SEISMIC LOSS ANALYSIS COMPARISON BETWEEN 2D AND 3D MODEL FRAMES FOR A PERUVIAN UNIVERSITY BUILDING WITH SIMULATED FRAGILITY FUNCTIONS

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Abstract

University buildings in Perú are considered critical structures which do not present enough information about its seismic vulnerability. This research develops a probabilistic methodology that estimates seismic losses for a Peruvian university building based on fragility functions. These functions represent the overall building behavior through 2D and 3D model frames. These results will permit to know if 2D analyzed results are sufficient to know the real 3D structure behavior. Latin Hypercube technique, an improved Montecarlo-based method, allowed to generate fragility functions through a simulation process. This method creates 100 reliable samples of structural parameters for every level of seismic demand. Three structural parameters were considered in the simulation process as follows: Concrete compressive strength, maximum concrete strain and yield stress of the reinforcing steel. Synthetic records defined seismic demand and these signals were compatible with the elastic Peruvian design spectrum. Acceleration records were scaled based on the peak ground acceleration on rigid soil (PGA) which goes from 0.05g to 1.00g. A total of 2000 structural models were created considering both structural and seismic variability.

The university building shows an expected Mean Damage Factor of 18.40% and 20.25% in X direction and 12.65% and 8.80% in Y direction, for 3D and 2D model frames respectively; considering a 0.22g-PGA scenario, which was amplified by the soil type coefficient and resulted in 0.26g-PGA. These ratios were computed considering a seismic demand related to 10% of probability of exceedance in 50 years which is a requirement in the Peruvian seismic code. These results show an acceptable seismic performance.

Keywords: Seismic Vulnerability, Fragility Curves, Monte Carlo simulation, University Buildings.

1. Introduction

Risk mitigation plans allow us to estimate structure seismic vulnerability for essential buildings. Peru is located at a high seismic zone called Pacific Fire Ring and seismic vulnerability studies are highly necessary here. This study focuses on the analysis of one reinforced concrete frame building located in Cusco city which is represented by 2D and 3D model frames. The lack of information from recent earthquakes is an important factor which may not guaranty a good structural behavior according to Peruvian seismic design code. The main purpose is developing fragility functions for seismic loss assessment of a Peruvian university building. Fragility functions provide the probability of exceedance for a given damage state (LS) and an Intensity Measure (IM). Empirical fragility functions are not accurate solutions for buildings located in a country that has a lack of after-earthquake damage information. For that reason, this research applies a simulation-based analytical method. The fragility functions are calculated by considering uncertainties in structural capacity and seismic demand. Structural capacity is considered by three parameters: the compressive concrete strength (f'_c), the ultimate concrete strain (ε_{cu}) and the yielding stress of the reinforcement steel (f_y). Variability of seismic demand is considered by artificial accelerograms which were constructed for different IM levels. Loss estimation is predictable considering a simplified methodology proposed by Hwang and Lin [1]. This procedure defines a Mean Damage Factor (MDF) related to the most likely seismic hazard scenario which is related to fragility functions.

2. University building in Cusco

Nursing Faculty Building (NFB), at the UNSAAC (which stands for Universidad Nacional de San Antonio Abad del Cusco, in Spanish), is the analyzed building. Building structural system was design with reinforced concrete frame elements. A total of 2000 structural models were analyzed for each 2D and 3D models in both directions X and Y. Both models were represented by frames defined in Fig.1. This simplified representation is permitted for a building which has a regular configuration along both directions. Structural models did not consider torsion effects. NFB presents 20-cm depth diaphragms made of light concrete and its structural elements distribution is shown in Table 1. The H-frame and 1-frame were selected for the nonlinear analysis.

Table 1 – Structural elements distribution in the NFB

Description	Shape	Name	Section (m)	Direction
Main beams	Rectangular	VP1, VP2	0.30x0.45 and 0.30x0.65, respectively	Y
Secondary beams	Rectangular	VS1, VS2	0.30x0.45 and 0.30x0.65, respectively	X
Column	Rectangular	C1, C2, C6 and C7	0.30x0.60	Y (depth)
Column	Rectangular	C3 and C5	0.30x0.45	Y (depth)
Column	Rectangular	C4	0.30x0.50	Y (depth)

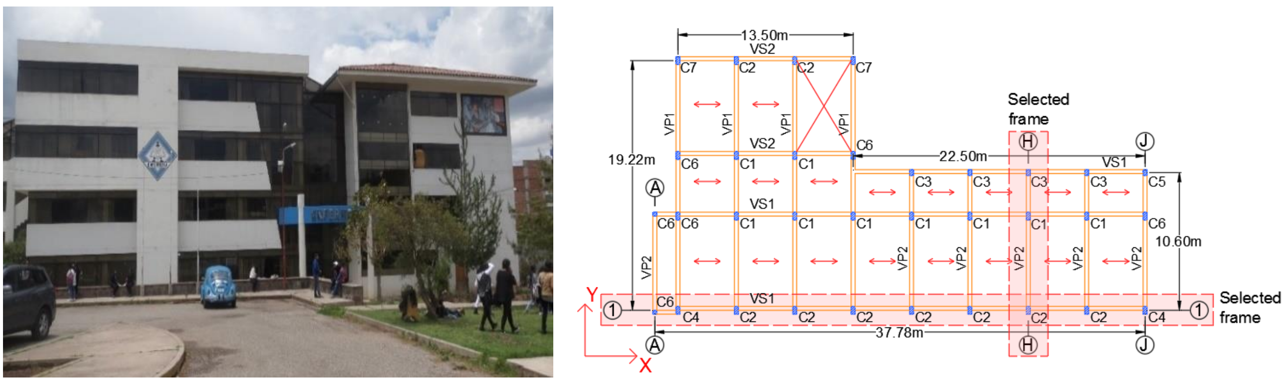


Fig. 1 – NFB front view (left) and structural plan view (right)

1997-Peruvian seismic code [2] defined NFB seismic performance. 1996 Nazca earthquake changed lateral drift limits from 1977-seismic code and these limits were more restrictive. 1997-code included shear forces 1.25 times and displacements 2.50 times larger than the 1977-seismic code parameters.

3. Seismic hazard in Cusco city

In order to estimate the seismic demand in Cusco city, a Probabilistic Seismic Hazard Assessment (PSHA) was performed. Ground motion and structural behavior response were considered in analytical model for decision-making purposes. Previous seismic hazard research has been carried out in Peru. This study considered Tavera et al. [3] seismic sources for PSHA.

3.1 PSHA review

The probability of occurrence for strong ground motions is accurately represented by the Poisson model which is an important probabilistic model in engineering. The Poisson model is defined by Eq. (1).

$$P(N=n)=\frac{e^{-\lambda t}(\lambda t)^n}{n!} \tag{1}$$

Where t is the time frame to be considered, λ is the annual rate of exceedance for earthquakes considering a given magnitude related to an Intensity Measure (IM) such as peak ground acceleration (PGA) and N is the number of earthquakes that occur during this time t.

The acceleration A, which can be defined as the PGA, is considered as a function of the earthquake size (magnitude) and the distance to the site. The earthquake size S and its epicenter location R are considered random continuous variables and these values are defined by probability density functions $f_S(s)$ and $f_R(r)$, respectively. Then, the probability that A is equal or higher than a certain acceleration a, that is $P(A \geq a)$, is defined by PSHA. This is given by Eq. (2).

$$P(A \geq a)=\int\int P[A>a|m,r] f_S(s) f_R(r)\, dsdr \tag{2}$$

Cornell [4] methodology considers the application of this integral for a typical PSHA. The program CRISIS, developed by Ordaz et al. [5], allowed to evaluate this integral in a numerically way.

3.2 Seismic sources

This study considered the 33 Peruvian seismic sources in Peru proposed by Tavera et al. These sources were defined according its spatial distribution of the seismic occurrence related to the subduction process (interface), the main fault systems (cortical) and the Nazca plate geometry underneath the continent (intraplate). The seismic sources are classified as follows: F-1 to F-8 for the interface seismicity, F-9 to F-19 for the seismicity related to the cortical deformation and F-20 to F-33 for the intraplate seismicity.

3.3 Recurrence law and seismic intensity

Gutenberg-Richter recurrence law permits to characterize seismic sources as shown in Eq. (3). This expression represents the number of seismic events N_m whose magnitudes are higher than m. a and b are constants taken as statistical parameters and these values were computed for each seismic source. This recurrence law has two limits: maximum and minimum, and becomes Eq. (4).

$$log N_m = a - bm \tag{3}$$

$$N_m = v\left[\frac{e^{-\alpha(m-m_{min})}-e^{-\beta(m_{max}-m_{min})}}{1-e^{-\beta(m_{max}-m_{min})}}\right]; m_{min} \leq m \leq m_{max} \tag{4}$$

Where υ denotes the average annual exceedance rate, α and β are constants for each seismic source, m_{max} and m_{min} represent the maximum and minimum magnitude, respectively. These values are known as seismological parameters. Recurrence law is described by these seismological parameters for each Peruvian seismic source. Some typical values are shown in the Table 2.

Table 2 – Typical seismological parameters in three seismic sources

Source	m_{min} (Mw)	m_{max} (Mw)	Beta (β)	Average annual rate (υ)
1	5.20	8.80	1.84	2.03
2	4.30	8.20	1.66	11.54
3	4.30	8.00	1.78	12.83

3.4 Attenuation relationship

The dependence of two main seismic parameters are described by attenuation relationship: magnitude (m) and earthquake origin distance (R). Seismic ground movement can be characterized by these parameters. Attenuation laws describe ground movement decreasing as a function of m and R. This research considered two attenuation laws for PSHA assessment: Youngs et al. [6] model for subduction earthquakes related to interface and intraplate processes (sources from F1 to F8 and from F20 to F33, respectively), and Sadigh et al. [7] model for cortical earthquakes (sources from F9 to F19).

3.5 PSHA assessment in Cusco

The variation of the peak ground acceleration (PGA), velocity, displacement or any other Intensity Measure (IM) are shown in the Seismic Hazard Curve (SHC). SHC is a function of the annual rate of exceedance. Fig.2 shows this curve in terms of PGA and the Uniform Hazard Spectrum curve determined by CRISIS, for the city of Cusco.

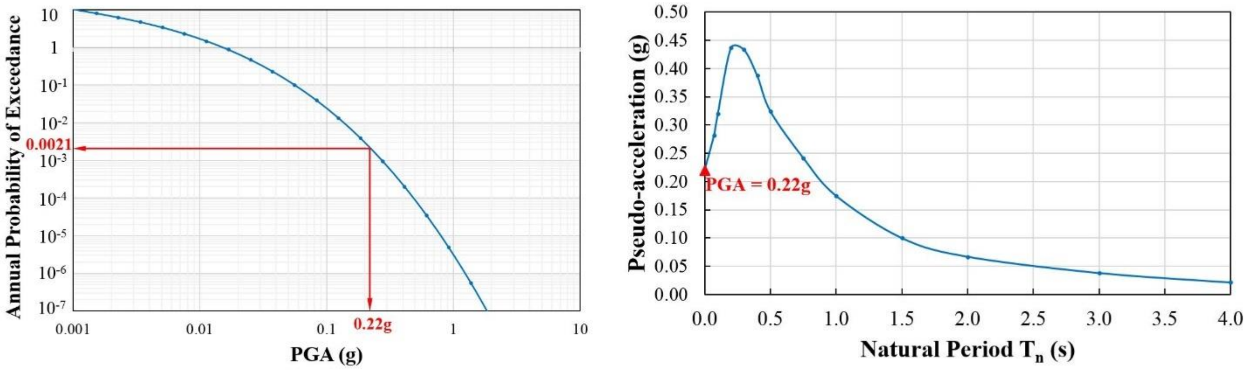


Fig. 2 – Probabilistic seismic hazard curve (left) and uniform hazard spectrum (right), for a return period of 475 years.

A PGA of 0.22g was obtain from SHC. This value allowed to estimate damage ratios in analytical fragility functions. A return period of 475 years (10% of exceedance in 50 years) was considered in PSHA according to Peruvian seismic design code. The uniform spectrum is obtained for NFB location.

4. MonteCarlo simulation

Montecarlo simulation process permits to compute physical systems response against probabilistic events by considering various analyzed samples of these systems. Principal input data was defined by probability density functions in this process. Remember that, structures with a similar building process can have distinct mechanical properties. Thus, its structural behavior may differ. MonteCarlo simulation technique considers this variation.

4.1 Uncertainty in the structural parameters

Three structural parameters were considered inside simulation process as follows: the concrete compression strength (f'_c), the ultimate concrete strain (ε_{cu}) and the yielding stress of the reinforcing steel (f_y). Velásquez et al [8] suggested some values for these parameters according to experimental data. A normal probability density function with a coefficient of variation of 15% and an average of 21 MPa is accurate enough for f'_c. Ultimate deformation strain was considered with same distribution as f'_c, an average of 0.004 (Hognestad model) and 15% coefficient of variation. For the f_y, a lognormal probability density function with 6% coefficient variation and an average of 420 MPa was defined. The Latin Hypercube technique was defined for the sampling procedure. Probability density functions permitted to select random values of each structural parameter from the probability density functions shown in Fig.3. Hence, 100 random samples were generated for each IM by using a MATLAB script.

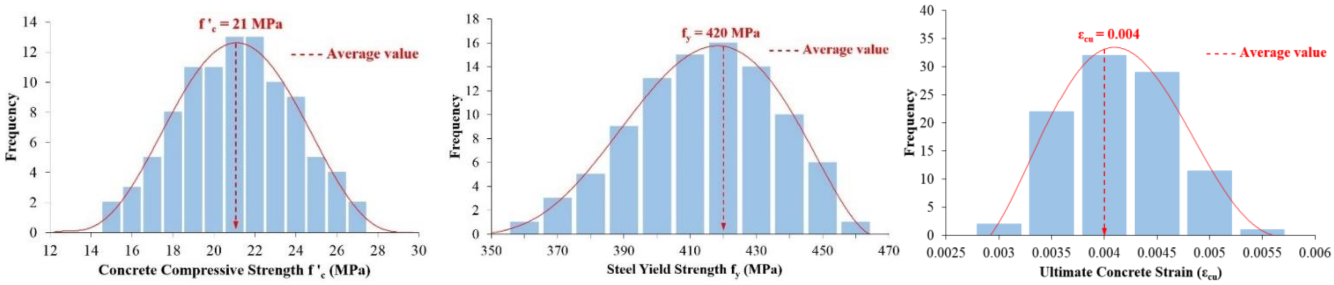


Fig. 3 – Sample population for random structural parameters

4.2 Uncertainty in the seismic parameters

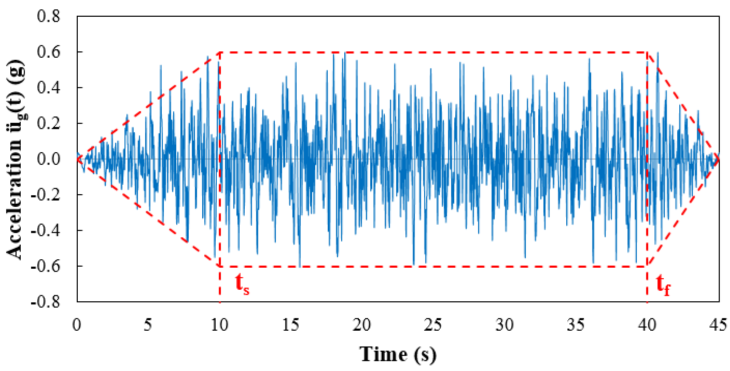
Artificial records were used to consider the seismic action in the Nonlinear Time History Analysis (NTHA). These were generated due to the lack of recent seismic records. Also, in order to bring the building into its inelastic range, it was necessary to consider a wide range of seismic intensities. Besides, these accelerograms were scaled to the elastic response design spectrum detailed in the Peruvian seismic code E030. Common artificial generation methods use periodic functions, such as the acceleration $\ddot{u_g}(t)$, to create new signals. This periodic function which defines the seismic action, can be expressed as a series of sinusoidal waves as shown in Eq. (5).

$$\ddot{u_g}(t) = \sum_{k=1}^{n} A_k sen(w_k t + \emptyset_k) \tag{5}$$

Where A_k is the amplitude, w_k is the angular frequency and $\emptyset_k$ is the phase angle of the k sinusoidal contribution. To consider the shape of the transient behavior of real earthquakes, the stationary movement $\ddot{u_g}(t)$ is multiplied by a function called deterministic intensity envelope $I(t)$. Finally, the simulated movement is given by Eq. (6).

$$\ddot{u_g}(t) = I(t) \sum_{k=1}^{n} A_k sen(w_k t + \emptyset_k) \tag{6}$$

According to Moreno [9] guidelines, a trapezoidal intensity function was chosen, and accelerograms were defined with the following considerations: a total duration of 45 seconds was defined which included 10 seconds of rising time (t_s) and 40 seconds of strong motion (t_f). A typical artificial signal is shown in Fig.4.



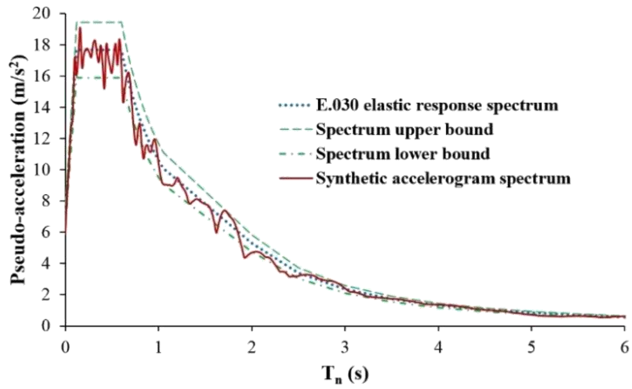
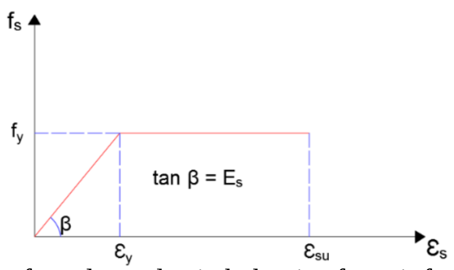


Fig. 4 – Synthetic signal scaled up to a PGA of 0.60g (left) and its corresponding elastic compatible response spectrum according to the E030 [MVCS 2014] (right)

SeismoArtif [10] program permitted to generate the artificial signals considering a random process, which produced records compatible to the uniform hazard spectrum proposed by the Peruvian seismic design code. PGA values went from 0.05g to 1.00g with increments of 0.05g.

4.3 Nonlinear time history analysis (NTHA)

Latin Hypercube technique permitted to generate a random population of 3D and 2D frames and these samples were modelled in SAP2000 [11]. Plastic hinges represented material plasticity in nonlinear behavior. Concentrated inelastic behavior in structural elements was achieved with plastic hinges. Paulay and Priestley [12] determined plastic hinges at an equivalent length (Lp) of 0.5h, where h is the height of the cross section of the element. If we consider a constant curvature along this plastic hinge length, the rotation can be computed by multiplying the length and the curvature. Constitutive laws of materials and hysteretic relationships allow to define plastic hinge models. A perfect elastoplastic behavior was considered for the reinforcing steel, and the Hognestad model for unconfined concrete was selected to the concrete cross sections. Material constitutive laws are shown in Fig.5.



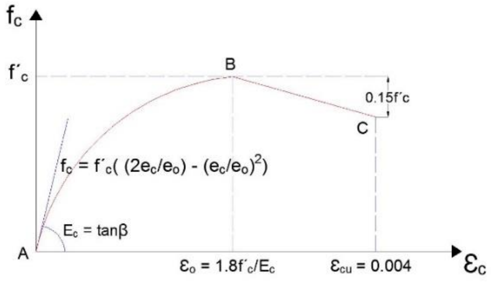


Fig. 5 – Perfect elastoplastic behavior for reinforcing steel (left) and Hognestad model for the concrete (right)

Inelastic deformation capacity of a structural element can be computed by moment-curvature relationships. VP2 and C1 first floor sections of the NFB are shown in Fig.6. The VP2 beam was reinforced with 3Φ5/8” on the compression side and with 3Φ5/8” + 4Φ3/4” on the tension side. Also, C1 column had a reinforcement of 4Φ3/4” + 8Φ1”. Fig.7 shows moment-curvature relationships for VP2 beam and C1 column sections. C1 moment-curvature diagram is highly affected by the axial load level which was considered from the second load combination of the Peruvian concrete design code E060 [13] (i.e. 1.25 DL + 1.25 LL). This load level was taken in the moment-curvature relationships for the reinforced concrete cross sections during the NTHA.

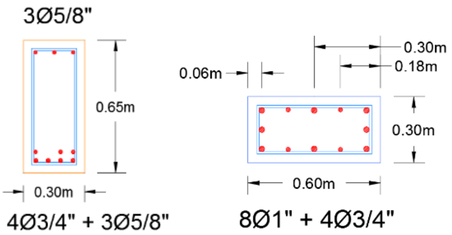


Fig. 6 – Cross sections of the V2 beam (left) and the C1 column (right) for the 1st story of the NFB

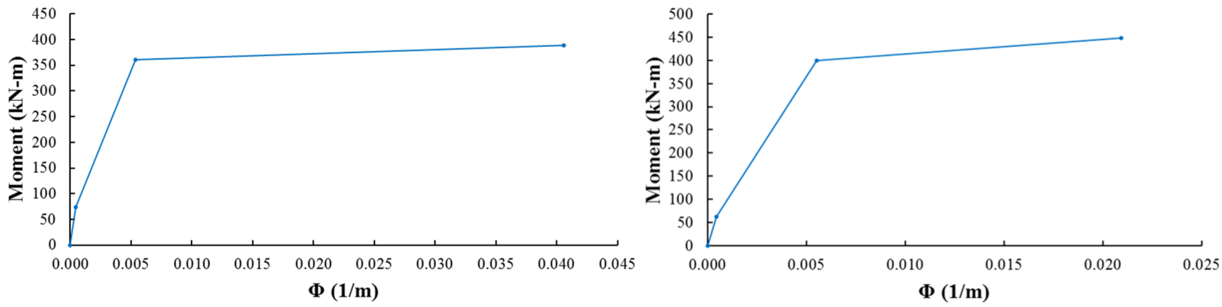


Fig. 7 – Moment-curvature plots for sections V2 (left) and C1 (right) for the 1st story of the NFB

Takeda et al. [14] model defined hysteresis behavior in concrete sections, and it is considered appropriated for the concrete structural elements when we describe energy dissipation and dominant flexure failure. Takeda model and the corresponding Moment-Rotation relationship are shown in Fig.8. This model was applied for all the plastic hinges used in the NTHA. With the previous hypothesis and using an algorithm involving SAP2000, a total of 2000 random structural models were analyzed.

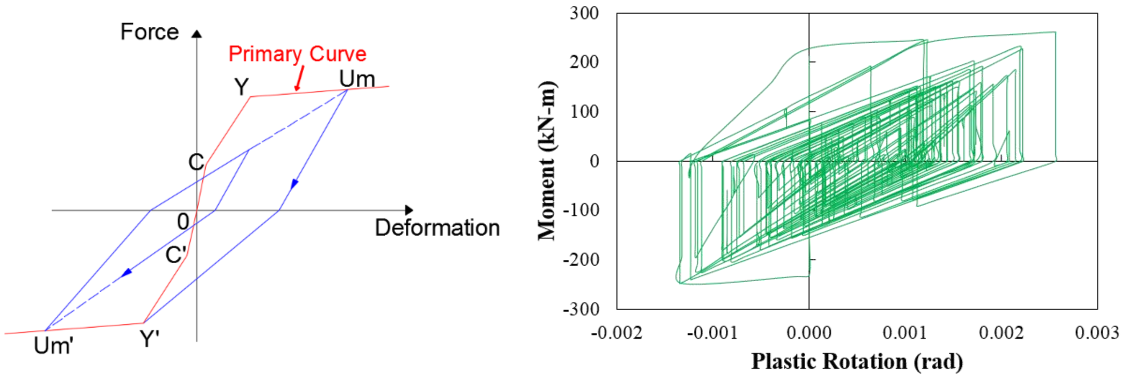


Fig. 8 – Takeda et al. model (left) and hysteretic behavior of the C2 column (right)

5. Fragility functions

5.1 Damage limit states

Damage limit states were defined by a Pushover analysis. This method permits us to obtain the Capacity Curve of the building which was constructed considering a series of incremental static nonlinear analysis. Material constitutive models and cross section hysteresis behaviors defined previously are essential elements in the Capacity Curve definition. To build this curve, it is usual to consider the first mode response (fundamental mode) which is an important parameter in regular low-rise buildings. Roof displacement was increased progressively and at the same time the base shear was registered. This procedure is repeated until achieving the maximum lateral displacement capacity [15]. Aguiar [16] proposed simplified bilinear and trilinear models where inside and outside areas of capacity curves are equivalent. Both cases can be defined in SAP2000 and Fig.9 shows NFB Capacity Curves its corresponding Limit States.

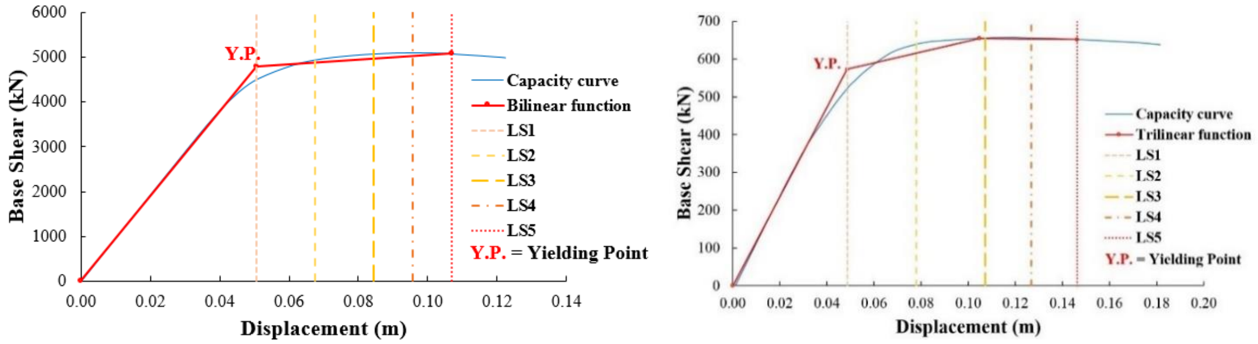


Fig. 9 – Capacity Curves for 3D model frame (left) and 2D model frame (right), both in NFB X direction

VISION 2000 committee [17] proposed Limit States for buildings considering its elastic and inelastic range between the yielding point and the ultimate displacement in the Capacity Curve. Each damage state is related to a performance level and roof displacement. VISION 2000 limits were applied in this study to define the damage limit states. These are shown in Table 3 and Table 4 for X and Y directions.

Table 3 – Damage limit states for 3D and 2D model frames, both for NFB X direction in terms of the interstory drift ratio

Damage limit state	**3D Model frame – X direction**		**3D Model frame – X direction**	
	Max. interstory drift ratio	**Roof displacement (m)**	**Max. interstory drift ratio**	**Roof displacement (m)**
LS1: Fully operational	0.56%	0.051	0.50%	0.045
LS2: Operational	0.86%	0.068	0.82%	0.067
LS3: Life safety	1.07%	0.084	1.14%	0.090
LS4: Collapse prevention	1.26%	0.096	1.33%	0.105
LS5: Collapse	1.44%	0.107	1.52%	0.120

Table 4 – Damage limit states for 3D and 2D model frames, both for NFB Y direction in terms of the interstory drift ratio

Damage limit state	**3D Model frame – X direction**		**3D Model frame – X direction**	
	Max. interstory drift ratio	**Roof displacement (m)**	**Max. interstory drift ratio**	**Roof displacement (m)**
LS1: Fully operational	0.49%	0.039	0.54%	0.049
LS2: Operational	0.89%	0.066	0.87%	0.078
LS3: Life safety	1.09%	0.093	1.22%	0.107
LS4: Collapse prevention	1.28%	0.111	1.46%	0.127
LS5: Collapse	1.47%	0.129	1.68%	0.146

5.2 Damage probability density functions

The function $f(x)$ defines the probability density function of a random continuous variable. This function can be integrated in the interval $[a, b]$ and represents the probability of occurrence for real values between a and b. This is shown in Eq. (7).

$$P[a \leq x \leq b] = \int_a^b f(x)dx \tag{7}$$

A cumulative probability density function $F_X(x_i)$ (CDF) is obtained from the integration of Eq. (7) between -∞ and x. This can be determined numerically by organizing the results in ascending order and applying Eq. (8). This was proposed by Ruiz [18] as follows.

$$F_X(x_i) = \frac{i}{n(S)} \tag{8}$$

where x_i is the event that may repeat i times inside a population with sample size $n(S)$. If we compute the CDF over the entire real domain, Eq. (9) is obtained.

$$\int_{-\infty}^{\infty} f(x)dx = F_X(\infty) = \lim_{x\to+\infty} F(x) = 1 \tag{9}$$

The probability of being less than infinity is 1. Besides, the complementary event has the probability named as the Probability of Exceedance, and it can be computed by Eq. (10).

$$P[X > x] = 1 - F_X(x) \tag{10}$$

Fragility functions can be represented by The Probability of Exceedance and its rate represents the limit value of damage states. A total of 2000 models were generated by the simulation process and these samples represented numerical probability density functions. The simulation process was implemented with MATLAB [19] scripts. Scripts included different simulation procedures as follows: input data definition related to the geometry and properties of the buildings, PGA parameters generated using Latin Hypercube, NTHA parameters definition by using the interaction between MATLAB and SAP2000, and PDF and CDF parameter generation. At the end, the CDFs for the maximum interstory drift ratios were determined for several IM values. In this research, the peak ground acceleration (PGA) is defined as the IM, with values from 0.05g to 1.00g.

5.2 Fragility functions and seismic loss assessment

The fragility functions show the probability of exceedance for a Damage Limit State (DLS) given an Intensity Measure IM. The IM was considered in terms of the PGA. Fragility functions generating methods adjust values into a lognormal probability density functions ϕ such as the one shown in Eq. (11).

$$P(DLS \geq DLS_i/IM) = \phi\left[\frac{1}{\beta_{IM,DLS_i}} \ln\left(\frac{IM}{\overline{IM}}\right)\right] \tag{11}$$

Where $\overline{IM}$ is the average value of IM for the damage limit state DLS_i. The β_{IM,DLS_i} is the coefficient of variation for the DLS_i, and ϕ is the lognormal cumulative probability density function. Fragility functions were built for NFB as shown in Fig.10 and Fig.11, applying an analytical method implemented in MATLAB scripts.

Four limit states were defined in these figures. The upper bound for the collapse of the structure defined as LS5 was neglected. Further than this point there is not additional damage, so the collapse limit is specified between LS4 and LS5.

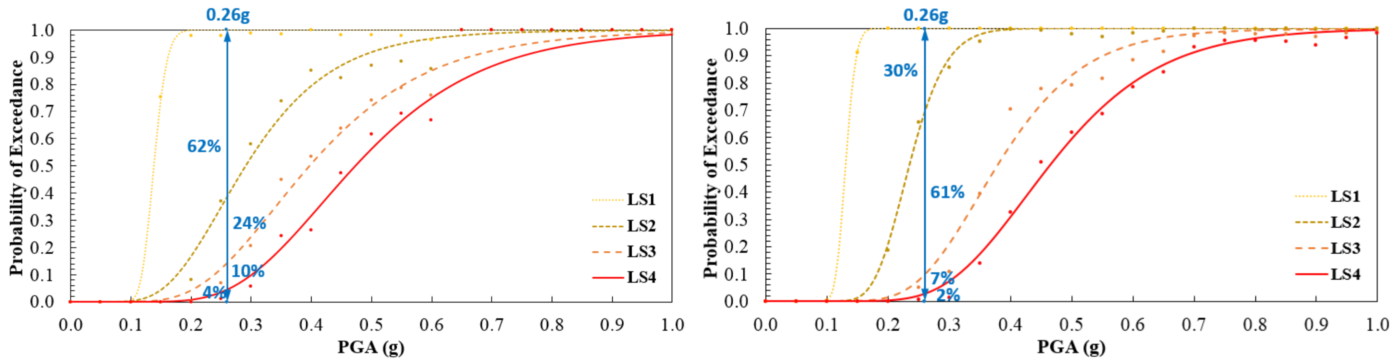


Fig. 10 – Simulated fragility functions for 3D model frame (left) and 2D model frame (right) in terms of PGA, both in NFB X direction

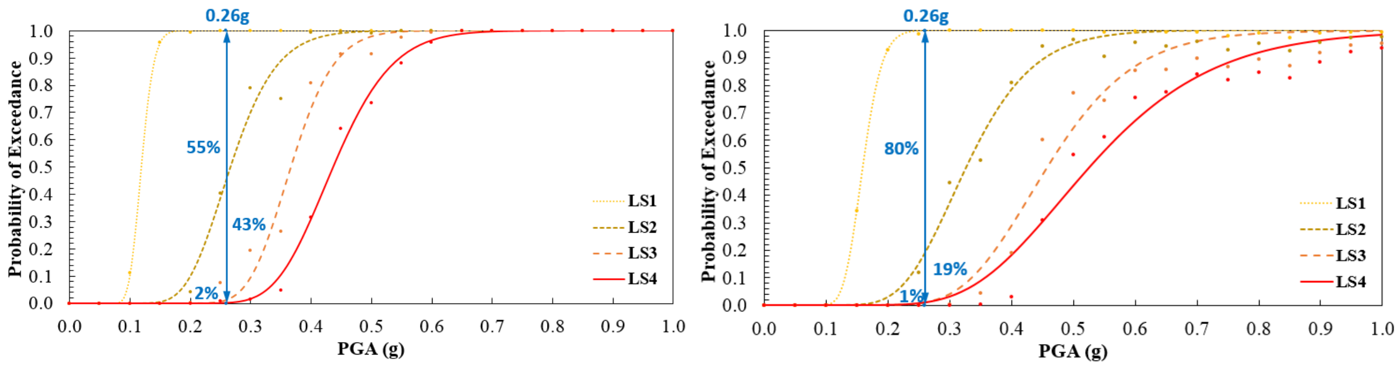


Fig. 11 – Simulated fragility functions for 3D model frame (left) and 2D model frame (right) in terms of PGA, both in NFB Y direction.

Loss ratios can be estimated by choosing seismic intensity scenarios. First, it is necessary to determine the probability of exceedance for all damage limit states given a certain PGA scenario. These results permit to define a Mean Damage Factor (MDF). MDF represents a mean loss rate which is also defined as the ratio between the repairing and the total replacement cost for a given building. Only structural damage is considered. It will be necessary to amplify the seismic intensity obtained by the PSHA for Cusco city (0.22g), by the soil type coefficient S defined as 1.2 by the Peruvian seismic code. Local soil conditions are represented by this coefficient wherein buildings are located, and its result is a PGA of 0.26g. Fig.10 and Fig.11 show the loss ratios between Damage Limit States represented by the fragility functions, and for a given PGA scenario of 0.26g. The MDFs were computed in Table 5. In this Table, the 0.45g-PGA scenario is also included, because it represents the Peruvian zone with the highest seismicity.

Table 5 – MDFs for 3D and 2D model frames, both for NFB X and Y directions in terms of 0.26g and 0.45g PGA scenarios

Damage state	DF	X Direction				Y Direction			
		3D Model Frame		2D Model Frame		3D Model Frame		2D Model Frame	
		0.26g	0.45g	0.26g	0.45g	0.26g	0.45g	0.26g	0.45g
No damage	0%	0%	0%	0%	0%	0%	0%	0%	0%
Slight	5%	62%	13%	30%	0%	55%	1%	80%	11%
Moderate	20%	24%	25%	61%	28%	43%	8%	19%	42%
Extensive	65%	10%	18%	7%	26%	2%	34%	0%	17%
Collapse	100%	4%	44%	2%	46%	0%	57%	1%	30%
MDF		**18.40%**	**61.35%**	**20.25%**	**68.50%**	**12.65%**	**80.75%**	**8.80%**	**50.02%**

MDFs allow to estimate repairing costs in a certain building by multiplying these factors by the total construction area and by a unitary cost related to the structural replacement cost. Table 6 shows these calculations considering a unitary cost of USD 250/m2.

Table 6 – Estimated repairing cost for 3D and 2D model frames, both for NFB X and Y directions in terms of 0.26g and 0.45g PGA scenarios

		0.26g-PGA	0.45g-PGA					0.26g-PGA	0.45g-PGA
Model Frame	**Dir.**	**MDF**	**MDF**	**Area (m2)**	**Stories**	**USD/m2**	**Cost (USD)**	**Cost (USD)**	**MDF**
3D	X	18.40%	61.35%	527.72	4	250	97 100.48	323 756.22	X
2D	X	20.25%	68.50%	527.72	4	250	106 863.30	361 488.20	X
3D	Y	12.65%	80.75%	527.72	4	250	66 756.58	426 133.90	Y
2D	Y	8.80%	50.02%	527.72	4	250	46 439.36	263 965.54	Y

6. Conclusions

A rational analytical method, to generate fragility functions, is proposed in this study for Peruvian university buildings. A simulation process was applied in order to obtain fragility functions. For this building, 2000 structural models were analyzed considering the variability of structural and seismic parameters. Also, building damage ratios were computed using the simplified procedure proposed by Hwang and Lin [2002] for a seismic scenario related to a Probabilistic Seismic Hazard Assessment (PSHA). A peak ground acceleration of 0.22g is expected in Cusco city according to the PSHA. This value is amplified by the soil factor due to local conditions. Therefore, a value of 0.26g was determined for the PGA. With the fragility curves, it was possible to assess an expected Mean Damage Factor of 18.40% and 20.25% for 3D and 2D model frames in X direction, and 12.65% and 8.80% for 3D and 2D model frames in Y direction, respectively. These ratios can be understood as an acceptable structural performance during the severe earthquake related to 475 years of return period or a 10% of probability of exceedance in 50 years of exposure. There is a good approximation between Mean Damage Factors in the X direction for 3D and 2D models, but it is necessary to verify other frames in Y direction because results do not converge adequately between 3D and 2D models. It is also shown that if NFB was built in the highest seismic zone of Peru, i.e. zone 4, the damage ratios would exceed 50% for 3D or 2D model in both directions. This scenario will probably require extensive repairing or most likely a demolition.

7. References

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