SEISMIC LOSS ASSESSMENT FOR PERUVIAN UNIVERSITY BUILDINGS WITH SIMULATED FRAGILITY FUNCTIONS

José VELÁSQUEZ\textsuperscript{1}, José RUIZ\textsuperscript{2}, Holger LOVON\textsuperscript{3}

ABSTRACT

Peruvian university buildings are critical structures for which very little research about its seismic vulnerability is available. This paper develops a probabilistic methodology that predicts seismic loss for university buildings with simulated fragility functions. Two university buildings located at the city of Cusco were analyzed. Fragility functions were developed considering seismic and structural parameters uncertainty. The fragility functions were generated with the Latin Hypercube technique, an improved Monte Carlo-based method, which optimizes the sampling of structural parameters and provides at least 100 reliable samples for every level of seismic demand. Concrete compressive strength, maximum concrete strain and yield stress of the reinforcing steel were considered as the key structural parameters. The seismic demand is defined by synthetic records which are compatible with the elastic Peruvian design spectrum. Acceleration records are 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 considered to account for both structural and seismic variability. These functions represent the overall building behavior because they give rational information regarding damage ratios for defined levels of seismic demand. The university buildings show an expected Mean Damage Factor of 8.8\% and 19.1\%, respectively, for the 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 for both buildings.

Keywords: Fragility Functions; University Buildings; Loss Assessment; Montecarlo Simulation; Latin Hypercube.

1. INTRODUCTION

Peru is located at the Pacific Ring of Fire zone. This area concentrates approximately 80\% of the largest earthquakes around the world (Kious and Tilling 1996). However, in the last 50 years no important earthquakes have been reported in Peru. This can be interpreted as a long seismic gap. Seismologists predict that the next earthquake will attain a magnitude of around Mw 8.0. This amount of energy may cause severe damage in essential structures such as university buildings. Estimating the risk of these structures is mandatory to elaborate risk mitigation plans. This study focuses on the analysis of two reinforced concrete frame buildings located in Cusco city which represent two Faculty Buildings. Cusco is considered as an intermediate seismic zone according to the Peruvian design code. Two important earthquakes were reported in this region: (1) the first one occurred in 1650 which some authors have recognized as Mw 7.6, and (2) the other one occurred in 1950 with Mw 6.0. The latter destroyed approximately 90\% of the city (Rudenstine and Galea 2012).

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Although most university buildings meet the seismic provisions required by the Peruvian design code, its good behavior may not be guaranteed due to the lack of information from recent earthquakes. This has not allowed the improvement of the current design code. This work aims at developing fragility functions for seismic loss assessment of Peruvian university buildings. Fragility functions provide the probability of exceedance for a given damage state and an Intensity Measure (IM). Developing empirical fragility functions for university buildings is not possible because of the lack of after-earthquake damage information. Instead, this research applies a simulation-based analytical method. The fragility functions are obtained by considering uncertainties in structural capacity and seismic demand. The uncertainties in the structural capacity are considered with three parameters: the compressive concrete strength ($f'_{c}$), the ultimate concrete strain ($\varepsilon_{cu}$) and the yielding stress of the reinforcement ($f_y$). Fragility functions enables loss assessment by following the simplified methodology proposed by Hwang and Lin (2002). This procedure leads to the definition of the Mean Damage Factor (MDF) related to the most likely seismic hazard scenario.

2. UNIVERSITY BUILDINGS IN CUSCO

The two analyzed buildings correspond to the Nursing Faculty Building (NFB) and the Electronic Engineering Faculty Building (EEFB) at the UNSAAC (which stands for Universidad Nacional de San Antonio Abad del Cusco, in Spanish). The elements of both buildings configure a reinforced concrete frame structure. To reduce the computational effort of analyzing many 3D models, only 2D frame models were considered. The planar model was defined by the most critical frame of the structure as shown in Figures 1 and 2. This representation is allowed since both buildings have regular configurations along both directions. Torsion effects were neglected.

2.1 Nursing Faculty Building (NFB)

This building consists of diaphragms made of 20-cm depth light concrete slabs, which are ribbed in the X direction as shown in Figure 1. Structural elements distribution is shown in Table 1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Shape</th>
<th>Name</th>
<th>Section (m)</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main beam</td>
<td>Rectangular</td>
<td>VP1</td>
<td>0.30x0.45</td>
<td>Y</td>
</tr>
<tr>
<td>Main beam</td>
<td>Rectangular</td>
<td>VP2</td>
<td>0.30x0.65</td>
<td>Y</td>
</tr>
<tr>
<td>Column</td>
<td>Rectangular</td>
<td>C3 and C5</td>
<td>0.30x0.45</td>
<td>Y (depth)</td>
</tr>
<tr>
<td>Column</td>
<td>Rectangular</td>
<td>C4</td>
<td>0.30x0.50</td>
<td>Y (depth)</td>
</tr>
<tr>
<td>Column</td>
<td>Rectangular</td>
<td>C1, C2, C6 and C7</td>
<td>0.30x0.60</td>
<td>Y (depth)</td>
</tr>
</tbody>
</table>

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

The H-frame was selected for the nonlinear analysis, as shown in Figure 1, due to its largest gravity load and its low lateral stiffness. This frame is composed by three columns: C1, C2, C3; with
0.30x0.65 m beams for all stories. Fragility curves were also developed for the critical frame in the X direction. This resulted in similar responses to those obtained by a 3D model. By analyzing a frame model in the Y critical direction, a huge computational effort is saved and the results are accurate enough for the fragility analysis.

The NFB was built according to the provisions required by the 1997-Peruvian seismic code (MVCS 2014). Due to 1996 Nazca earthquake, the 1977-code was improved in terms of lateral drift limits. This 1997-code includes shear forces 1.25 times and displacements 2.50 times larger than the 1977-code.

2.2 Electronic Engineering Faculty Building (EEFB)

The Electronic Engineering Faculty Building was designed according to the 2003-Peruvian seismic code. 1997 and 2003 codes share many similarities regarding the seismic demand. This building consists of diaphragms made of 25-cm depth light concrete slabs, ribbed in the X direction. Structural elements dimensions are shown in Table 2.

Table 2. Main structural elements distribution in the EEFB

<table>
<thead>
<tr>
<th>Description</th>
<th>Shape</th>
<th>Name</th>
<th>Section (m)</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main beam</td>
<td>Rectangular</td>
<td>VP</td>
<td>0.30x0.65</td>
<td>Y</td>
</tr>
<tr>
<td>Column</td>
<td>Rectangular</td>
<td>C-3 to C-6</td>
<td>0.30x0.70, 0.30x0.80, 0.25x0.30 and 0.30x0.60</td>
<td>Y (depth)</td>
</tr>
<tr>
<td>Column</td>
<td>Tee</td>
<td>C-2, C-9, C-11 and C-14</td>
<td>1.00x1.00, 1.00x0.80 and 1.00x1.05</td>
<td>Y (depth)</td>
</tr>
<tr>
<td>Column</td>
<td>L</td>
<td>C-1, C-7, C-8, C-10, C12, C-13 and C-15</td>
<td>0.30x0.60</td>
<td>Y (depth)</td>
</tr>
<tr>
<td>Shear wall</td>
<td>Tee</td>
<td>PL-1 and PL-2</td>
<td>Variable</td>
<td>X and Y</td>
</tr>
</tbody>
</table>

As shown in Figure 2, the H-frame is the most vulnerable in the Y direction for the nonlinear analysis due to the same reasons considered for the NFB. This frame is composed by two C-2 and one C-5 columns, with 0.30x0.65 m beams for all stories.

![Figure 2. EEFB front view (left) and structural plan view (right)](image)

3. SEISMIC HAZARD IN CUSCO CITY

A Probabilistic Seismic Hazard Assessment (PSHA) was carried out to estimate the seismic demand in Cusco city. This analysis is useful for decision-making purposes because it considers the ground motion and the structural response. Previous seismic hazard research has been carried out in Peru. In this study, the seismic sources defined by Tavera et al. (2014) were considered for this PSHA.
3.1 PSHA Review

The Poisson model is adequate to represent the probability of occurrence for strong ground motions, which are the most important for engineering purposes. This model is represented by Equation 1.

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

(1)

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

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

$$P(A \geq a) = \int \int P[A > a|m,r]f_S(s)f_R(r) \, ds \, dr$$

(2)

This integral was numerically computed by the program CRISIS, developed by Ordaz et al. (2007).

3.2 Seismic Sources

Tavera et al. (2014) proposed 33 seismic sources in Peru according to the 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 distributed 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

Each seismic source is characterized by a Gutenberg-Richter recurrence law as shown in Equation 3. This expression denotes the number of seismic events $N_m$ whose magnitudes are higher than $m$. Constants $a$ and $b$ are statistical parameters computed for each seismic source. This recurrence law is bounded by a minimum and maximum magnitude, and becomes Equation 4,

$$\log N_m = a - bm$$

(3)

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

(4)

where $\nu$ represent the mean annual exceedance rate, $\alpha$ and $\beta$ 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. These parameters describe the recurrence law for each seismic source in Peru. Some typical values are shown in the Table 3.

<table>
<thead>
<tr>
<th>Source</th>
<th>$m_{min}$ (Mw)</th>
<th>$m_{max}$ (Mw)</th>
<th>Beta ($\beta$)</th>
<th>Mean annual rate ($\nu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.20</td>
<td>8.80</td>
<td>1.84</td>
<td>2.03</td>
</tr>
<tr>
<td>2</td>
<td>4.30</td>
<td>8.20</td>
<td>1.66</td>
<td>11.54</td>
</tr>
<tr>
<td>3</td>
<td>4.30</td>
<td>8.00</td>
<td>1.78</td>
<td>12.83</td>
</tr>
</tbody>
</table>

Table 3. Typical seismological parameters in three seismic sources (Tavera et al. 2014)
3.4 Attenuation Relationship

Attenuation laws describe dependence between two seismic parameters: magnitude \((m)\) and earthquake origin distance \((R)\). These fundamental factors define ground movement during seismic events. Decrease in ground movement as a function of \(m\) and \(R\) is described by attenuation laws. Two attenuation laws were used for this PSHA assessment: Youngs et al. (1997) for subduction earthquakes of interface and intraplate (sources from F1 to F8 and from F20 to F33, respectively), and Sadigh et al. (1997) model for continental earthquakes (sources from F9 to F19).

3.5 PSHA Assessment in Cusco

The Seismic Hazard Curve shows the variation of the peak ground acceleration (PGA), velocity, displacement or any other Intensity Measure (IM) as a function of the annual rate of exceedance. Figure 3 shows this curve in terms of PGA and the Uniform Hazard Spectrum curve computed by CRISIS, for the city of Cusco in terms of pseudo-acceleration.

![Figure 3. Probabilistic Seismic Hazard Curve (left) and Uniform Hazard Spectrum (right), for a return period of 475 years.](image)

The Seismic Hazard Curve gives a PGA of 0.22g. This value allowed to estimate the damage ratios in the fragility functions. The return period considered was 475 years (10% of exceedance in 50 years) required by the Peruvian seismic design code. The spectrum is computed for the location of the studied buildings.

4. MONTE CARLO SIMULATION

Monte Carlo simulation is a technique which allows to compute the response of physical systems against stochastic events by analyzing many samples of these systems. The probability density functions are the most important input data in this process. Even if the buildings were built under similar conditions, the possibility of having distinct mechanical properties will always exist.

4.1 Uncertainty in the Structural Parameters

Three parameters were selected for the simulation process: the concrete compression strength \((f'_c)\), the ultimate concrete strain \((\varepsilon_{cu})\) and the yielding stress of the reinforcing steel \((f_y)\). According to experimental data suggested by Velásquez (2006), a normal probability density function with a standard deviation of 15% and a mean of 21 MPa is good enough for the \(f'_c\). The same distribution was assumed for the ultimate deformation strain with a mean of 0.004 (Hognestad model) and 15% of standard deviation. For the \(f_y\), a lognormal probability density function with standard deviation of 6% and a mean of 420 MPa was defined. The Latin Hypercube technique was used for the sampling procedure. The values of each structural parameters are chosen randomly from the probability density functions shown in Figure 4. Hence, 100 random samples were generated for each IM by using a MATLAB script.
4.2 Uncertainty in the Seismic Parameters

The seismic action was modeled in the Nonlinear Time History Analysis (NTHA) by synthetic records. These were generated due to the lack of recent seismic records. On the other hand, it was important to cover a wide range of seismic intensity to bring the building into the inelastic range. Also, these signals were scaled to the elastic response design spectrum proposed in the Peruvian code E030 (MVCS 2014). Most methods used for the generation of compatible records are based on periodic functions, such as the acceleration $\ddot{u}(t)$. This function which defines the seismic action, can be expressed as a series of sinusoidal waves as shown in Equation 5.

$$\ddot{u}(t) = \sum_{k=1}^{n} A_k \sin(w_k t + \phi_k)$$

Equation 5

$A_k$ is the amplitude, $w_k$ is the angular frequency and $\phi_k$ is the phase angle of the $k$ sinusoidal contribution. To simulate the transient behavior of real earthquakes, the stationary movements $\ddot{u}(t)$ is multiplied by a function of predefined deterministic intensity envelope $I(t)$. This way, the final simulated movement is given by Equation 6.

$$\ddot{u}(t) = I(t) \sum_{k=1}^{n} A_k \sin(w_k t + \phi_k)$$

Equation 6

A trapezoidal intensity function was chosen. Following Moreno (2006) guidelines, the signal was defined with 45 seconds of total duration which included 10 seconds of rising time ($t_r$) and 40 seconds of strong motion ($t_f$). A typical synthetic signal is shown in Figure 5.

These signals were generated in a random process by using the program SeismoArtif (Seismosoft 2016) which produced random synthetic 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.

It should be noted that also real seismic records were scaled up to defined levels of PGA and were used in this analysis. However, very conservative responses were attained on the frames. This was not consistent with expected results between 0.30 and 0.45g PGA. This might be due to the frequency content of these real earthquakes. Also, not enough seismic records are available in this region.
4.3 Nonlinear Time History Analysis (NTHA)

The random population of 2D frames generated with the Latin Hypercube technique were modelled in SAP2000 (Computers and Structures Inc. 2016). Material plasticity was modeled by concentrated plastic hinges. These plastic hinges represent concentrated inelastic behavior achieved by a structural element. Paulay and Priestley (1992) assigned plastic hinges at an equivalent length \( L_p \) of 0.5\( h \), where \( h \) is the height of the cross section of the element. Then, by assuming a constant curvature along this plastic hinge length, the rotation can be computed by multiplying this length and the curvature. The plastic hinge models were computed from the constitutive laws of materials and their hysteretic relationships. Perfect elastoplastic behavior was assumed for the reinforcing steel, and the Hognestad model for unconfined concrete was assigned to the concrete cross sections. These material constitutive laws are shown in Figure 6.

![Figure 6. Perfect elastoplastic behavior for reinforcing steel (left) and Hognestad model for the concrete (right) moment-curvature relationships](image6.png)

Moment-curvature relationships allow the computation of inelastic deformation capacity of a structural element. Typical V2 and C1 sections for the first story of the NFB are shown in Figure 7. The V2 beam is reinforced with 3\( 
\Phi 5/8" \) on the compression side and with 3\( 
\Phi 5/8" + 4\Phi 3/4" \) on the tension side. Also, the column C1 is uniformly reinforced with 4\( 
\Phi 3/4" + 8\Phi 1" \). Figure 8 shows the moment-curvature relationships for the V2 beam and C1 column sections. It is observed that the C1 moment-curvature diagram is highly influenced by the axial load level which is taken from the second load combination of the Peruvian concrete design code E060 (MVCS 2009) (i.e. 1.25 \( DL \) + 1.25 \( LL \)). These loads were considered in the moment curvature relationships for the reinforced concrete cross sections during the NTHA.

![Figure 7. Cross sections of the V2 beam (left) and the C1 column (right) for the 1st story of the NFB.](image7.png)

![Figure 8. Moment-curvature plots for sections V2 (left) and C1 (right) for the 1st story of the NFB.](image8.png)
Hysteresis behavior was defined according to Takeda et al. (1971) model, since it is appropriated for the concrete structural elements detailed for energy dissipation and dominant flexure failure. This Takeda model and the corresponding Moment-Rotation relationship are shown in Figure 9. This model was applied for all the plastic hinges used in the NTHA.

With the previous hypothesis and by means of an algorithm involving SAP2000, a total of 2000 random structural models were analyzed.

![Figure 9. Takeda et al. (1970) model (left) and hysteretic behavior of the C2 column (right)](image)

5. FRAGILITY FUNCTIONS

5.1 Damage Limit States

To define damage states for a building, it is necessary to carry out a static nonlinear analysis which is commonly known as Pushover analysis. This method consists of a series of incremental static nonlinear analysis which results in defining the Capacity Curve of the building. The constitutive models of the materials and the hysteresis model defined in the NTHA are key elements in defining this Capacity Curve. This curve is usually built by using the first mode response (fundamental mode) which is the most significant in regular low-rise buildings. The roof displacement is increased progressively and at the same time the base shear is recorded. This process is repeated until achieving the maximum lateral displacement capacity (Chunque 2013). A simplified trilinear model was constructed according to Aguiar (2003), where areas inside and outside the capacity curves are equivalent. Two load patterns were defined for the development of the Capacity Curves. One pattern considers user coefficients and the other pattern is proportional to the first mode. Both can be easily defined in SAP2000 and Figure 10 shows Capacity Curves for both NFB and EEFB, and their corresponding Limit States.

![Figure 10. Capacity Curves for NFB (left) and EEFB (right)](image)

The SEAOC (1995) in the document VISION 2000 proposed Limit States based on the elastic range and the inelastic range between the yielding point and the ultimate displacement in the Capacity...
Curve, as shown in Table 4. Each performance level is related to a range of roof displacement. These limits were applied in this research to define the Damage Limit States. These are shown in Table 5.

Table 4. Displacement range related to performance levels (SEAOC 1995)

<table>
<thead>
<tr>
<th>Performance level</th>
<th>Displacement range</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully Operational</td>
<td>ΔYP</td>
<td>ΔYP: Elastic displacement. Range between zero displacement and yielding point (Y.P.).</td>
</tr>
<tr>
<td>Operational</td>
<td>0.30ΔP</td>
<td>ΔP: Inelastic displacement. Plastic range between yielding point (Y.P.) and collapse performance level.</td>
</tr>
<tr>
<td>Life Safety</td>
<td>0.30ΔP</td>
<td></td>
</tr>
<tr>
<td>Collapse Prevention</td>
<td>0.20ΔP</td>
<td></td>
</tr>
<tr>
<td>Collapse</td>
<td>0.20ΔP</td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Damage Limit States for the NFB (left) and for the EEFB (right) in terms of the interstory drift ratio

<table>
<thead>
<tr>
<th>Damage Limit State</th>
<th>Nurse Faculty Building (NFB)</th>
<th>Electronic Engineering Faculty Building (EEFB)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Interstory Drift ratio</td>
<td>Roof Displacement (m)</td>
</tr>
<tr>
<td>LS1: Fully Operational</td>
<td>0.53%</td>
<td>0.049</td>
</tr>
<tr>
<td>LS2: Operational</td>
<td>0.89%</td>
<td>0.078</td>
</tr>
<tr>
<td>LS3: Life Safety</td>
<td>1.23%</td>
<td>0.107</td>
</tr>
<tr>
<td>LS4: Collapse Prevention</td>
<td>1.48%</td>
<td>0.127</td>
</tr>
<tr>
<td>LS5: Collapse</td>
<td>1.71%</td>
<td>0.146</td>
</tr>
<tr>
<td>LS1: Fully Operational</td>
<td>0.53%</td>
<td>0.049</td>
</tr>
</tbody>
</table>

5.2 Damage Probability Density Functions

The probability density function of a random continuous variable is defined as \( f(x) \). This function can be integrated in the interval \([a, b]\) to represent the probability of occurrence for real values between \( a \) and \( b \). This is shown in Equation 7.

\[
P[a \leq x \leq b] = \int_a^b f(x) \, dx
\]  

(7)

If Equation 7 is integrated between \(-\infty\) and \( x \), the resulting function is known as the cumulative probability density function \( F_x(x) \) (CDF). This can be computed numerically by sorting the results in ascending order and applying Equation 8 (Velásquez 2006) as follows

\[
F_x(x_i) = \frac{i}{n(x)}
\]

(8)

where \( x_i \) is the event that may repeat \( i \) times inside a population with sample size \( n(x) \). Also, if we compute the CDF over the entire real domain, Equation 9 is obtained.

\[
\int_{-\infty}^{\infty} f(x) \, dx = F_x(\infty) = \lim_{x \to +\infty} F(x) = 1
\]

(9)

This means that the probability of being less than infinity is 1. Also, the probability of the complementary event defined as the Probability of Exceedance can be computed by Equation 10.

\[
P[X > x] = 1 - F_x(x)
\]

(10)

This Probability of Exceedance represent the limit value of the damage states and can be used to plot the fragility functions.
The 2000 models generated by the simulation process represent numerical probability density functions. This simulation was implemented with several scripts written in MATLAB (Mathworks Inc. 2015). Each part of the simulation was performed with scripts concerning: the definition of the input data related to the geometry and properties of the buildings, parameters related to sampling for each PGA using Latin Hypercube, parameters for the NTHA by using the interaction between MATLAB and SAP2000, parameters related to PDF and CDF generation. Finally, the CDFs for the maximum interstory drift ratio are shown in Figure 11 for several values of the Intensity Measure (IM). In this case the peak ground acceleration (PGA) is defined as the IM ranging for values up to 1.00g.

Figure 11. CDFs for NFB (left) and EEFB (right)

5.3 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 (Rojas 2010). Here, the IM was defined in terms of the PGA and the pseudo-acceleration ($S_a$). Generally, all methods for generating fragility functions adjust values into a lognormal probability density functions such as the one shown in Equation 11.

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

(11)

where $\bar{IM}$ is the mean value of IM for the damage limit state DLS$_i$. The $\beta_{IM,DLS_i}$ is the standard deviation for the DLS$_i$, and $\phi$ is the lognormal cumulative probability density function. Using an analytical method implemented in MATLAB scripts, fragility functions were developed for the university buildings as shown in Figure 12. Four limit states were used in these plots. LS5 was neglected because it represents the upper bound for the collapse of the structure. Beyond this point there is no additional damage, so the collapse is defined between LS4 and LS5.

Figure 12. Simulated Fragility Functions for NFB (left) and EEFB (right) in terms of PGA.

Loss ratios can be computed by choosing seismic intensity scenarios. This procedure consists in calculating the probability of exceedance for all damage limit states given a PGA scenario. This results
in a Mean Damage Factor (MDF) (Hwang and Lin 2002). This value represents the mean loss ratio which is also defined as the ratio between the repairing and the total replacement cost for a given building (Velásquez 2006). Only structural damage is considered. For the seismic intensity obtained by the PSHA for Cusco city (0.22g), which is amplified by the soil type coefficient $S$ defined as 1.2 by the Peruvian code. This coefficient represents local soil conditions wherein buildings are located. This leads to a PGA of 0.26g. Figure 11 shows 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 6. In this Table, the 0.45g-PGA scenario is also included, because it represents the Peruvian zone with the highest seismicity.

### Table 6. Mean Damage Factor (MDF) for 0.26g and 0.45g of PGA for both NFB and EEFB.

<table>
<thead>
<tr>
<th>Damage State</th>
<th>NFB</th>
<th>EEFB</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.26g-PGA</td>
<td>8.80%</td>
<td>50.02%</td>
</tr>
<tr>
<td>0.45g-PGA</td>
<td>19.05%</td>
<td>57.05%</td>
</tr>
</tbody>
</table>

These MDFs allow to estimate repairing costs in every building by multiplying these factors by the total construction area and by a unitary cost related to the structural replacement cost. Table 7 shows these calculations considering a unitary cost of USD 250/m².

### Table 7. Estimated repairing cost for NFB and EEFB

<table>
<thead>
<tr>
<th>Faculty</th>
<th>MDF</th>
<th>Area (m²)</th>
<th>Stories</th>
<th>USD/m²</th>
<th>Cost (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NFB</td>
<td>8.80%</td>
<td>527.72</td>
<td>4</td>
<td>250</td>
<td>46 439.36</td>
</tr>
<tr>
<td>EEFB</td>
<td>19.05%</td>
<td>561.35</td>
<td>5</td>
<td>250</td>
<td>141 039.19</td>
</tr>
</tbody>
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<table>
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<tr>
<th>Faculty</th>
<th>MDF</th>
<th>Area (m²)</th>
<th>Stories</th>
<th>USD/m²</th>
<th>Cost (USD)</th>
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</thead>
<tbody>
<tr>
<td>NFB</td>
<td>50.02%</td>
<td>527.72</td>
<td>4</td>
<td>250</td>
<td>383 388.58</td>
</tr>
<tr>
<td>EEFB</td>
<td>57.05%</td>
<td>561.35</td>
<td>5</td>
<td>250</td>
<td>400 312.72</td>
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</tbody>
</table>

Fragility functions were also computed for the NFB en the X direction. For the 0.26g PGA scenario a MDF of 5.80% was obtained. The 0.45g PGA scenario resulted in a MDF of 26.4%. These proves that the Y direction is more critical and that it is not necessary to use a 3D model. Results accurate enough are obtained with a planar model.

### 6. CONCLUSIONS

This paper proposes a rational analytical method for generating fragility functions for Peruvian university buildings. Fragility functions were obtained with simulation. For each 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 8.80% and 19.05% for the Nursing Faculty Building (NFB) and for the Electronic Engineering Faculty Building (EEFB), 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
It is also shown that if these Faculty buildings were built in the highest seismic zone of Peru, i.e. zone 4, the damage ratios would exceed 50%. This scenario would probably require extensive repairing or most likely demolition.

Future research is suggested to compare the use of simulated against real recorded events. The method proposed herein to generate fragility functions is suggested for seismic regions where seismic records are scarce or not available.

7. REFERENCES


