure, the threat to the safety of the occupants of the building induced by these damages and the functionality of the post-earthquake building [13]. Fragility curves were defined according to performance levels that represent different indicators of damage. Thus, three (3) levels of performance, level of immediate occupation performance (OI), level of life prevention performance (PV) and level of collapse safety performance (SC) have been defined according to Carrillo and Alcocer (2012) [3]. The OI level refers to minimal damage to the structure, the PV level refers to damage that can be repaired, but with a low probability of failure and the SC level to severe structural damage with a high probability of collapse of the structure. Table 7 shows the relationship between performance levels and design resistance.

Table 7 – Indicators of performance

Performance level	Design resistance	Force (kN)	Displacement (mm)
OI	0.25 V_n	954.31	1.01
PV	0.75 V_n	2862.93	3.03
SC	1.00 V_n	3817.00	11.75

5.2 Fragility Curves

The fragility curves of the structure are constructed from the response of the story. The fragility curves presented in this section are constructed from the results of the incremental dynamic analysis. In general, the probability associated with an event can be calculated with the Eq. 7.

$$P(E) = \frac{Item\ number\ of\ E}{Item\ number\ of \Omega} \tag{7}$$

Where, $P(E)$ is the probability that an event occurs, E is the event and Ω is the sample space. Therefore, the probability of exceeding a certain level of performance was calculated by dividing the number of seismic events, for a given level of seismic intensity (IM) that generate a displacement value greater than or equal to those defined in Table 7, on the Total seismic events considered. Additionally, for each of the three performance levels, a log-normal distribution curve with an average equal to the median of the recorded collapse intensities and standard deviation β is adjusted. The adjustment proposed by Baker (2014) [4], is used to represent the fragility curves obtained from the IDA analyzes. The adjustment consists in representing a fragility curve by means of a log-normal cumulative distribution function according to the Eq. 8.

$$P(C|IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \tag{8}$$

$P(C|IM = x)$, is the probability that an earthquake with intensity $(IM = x)$ causes the structure to collapse, Φ is the normal cumulative distribution function with a mean equal to θ and β the standard deviation of $ln(IM)$ [4]. θ is the median of the fragility function (the level of IM with 50% probability of collapse); and β is the standard deviation of $ln(IM)$, (sometimes referred to as the IM dispersion) [4].

$$ln(\theta) = \frac{1}{n}\sum_{i=1}^{n} ln(IM)_i$$

$$\beta = \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n}[ln(IM_i/\theta)]^2}$$

Table 8 shows the parameters θ and β for the adjustment of the fragility curves of story 1 and 2 respectively.

Table 8 – Parameters θ and β for fragility curve adjustment - Story 1 and 2

Parameters θ y β						
Parameter/ performance level	Story 1			Story 2		
	OI	PV	SC	OI	PV	SC
θ	0.212	0.578	1.571	0.238	0.700	3.578
β	0.292	0.281	0.533	0.244	0.360	0.239

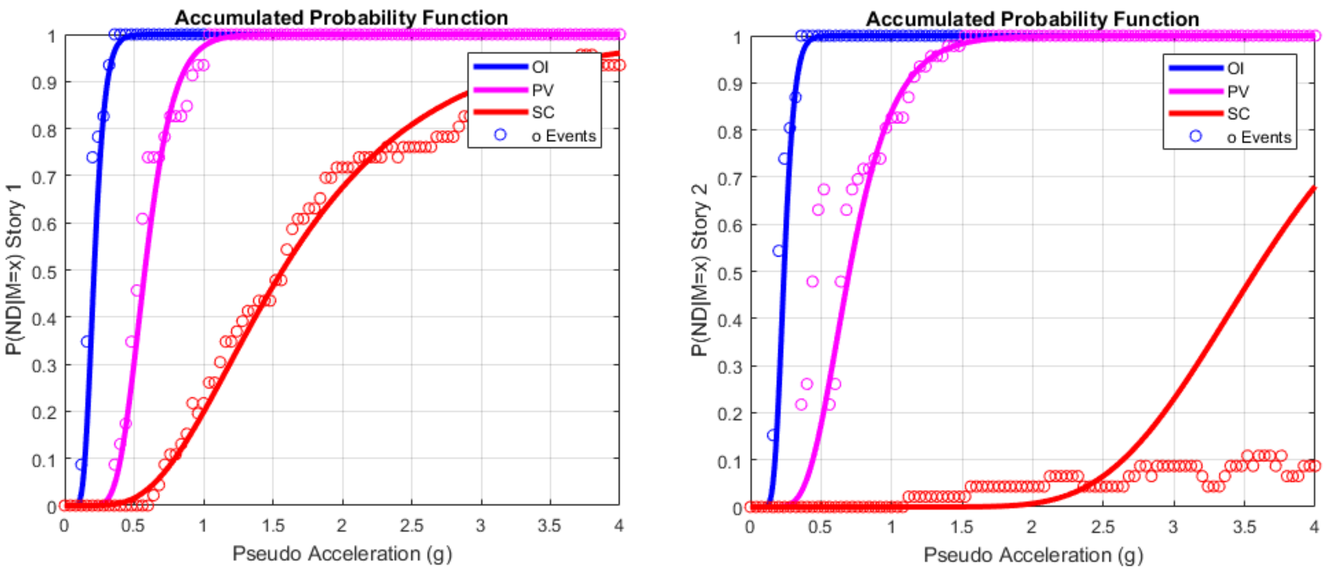

Fig. 8 – Fragility curves – Story 1 and 2

Fig. 8 shows the fragility curves for story 1 and 2. For story 1 of the structure the results show that the probabilities of exceedance associated with the level of performance of OI and PV are quite high. A pseudo acceleration of 1.00g means that the probability of exceeding the level of performance of OI is 100%, that is, with a pseudo acceleration of 1.00g, the displacement of 1.01 mm would be exceeded. A pseudo acceleration of 1.00g means that the probability of exceeding the PV performance level is 98%, that is, with a pseudo acceleration of 1.00g, the displacement of 3.03 mm would be exceeded. A pseudo acceleration of 1.00g means that the probability of exceeding the performance level of SC is 20%, that is, with a pseudo acceleration of 1.00g, the displacement of 11.75 mm would be exceeded.

6. Conclusiones

1. Fragility curves allow estimating the level of damage of a structure given an earthquake intensity. The incursion into the inelastic range of a building is expected to be present in the story 1.
2. To obtain the curves of the incremental dynamic analysis, the pseudo acceleration corresponding to the fundamental period of the structure was used as a seismic intensity parameter. The use of PGA or pseudo acceleration values will depend on the period of the structure. For rigid structures the use of PGA would be reasonable, but for flexible structures the use of PSA is more advisable.

3. The results show that none of the story of the structure enter the level of OI performance. This level of performance is quite restrictive and quite high floor stiffness values would be required.
4. In this study, seismic events of greater magnitude do not necessarily generate large displacements in the structure. The analysis of the response spectra are much more conclusive regarding the structural response.

7. Recomendaciones

1. Opensees is a fairly powerful computational tool. The generation of more elaborate mathematical models would be a substantial improvement when conducting this type of study.
2. It is recommended to make a comparison between the fragility curves obtained considering the parameter of the pseudo acceleration and the PGA value of the seismic records. Also consider the differences between floor rigidities.
3. It is recommended to expand the criteria for selecting seismic records and perform the respective evaluations.
4. Consider in the work intermediate and deep seismic events for the generation of fragility curves.
5. Associate the results of the fragility curves with the seismic vulnerability and risk of masonry buildings.

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