Paper:

Assessment of Seismic Performance of High-Rise Thin RC Wall Buildings in Lima, Peru Using Fragility Functions

Luis G. Quiroz and Yoshihisa Maruyama

Department of Urban Environment Systems, Chiba University
1-33 Yayoi-cho, Inage-ku, Chiba 268-8522, Japan
E-mail: lgquiroz@uni.edu.pe
[Received June 23, 2014; accepted August 19, 2014]

The actual behavior of thin RC wall high-rise buildings during an earthquake in Lima, Peru, and the associated seismic loss is unknown. This type of building was assessed done using analytical fragility functions. The numerical model was based on full-scale tests done in Lima, Peru. Nonlinear dynamic response analysis was performed using records simulated for Lima. The damage ratio was estimated for four damage states and fragility functions were obtained assuming that the damage ratio followed log-normal distributions. Seismic performance was evaluated by considering the probability of different damage states for three seismic hazard levels. It was found that high-rise buildings present a low probability of collapse in severe earthquakes.

Keywords: thin RC wall, high-rise building, fragility functions, seismic performance

1. Introduction

Peru is located in a seismic-prone region. The earthquakes of 1746 and 1868, for instance, shook the territory with an intensity of up to XI on the MMI scale. During the last 145 years, however, it has had earthquakes with a maximum intensity of only IX on the MMI scale, so buildings in Lima have not been subjected to severe ground shaking for over 100 years. It has thus not yet been possible to determine the seismic performance in actual earthquakes of the new structural systems appearing in the last century.

Lima buildings whose main structural components support vertical and lateral loads are thin reinforced concrete (RC) walls that have been constructed since 1998. These walls mainly feature thinness and the use of electrowelded wire mesh as the main reinforcement. This system was used only for low- and mid-rise buildings of a maximum 5 stories. The number of stories gradually increased, e.g., to 10 stories, due to the ease construction. These high-rise buildings have thus not been tested under severe seismic conditions, and their actual behavior still remains unknown. During the Maule earthquake – the last big earthquake – which hit Chile in 2010, some buildings whose vertical and lateral resistance involved thin walls, were severely damaged and, in some cases, collapsed [1].

One approach to assessing the seismic performance of structures, uses fragility functions that express the conditional probability of exceeding a certain damage state for a given intensity. Fragility functions are developed based on analytical, empirical, and combinational approaches together with expert opinion. Studies estimating seismic performance using fragility functions follow a similar flow: the definition of a numerical model, selection of intensity measures, structural responses, damage estimation, statistical analysis, construction of fragility functions, and the evaluation of seismic performance at certain hazard levels. Fragility functions have been widely used to evaluate seismic damage in buildings as discussed in [2].

This study assesses the seismic performance of thin RC wall high-rise buildings in Lima, Peru, using fragility functions. We evaluated the hysteretic response of thin RC walls in a previous study [3]. Multiple degree-of-freedom and macro models representing the overall behavior of RC elements were used to construct numerical model of high-rise buildings. Nonlinear dynamic analyses were done using records simulated for Lima to estimate structural response. Regression analyses between the damage ratio and the intensity measure of ground motion were performed to construct fragility functions. The seismic performance of high-rise buildings was assessed for three hazard levels and also for estimating weighted mean damage.

2. Archetypal Structure and Numerical Model

The main features of mid- and thin RC wall high-rise buildings were defined by Galvez et al. [4] based on the statistical analysis of existing buildings. Typical geometrical features of high-rise buildings include the number of stories, assumed to be ten, the typical height of a story of 2500 mm, five thin RC walls considered in the structural axis, the length of walls at 2700 mm, wall thickness at 120 mm, concrete slab thickness of 120 mm, and concrete slab width of 3,100 mm.

Thin RC walls present two types of reinforcement – edge reinforcement consisting of conventional rebar and main reinforcement using electrowelded wire mesh. The

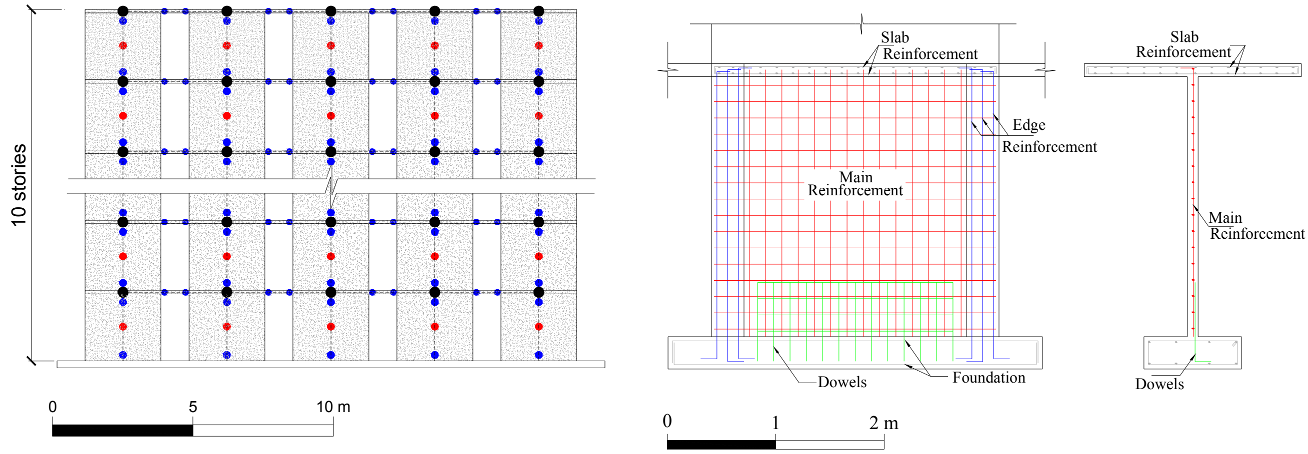

Fig. 1. Numerical model and general characteristics of a thin RC wall.

difference in the two types of reinforcement is that electrowelded wire mesh is made of nonductile material and while conventional rebar is ductile. The strain of conventional reinforcement is 4.5 times larger than that of electrowelded wire mesh. A single layer of main reinforcement is used in both in horizontal and vertical directions. **Fig. 1** shows a general numerical model and wall features.

Masses on each floor are considered to be lumped at wall-slab joints, symbolized by black circles in **Fig. 1**. Blue and red circles indicate locations of nonlinear elements.

Our study considered two archetypal structures. **Table 1** shows the distribution of reinforcement in walls for the two archetypal structures. The main difference between them is the amount of main reinforcement. Walls in archetypal building AB-1 used main reinforcement called QE188, formed by wires of 6 mm in the horizontal and vertical directions spaced at 150 mm. In walls of archetypal building AB-2, main reinforcement consists of mesh QE257, formed by wires of 7 mm in the horizontal and vertical directions spaced at 150 mm. Model AB-2 is considered representative of thin RC wall high-rise buildings constructed in Lima before 2004. The two numerical models were used to assess the impact of reducing main reinforcement on the seismic performance of the buildings.

Our study did not consider the randomness of structural features. Uncertainty in structural element capacity was reduced by selecting material strength based on experiments [4, 5] and appropriate inelastic models. The compression strength of concrete was set at 17.16 MPa. In conventional reinforcement, yield stress was set at 450 MPa with an associated strain of 0.002. For electrowelded wire mesh, yield strain was 0.0035 with a yield stress of 485 MPa.

To predict the hysteretic curve of walls, nonlinear material behavior should be modeled numerically. In concrete, unconfined concrete was assumed. The Kent and Park model was considered in this study [6] but the tensile strength of concrete was neglected. The ultimate strain was set at 0.0035 and other parameters were estimated

Table 1. Distribution of reinforcement in the two models.

Archetype building	Main reinforcement in walls	ρ_h and ρ_v	Edge Reinforcement
AB-1	QE188	0.188%	3 #4[a]
AB-2	QE257	0.257%	

[a] #4: corrugated bars with a diameter of 12.7 mm

using Kent and Park expressions [6]. For reinforcement, the uniaxial behavior of conventional reinforcement and electrowelded wire mesh were modeled using a trilinear model. Behavior was considered to be the same for compressive and tensile stress.

The numerical model represents the effects of the nonlinearity of walls considering concentrated springs idealized by a trilinear backbone curve and hysteretic rules. The bearing features of a cross-section are given through the moment-curvature relationship. The three-parametric model proposed by Park et al. [7], based on a trilinear curve, was adopted. The four parameters α, β_1, β_2, and γwere estimated in the previous study [3].

Archetypal structures were assumed to have been constructed in the firm soil that predominates in Lima. The mass of structures was estimated considering gravity loads distributed as dead load due to a concrete slab weight of 2870 N/m^2, nonstructural partitions of 1000 N/m^2, and floor finishing of 1000 N/m^2. Total dead weight was 4870 N/m^2 and the self-weight of the structure. The live load was considered to be 2500 N/m^2 for all stories except for the top, where the live load was 1000 N/m^2.

From eigenvalue analysis, the natural periods of the structure associated with 1-5 modes were 0.582, 0.150, 0.067, 0.039, and 0.026 s, with mass participation factors of 71.75%, 14.32%, 6.07%, 3.30%, and 1.97%. The natural vibration period seemed reasonable for thin RC wall high-rise buildings in Lima and are similar to periods presented by Galvez et al. [4].

3. Damage Index and Damage State

The selection of a parameter that defines structural damage and different damage states is important when constructing fragility functions. Several approaches have been proposed for defining damage indices. Interstory drift (θ) is usually used to show different damage states because damage is related to local deformation. Interstory drift is calculated as the ratio between the relative displacement of a story and the height of the story. Many drift limits are defined for interstory drift for walls, as discussed in [8].

In our study, maximum interstory drift ($\theta_{\max}$) among all stories was considered as the damage index and the definition of damage states by Ghobarah [9] was used because interstory drift associated with damage states was close to that observed during experiments [5]. $\theta_{\max}$ is sensitive to higher modes of vibration that first mode. **Table 2** defines the damage states considered in this study.

Table 2. Definition of damage states with respect to the interstory drift proposed by Ghobarah (2004).

Damage state	$\theta_{\max}$ (%)
No damage (ND)	0.0 – 0.1
Light (L)	0.1 – 0.2
Moderate (M)	0.2 – 0.4
Severe (S)	0.4 – 0.8
Collapse (C)	>0.8

Table 3. Seismic indices of the simulated records for Lima.

Parameter	Units	Range of values
Peak ground acceleration (PGA)	(cm/s^2)	[288.14–847.70]
Peak ground velocity (PGV)	(cm/s)	[14.55–101.92]
Acceleration RMS	(cm/s^2)	[32.64–82.10]
Velocity RMS	(cm/s)	[2.36–9.41]
Arias Intensity (AI)	(m/s)	[4.09–25.90]
Acceleration Spectrum Intensity (ASI)	(cm/s)	[220.29–860.25]
Velocity Spectrum Intensity (VSI)	(cm)	[48.12–492.36]
Period	(s)	[0.06–0.34]

4. Ground Motion Records

One way to overcome uncertainty related to ground motion considers records that reflect the seismicity of a specific location. Unfortunately, the number of ground motion records for evaluating structural performance is scarce in Lima. Hence, we used ten simulated ground motion records for Lima developed by Pulido et al. [10]. Pulido et al. [10] generated simulated records estimating slip scenarios for a future megathrust earthquake based on an interseismic coupling model at the megathrust and information on historical earthquakes. Simulated records are related to the seismic potential of an earthquake with a moment magnitude of 8.9. Each record has two horizontal components and one vertical component. Horizontal components of acceleration records were applied to the numerical model. **Table 3** lists the seismic indices of input ground motion records.

Figure 2 shows the location of sites where simulated records were obtained and acceleration response spectra for ten simulated records – two horizontal component per record – that are normalized to have a PGA of 1 g, with a damping ratio of 5%. The mean amplitude is shown in **Fig. 2** by a thick blue line. The thick red line represents the design acceleration response spectrum defined by Peruvian seismic design standard E.030 [11].

5. Construction of Fragility Functions

The IDARC2D program [12] was used to calculate the structural response of the archetypal buildings. IDARC2D is a macroelement program that has been extensively validated against the laboratory testing of structural systems and components types. IDARC2D is used for inelastic static and dynamic response analysis of RC structures.

Nonlinear dynamic analysis was done considering a combination of the Newmark-Beta integration method and the pseudoforce method. Values for the time increment step, damping value, and damping type were set at 0.005 s, 5%, and Rayleigh damping.

Different ground motion indices are used to construct fragility functions, e.g., PGA, PGV, AI, Sa (T1, 5%), duration time, and MMI. For our study, PGA was used as the ground motion index because the seismic hazard expected in Lima is based on this parameter.

To construct fragility functions, PGA values for all records were scaled to have different excitation levels. This means that the PGA for records was scaled from 25 cm/s^2 to three times its original PGA with an interval of 25 cm/s^2. The limitation in scaling of a record was set following the recommendations of Bommer and Acevedo [13]. Scaled records were applied to the numerical model to obtain the damage index (maximum interstory drift). Using the damage index, the number of occurrences for each damage state was estimated under individual excitation levels. The damage ratio was then obtained for each damage state. Based on these results, fragility functions for buildings were constructed assuming a log-normal distribution.

Cumulative probability P_R of the occurrence of damage equal or greater than a damage state is given by Eq. (1),

$$P_R = \Phi\left[\frac{\ln Y - \lambda}{\zeta}\right] \quad \ldots\ldots\ldots\ldots \quad (1)$$

where Φ is standard cumulative normal distribution, Y is the ground motion index (PGA), and λ and ζ are the mean and standard deviation of $\ln Y$. These statistical parameters of distribution are obtained by the least-squares method on log-normal probability paper.

Figure 3 shows the number of occurrences of each

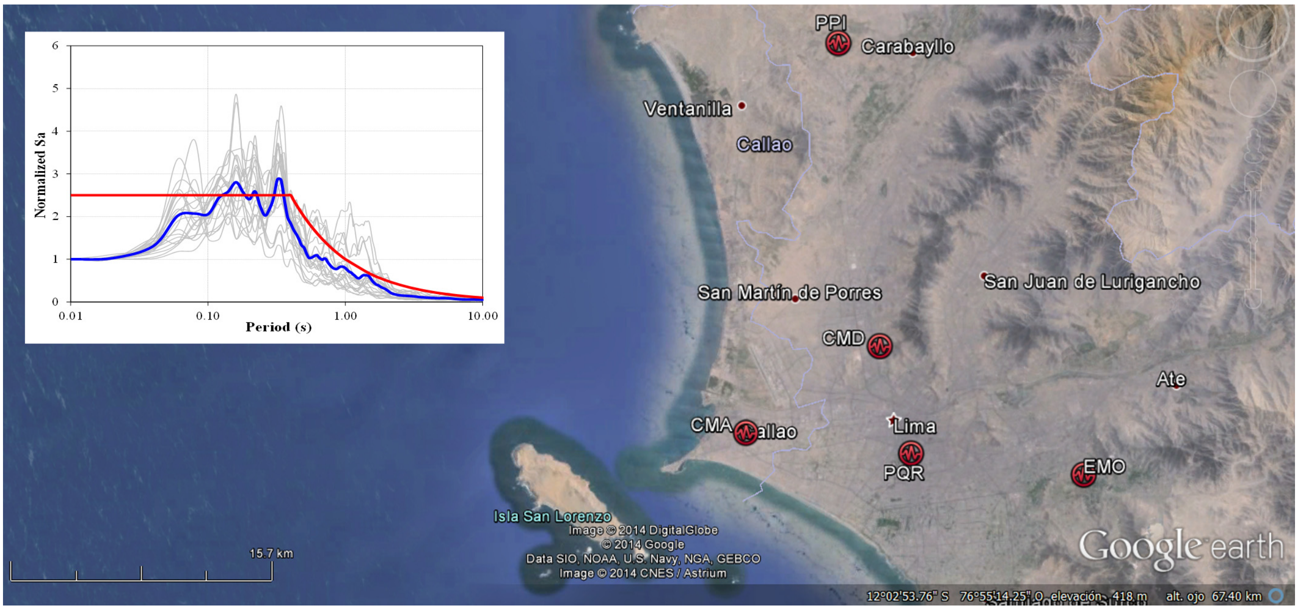

Fig. 2. Locations of simulated records and acceleration response spectra (normalized to have the PGA of 1 g) with the damping ratio of 5% for the simulated records (twenty time-histories).

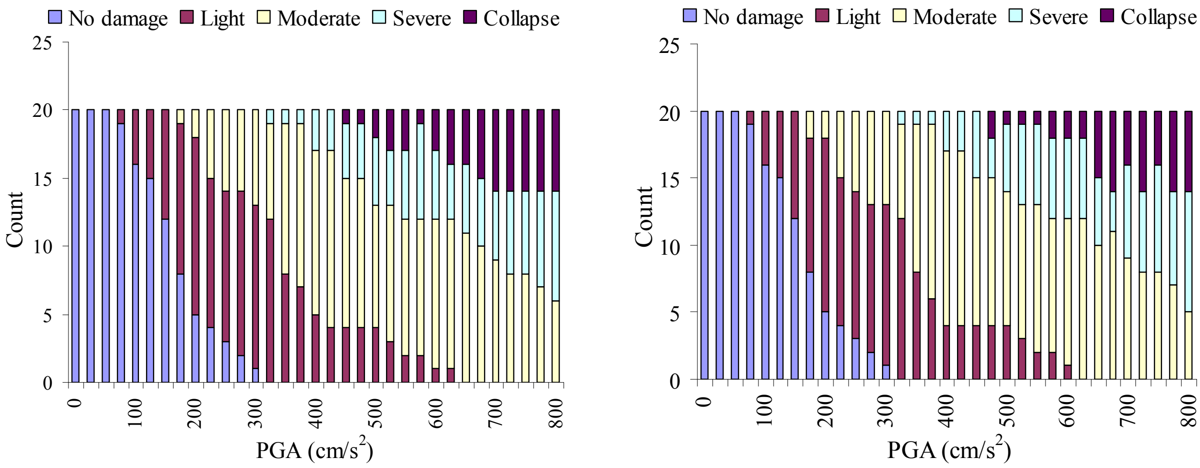

Fig. 3. Number of occurrences of each damage state under the simulated records for the buildings AB-1 (left) and AB-2 (right).

Table 4. Parameters of fragility functions for the Peruvian high-rise buildings with thin RC walls.

Archetype building	Damage state							
	DS > Light		DS > Moderate		DS > Severe		DS = Collapse	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
AB-1	5.048	0.434	5.792	0.405	6.481	0.422	6.712	0.377
AB-2	5.048	0.434	5.768	0.427	6.481	0.414	6.779	0.422

damage state at different excitation levels for archetypal buildings AB-1 and AB-2. **Table 4** lists statistical parameters for fragility functions. The fragility functions obtained for archetypal buildings AB-1 and AB-2 are shown in **Fig. 4**. Note that the reduction in the amount of main reinforcement caused few differences in fragility functions for three damage states – light, moderate and severe. The probability of collapse increases slightly when the amount of main reinforcement is reduced and the increment is higher under larger PGA values.

6. Assessment of Seismic Performance

Peru has no regulations for selecting seismic risk scenarios used in assessing seismic performance, which is why we chose seismicity represented by seismic hazard levels suggested by the SEAOC [14]: occasional earthquake – 50% of exceedance in 50 years, rare earthquake – 10% of exceedance in 50 years, and very rare earthquake – 5% of exceedance in 50 years. Peak ground acceleration values for the three seismicity levels for Lima were

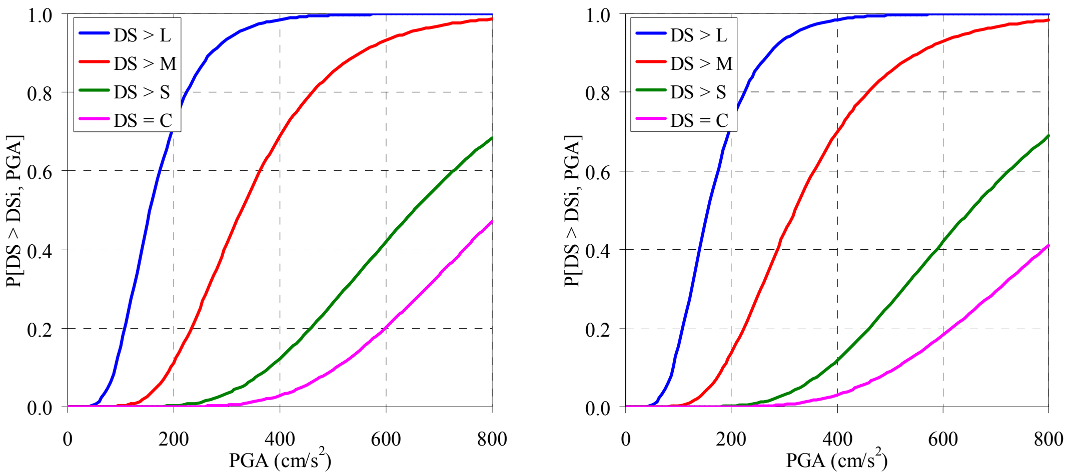

Fig. 4. Comparison of the fragility functions for buildings AB-1 (left) and AB-2 (right).

Table 5. Probabilities of each damage state for the Peruvian high-rise buildings with respect to the three seismic hazard levels.

DamageState	AB-1			AB-2		
	0.2g	0.4g	0.5g	0.2g	0.4g	0.5g
No Damage	29.7%	1.7%	0.4%	29.7%	1.7%	0.41%
Light Damage	60.0%	31.1%	15.5%	57.7%	30.0%	15.4%
Moderate Damage	10.1%	55.9%	59.2%	12.5%	57.4%	59.6%
Severe Damage	0.21%	8.9%	16.3%	0.17%	8.2%	16.2%
Collapse	0.01%	2.5%	8.6%	0.02%	2.8%	8.3%

estimated by Velasquez [15] as 0.2 g, 0.4 g, and 0.5 g.

The probabilities of each damage state at each specific hazard level were estimated based on the fragility functions presented in this study. **Table 5** compares buildings AB-1 and AB-2. Note that in occasional earthquakes, the probability is approximately 30% with no damage and 60% with light damage for both buildings. For a rare earthquake, which corresponds to the seismic intensity considered in the design code, both buildings present a 1.6% of probability with no damage, and the probability of light and moderate damage is approximately 87% for both building. A 2.65% probability for collapse is estimated on the average for the two archetypal buildings. In a very rare earthquake, the probability of light, moderate, or severe damage states is about 91% for both buildings. The slight increment in probability resulted from the decrease in main reinforcement.

Weighted mean damage state D_m [16] was calculated by Eq. (2),

$$D_m = \left(\frac{1}{4}\right) \sum_{i=0}^{4} DS_i P[DS_i] \quad \ldots\ldots\ldots \quad (2)$$

where DS_i takes values 0, 1, 2, 3, and 4 for damage states i considered in analysis and $P[DS_i]$ is the corresponding probability. Note that D_m is close to the most likely damage state of a structure. **Fig. 5** shows discrete values for D_m for three levels of intensity 0.2 g, 0.4 g, and 0.5 g.

Note that a structure may not be reparable if mean damage exceeds 60% [17]. It was found that buildings would

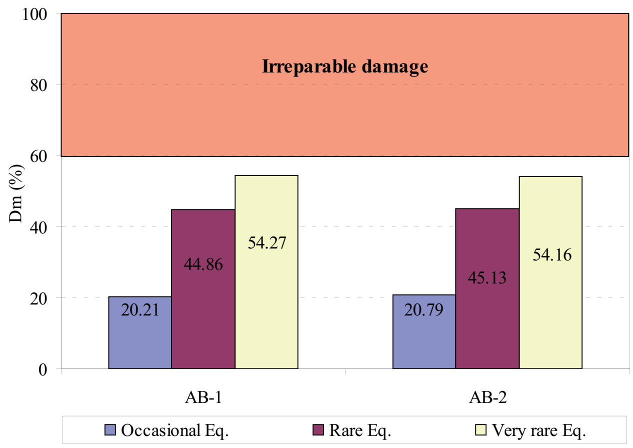

Fig. 5. Weighted mean damage for buildings AB-1 and AB-2 under the three levels of ground motion intensity (0.2 g, 0.4 g, and 0.5 g).

suffer mean damage of 20% in an occasional earthquake and 45% on the average in a rare earthquake. Hence, we concluded that walls of high-rise buildings would suffer reparable damage.

7. Conclusions

Fragility functions for thin RC wall high-rise buildings in Lima, Peru, have been developed and they were used to evaluate seismic performance. The following conclusions are drawn:

Archetypal buildings AB-1 and AB-2 were analyzed considering variations in the amount of electrowelded wire mesh as main reinforcement in high-rise buildings. The probability of light, moderate, and severe damage is similar for both buildings. The probability of collapse increases slightly when the amount of main reinforcement is reduced. Buildings behave in light and no damage under the occasional earthquake. In rare earthquake, buildings behave in moderate, light and no damage. The probability of collapse is around 2.6% for both buildings.

The estimation of weighted mean damage revealed that both buildings present a value of 45% for a rare earthquake. Considering ATC-21 (2001), the estimation of weighted mean damage shows that both buildings would show reparable damage.

Results show that the use of electrowelded wire mesh in walls for high-rise buildings produces a low probability of collapse, and weighted mean damage is acceptable even for a rare earthquake.

References:

[1] EERI (2010), "The Mw 8.8 Chile Earthquake of February 27, 2010," Special Earthquake Report – June, 2010.

[2] E. D. Jovanoska, "Fragility curves for reinforced concrete structures in Skopje (Macedonia) region," Soil Dynamic and Earthquake Engineering, Vol.19, No.6, pp. 455-466, 2000.

[3] L. Quiroz, Y. Maruyama, and C. Zavala, "Cyclic behavior of thin RC Peruvian shear walls: Full-scale experimental investigation and numerical simulation," Engineering Structures, Vol.52, pp. 153-167, 2013.

[4] V. Gálvez, M. Burgos, and A. Ortiz, "Proposal of reduction factor for seismic forces in structural systems composed by walls reinforced with electro-welded wire mesh and ductile bars," Lima, Peru, 2008 (in Spanish).

[5] C. Zavala, "Test on walls and a one-floor house reinforced with electro-wire mesh UNICO/FORSA/PRODAC, Research report," Japan-Peru Center for Earthquake Engineering Research and Disaster Mitigation. Lima, Peru, 2004 (in Spanish).

[6] D. C. Kent and R. R. Park, "Flexural members with confined concrete," Journal of the Structural Division, ASCE, Vol.97, No.7, pp. 1969-1990, 1971.

[7] Y. J. Park, A. M. Reinhorn, and S. K. Kunnath, "IDARC Inelastic Damage Analysis of Reinforced Concrete Frame-Shear-Wall Structures," Technical Report NCEER-87-0008, State University of New York at Buffalo, 1987.

[8] J. Carrillo and S. M. Alcocer, "Acceptance limits for performance-based seismic design of RC walls for low-rise housing," Earthquake Engineering and Structural Dynamics, Vol.41, No.15, pp. 2273-2288, 2012.

[9] A. Ghobarah, "On drift limits associated with different damage levels," Proceedings of an International Workshop – Performance-Based Seismic Design Concepts and Implementation, Bled, Slovenia, 2004.

[10] N. Pulido, "Earthquake rupture and slip scenarios for Central Andes Peru, and strong motion simulations for Lima (v1.0 February 7/2013)," 2013, available at http://www.j-shis.bosai.go.jp/staff/nelson/smsimu_Central_Peru_v1p0/Lima_SMsimu_v1p0.htm [accessed July 1, 2014]

[11] Ministry of Housing Peru, "National Technical Standard E-030 – Earthquake Resistant Design," Lima, Peru, 2003 (in Spanish).

[12] A. M. Reinhorn, H. Roh, M. Sivaselvan, S. K. Kunnath, R. E. Valles, A. Madan, C. Li, R. Lobo, and Y. J. Park, "IDARC2D Version 7.0: A Program for the Inelastic Damage Analysis of Structures," Technical Report MCEER-09-0006, State University of New York at Buffalo, 2009.

[13] J. J. Bommer and A. B. Acevedo, "The use of real earthquake accelerograms as input to dynamic analysis," Journal Earthquake Engineering, Vol.8, No.4, No.1-50, 2004.

[14] Structural Engineers Association of California (SEAOC), "Recommended Lateral Forces Requirements and Commentary (the Blue Book)," California, 1999.

[15] J. Velasquez, "Estimation of earthquake losses in Peruvian buildings by analytical fragility curves," Master Thesis, Graduate School, Catholic University of Peru PUCP, Lima, 2006 (in Spanish).

[16] A. H. Barbat, M. L. Carreño, L. G. Pujades, N. Lantada, O. D. Cardona, and M. C. Marulanda, "Seismic vulnerability and risk evaluation methods for urban areas. A review with application to a pilot area," Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance, Vol.6, Nos.1-2, pp. 17-38, 2010.

[17] Applied Technology Council ATC–21 (2001), "Rapid Visual Screening of Seismically Hazardous Buildings," Applied Technology Council, Redwood City, CA.

Name:
Luis G. Quiroz

Affiliation:
Graduate Student, Department of Urban Environment Systems, Chiba University

Address:
1-33, Yayoi-cho, Inage-ku, Chiba 268-8522, Japan

Brief Career:
2000 B.S. Civil Engineer, Faculty of Civil Engineering, National University of Engineering, Lima, Peru
2002-2004 Master student, Faculty of Civil Engineering, National University of Engineering, Lima, Peru
2004-2011 Research Assistant, Japan-Peru Center for Earthquake Engineering Research and Disaster Mitigation (CISMID)
2005-2011 Assistant professor, Faculty of Civil Engineering, National University of Engineering, Lima, Peru
2011- present Graduate Student, Chiba University

Selected Publications:
• "Cyclic behavior of thin RC Peruvian shear walls: Full-scale experimental investigation and numerical simulation," Engineering Structures, Vol.52, pp. 153-167, July, 2013.
• "Assessment of the Applicability of Drucker-Prager Model with Cap to Adobe," Proceedings of the 15th World Conference on Earthquake Engineering, Paper No.0677, p. 10, September 2012.

Academic Societies & Scientific Organizations:
• Peru Engineering Association (CIP)