Assessment of performance of Peruvian high-rise thin RC wall buildings using numerical fragility functions

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ABSTRACT

The real behavior of high-rise thin RC wall buildings during an earthquake located in Lima, Peru and their associated seismic losses are still unknown. The assessment of seismic performance of thin RC wall buildings is carried out using analytical fragility functions. The numerical model is based on full-scale test carried out in Lima, Peru. Nonlinear dynamic response analysis is performed using simulated records for Lima. The damage ratios were estimated with respect to four damage states, and the fragility functions were obtained assuming that the damage ratios follow lognormal distributions. The seismic performance is evaluated by considering the probability of being in various damage states at three seismic hazard levels and a weighted mean damage. It was found that high-rise buildings would present a low probability of collapse under the severe earthquake, and a reparable damage.

1. INTRODUCTION

Peru is located in a seismic-prone region. In its long seismic history, the earthquakes of 1746 and 1868 shook the territory with intensities up to XI (MMI). However, in the last 145 years, we have only had earthquakes with maximum intensities of IX (MMI). Hence, the buildings in Lima, Peru have not been subjected to severe earthquakes for more than 100 years, and it is not yet possible to determine the seismic performance of new structural systems that appeared in the last century.

In Lima, Peru, the buildings whose main structural components to support the vertical and lateral loads are thin reinforced concrete (RC) walls have been constructed since 1998. The main characteristics of these walls are the use of electro-welded wire mesh as main reinforcement and its thin thickness. At the beginning, the system was used only for low- and mid-rise buildings (maximum 5 stories), but the number of stories gradually increased (e.g. 10 stories) due to less construction labor. These high-

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High-rise buildings have not been tested under severe seismic conditions, and their real behavior is still unknown. During the last big earthquake that hit Chile in 2010 (Maule earthquake), some buildings, whose vertical and lateral resistance systems were thin walls, suffered from severe damage and in some cases collapsed (EERI 2010).

An approach for the seismic assessment of structures makes use of fragility functions. Fragility functions express the conditional probability of exceeding a certain damage state for a given intensity. The fragility functions can be developed based on analytical, empirical, expert's opinion and combinational approaches. Every study of estimation of seismic performance using fragility functions follows a similar flowchart: definition of numerical model, selection of intensity measures, structural responses, damage estimation, statistical analysis, construction of fragility functions and evaluation of seismic performance at certain hazard levels. Fragility functions have been widely used to evaluate the seismic damage in buildings e.g. Jovanoska (2000).

This study aimed to assess the seismic performance of high-rise thin RC wall buildings located in Lima, Peru using fragility functions. The hysteretic response of the thin RC walls was evaluated in our previous study (Quiroz et al. 2013). Multi-degree-of-freedom system and macro models to represent the overall behavior of the RC elements were used to construct the numerical model of high-rise buildings. Non-linear dynamic analyses were carried out using simulated records for Lima in order to estimate the structural response. Regression analyses between the damage ratios and the intensity measure of the ground motion were performed to construct the fragility functions. Finally, the seismic performance of the high-rise building is assessed for three hazard levels and also estimating a weighted mean damage.

2. ARCHETYPE STRUCTURE AND NUMERICAL MODEL

The main characteristics of mid- and high-rise thin RC wall buildings were defined by Galvez et al. (2008) based on statistical analysis of existing buildings. The typical geometrical characteristics of high-rise buildings were defined as follows: the number of stories considered was ten, the typical height of story is 2500 mm, five thin RC walls are considered in the structural axis, the length of the walls is 2700 mm, the thickness of the walls is 120 mm, the thickness of the concrete slab is 120 mm, and its width is 3100 mm.

The walls presented two types of reinforcement. The edge reinforcement consisted of conventional rebar. In case of main reinforcement, electro-welded wire mesh is used. The difference in the two type of reinforcement is that electro-welded wire mesh is made of non-ductile material, while conventional rebar is made of ductile material. The strain of conventional reinforcement is 4.5 times larger than that of electro-welded wire mesh. A single layer of main reinforcement is used in both directions. Figure 1 shows a general view of the numerical model and general characteristics of a wall.

The masses in each floor are considered to be lumped at the wall-slab joints and they are symbolized as black circles in Fig. 1. The blue and red circles indicate the locations of nonlinearity of the elements.
Table 1. Distribution of reinforcement in the two models

<table>
<thead>
<tr>
<th>Archetype building</th>
<th>Main reinforcement in walls</th>
<th>$\rho_h$ and $\rho_v$</th>
<th>Edge Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB-1</td>
<td>QE188</td>
<td>0.188%</td>
<td>3 #4&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>AB-2</td>
<td>QE257</td>
<td>0.257%</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup> #4: corrugated bars with a diameter of 12.7 mm

For the present study, two archetype structures were considered for high-rise buildings. Table 1 shows the distribution of reinforcement in the walls for the two archetype structures. The main difference is the amount of main reinforcement. The walls on archetype building AB-1 presents main reinforcement called QE188, which is formed by wires of 6 mm in the horizontal and vertical directions spaced at 150 mm. In case of walls of archetype building AB-2, the main reinforcement consists of mesh QE257, which is formed by wires of 7 mm in the horizontal and vertical directions spaced at 150 mm.

In the present study, the randomness of the structural characteristics has not been considered. The uncertainty in the capacity of the structural element was reduced by selecting material strengths based on the experiments (Zavala 2004; Galvez et al. 2008) and appropriate inelastic models. The compression strength of concrete was set to be 17.16 MPa. In case of the conventional reinforcement, the yielding stress was set to be 450 MPa with an associated strain of 0.002. As for the electro-welded wire mesh, the yield strain was 0.0035 with a yield stress of approximately 485 MPa.

To predict the hysteretic curve of the walls, the nonlinear behaviors of materials should be modeled numerically. In case of concrete, unconfined concrete is assumed. The Kent and Park model was considered in this study (Kent and Park 1971). The tensile strength of concrete was neglected. The ultimate strain was set to be 0.0035, and the other parameters have been estimated using the expressions of Kent and Park (1971). For reinforcement, the uniaxial behavior of conventional reinforcement and
electro-welded wire mesh is modeled by the trilinear model. The behavior is considered to be the same for compressive and tensile stresses.

The numerical model represents the effects of non-linearity of walls considering the concentrated springs idealized by a trilinear backbone curve and hysteretic rules. The bearing characteristics of a cross-section are given through the moment-curvature relationship. The three-parametric model proposed by Park et al. (1987), which is based on a tri-linear curve, was adopted. The three parameters $\alpha$, $\beta$, and $\gamma$ were estimated in the previous study (Quiroz et al. 2013).

It is assumed that the prototype structures are founded in firm soil, which is predominant in Lima city. The estimation of the masses of the structures was done considering that gravity loads are distributed dead load from concrete slab weight (2870 N/m$^2$), non-structural partitions (1000 N/m$^2$), and floor finishing (1000 N/m$^2$). The total dead weight is 4870 N/m$^2$ plus the structure selfweight. The live load was considered as 2500N/m$^2$ for all stories except for the top were the live load is 1000 N/m$^2$.

From the eigenvalue analysis, the natural periods of the structure in the 1-5 mode 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%, respectively. The natural vibration period seemed reasonable for high-rise thin RC wall buildings.

3. DAMAGE INDEX AND DAMAGE STATES

The selection of a parameter that defines the structural damage and different damage states is important to construct fragility functions. Several approaches have been proposed to define damage indices, e.g. one of those approaches considers three categories: non-cumulative, cumulative and combined damage indices. The structural parameters related to the categories mentioned before are the maximum deformation, hysteretic behavior, and deformation/energy absorption. The first category previously mentioned has the advantage of simplicity in estimation process. Typical structural responses related to non-cumulative category are interstory drifts and displacement ductility ratios.

The interstory drift ($\theta$) is usually used to show different damage states because the damage is related to local deformations. The 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 (e.g. Carrillo and Alcocer 2012).

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

<table>
<thead>
<tr>
<th>Damage state</th>
<th>$\theta_{\text{max}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No damage (ND)</td>
<td>0.0 – 0.1</td>
</tr>
<tr>
<td>Light (L)</td>
<td>0.1 – 0.2</td>
</tr>
<tr>
<td>Moderate (M)</td>
<td>0.2 – 0.4</td>
</tr>
<tr>
<td>Severe (S)</td>
<td>0.4 – 0.8</td>
</tr>
<tr>
<td>Collapse (C)</td>
<td>$&gt;$ 0.8</td>
</tr>
</tbody>
</table>
In the present study, the maximum interstory drift ($\theta_{\text{max}}$) among all stories is considered as damage index and the definition of damage states by Ghobarah (2004) was employed because the interstory drifts associated with the damage states are close to those observed during the experiments (Zavala 2004). Also, $\theta_{\text{max}}$ is sensitive to higher modes of vibration. Table 2 shows the definition of damage states considered in this study.

4. GROUND MOTION RECORDS

A way to overcome the uncertainty related to the ground motions is considering various records that reflect the seismicity of a specific place. Unfortunately, the number of ground motion records to evaluate structural performance is scarce in case of Lima. Hence, in the present study, ten simulated records for Lima city developed by Pulido (2013) were employed. Pulido (2013) generated the simulated records estimating the slip scenarios for a future megathrust earthquake based on an interseismic coupling model at the megathrust as well as information of historical earthquakes. The simulated records are related to a seismic potential of an earthquake with moment magnitude of 8.9. Each record has two horizontal components and one vertical component. The horizontal components of the acceleration records are applied to the numerical model. Table 3 presents the seismic indices of the input ground motion records. Figure 2 shows the acceleration response spectra for the simulated records, which are normalized to have the PGA of 1g, with the damping ratio of 5%. The mean amplitude is also shown in Fig. 2, illustrated with a thick blue line. The thick red line represents the design acceleration response spectrum defined by the Peruvian seismic design standard E.030 (Ministry of Housing 2003).
Table 3. Seismic indices of the simulated records for Lima

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

5. CONSTRUCTION OF FRAGILITY FUNCTIONS

IDARC2D program (Reinhorn et al. 2009) was used to calculate the structural response of the archetype buildings. IDARC2D is a macro-element program that has been extensively validated against laboratory testing of structural systems and components types, and it is used for the inelastic static and dynamic response analysis of RC structures.

The non-linear dynamic analysis is carried out considering a combination of the Newmark-Beta integration method and the pseudo-force method. The values for time increment step, damping value and damping type are set to be 0.005 s, 5% and Rayleigh damping, respectively.

Different ground motion indices can be used to construct the fragility functions, e.g. PGA, PGV, AI, Sa(T$_1$, 5%), duration time, MMI, etc. Some of them can be obtained directly from the records (PGA), and the others need a processing of the record (PGV, AI, Sa(T$_1$, 5%)). For the present study, PGA was used as the ground motion index because the seismic hazard expected in Lima is based on that parameter.

To construct the fragility functions, the necessary steps are as follows: the values of PGA for all records were scaled to have different excitation levels. Hence, the PGA for the records was scaled from 25 cm/s$^2$ to three times of its original PGA with the interval of 25 cm/s$^2$. The limitation in scaling of a record is set following the recommendations of Bommer and Acevedo (2004). The scaled records were applied to the numerical model to obtain the damage index (maximum interstory drift). Using the damage index, the number of occurrence for each damage state was estimated under each excitation level. Then, the damage ratio was obtained for every damage state. Finally, based on these results, fragility functions for the buildings are constructed assuming a lognormal distribution.

The cumulative probability $P_R$ of occurrence of the damage equal or higher than a damage state is given by Eq. (1)

$$P_R = \Phi \left[ \frac{\ln Y - \lambda}{\varsigma} \right]$$

(1)
Figure 3 shows the number of occurrences of each damage state under different excitation levels for the archetype buildings AB-1 and AB-2. Table 4 shows the statistical parameters of fragility functions. The fragility functions obtained for the archetype buildings AB-1 and AB-2 are presented in Figure 4. As can be observed, the reduction of the amount of main reinforcement caused few differences of the fragility functions for the three damage states (light, moderate and severe). The probability of collapse slightly increases when the amount of main reinforcement is reduced, and the increment is higher under larger PGA values.

6. ASSESSMENT OF SEISMIC PERFORMANCE

For the evaluation of seismic risk scenarios, the seismicity represented by seismic hazard levels suggested by the SEAOC (1999) was chosen: 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). The peak ground
acceleration values for the three seismicity levels are 0.2 g, 0.4 g and 0.5 g, respectively.

Based on the fragility functions presented in this study, the probabilities of each damage state at each specific hazard level were estimated. Table 5 shows the comparison for the buildings AB-1 and AB-2. It is observed that in case of occasional earthquake, 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, the probability of light and moderate damage is approximately 87% for both building. Approximately 2.65% probability for collapse is estimated in average for the archetype buildings. In case of a very rare earthquake, the probability of light, moderate and severe damage states is about 91% for both buildings. The slight increment of the probability is resulted from the decrease of the main reinforcement.

Finally, a weighted mean damage state $D_m$ (Barbat et al. 2010) was calculated by Eq. (2)

$$D_m = \left( 1 \over 4 \right) \sum_{i=0}^{4} DS_i \cdot P[DS_i]$$

(2)

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

Table 5. Comparison of probability of each damage state for the three levels of ground motion intensity

<table>
<thead>
<tr>
<th>Damage State</th>
<th>AB-1</th>
<th>AB-2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.2g</td>
<td>0.4g</td>
</tr>
<tr>
<td>No Damage</td>
<td>29.69%</td>
<td>1.66%</td>
</tr>
<tr>
<td>Light Damage</td>
<td>60.02%</td>
<td>31.12%</td>
</tr>
<tr>
<td>Moderate Damage</td>
<td>10.08%</td>
<td>55.85%</td>
</tr>
<tr>
<td>Severe Damage</td>
<td>0.21%</td>
<td>8.88%</td>
</tr>
<tr>
<td>Collapse</td>
<td>0.01%</td>
<td>2.49%</td>
</tr>
</tbody>
</table>
where $DS_i$ takes the values 0, 1, 2, 3 and 4 for the damage states $i$ considered in the analysis, and $P[DS_i]$ are the corresponding probabilities. It can be considered that $D_m$ is close to the most likely damage state of a structure. Figure 5 shows the discrete values for $D_m$ for the three levels of intensity (0.2 g, 0.4 g, and 0.5 g).

It can be considered that a structure may be irreparable in case the mean damage is more than 60% (ATC-21 2001). It was found that the buildings would suffer from a mean damage of 20% in case of occasional earthquake and 45% in average in case of rare earthquake. Hence, it is interpreted that the walls of high-rise buildings will suffer a reparable damage.

7. CONCLUSIONS

In the present study, the fragility functions for high-rise thin RC wall buildings constructed in Lima, Peru were developed, and they were used to evaluate the seismic performance. The following conclusions can be drawn:

The archetype buildings AB-1 and AB-2 were analyzed to consider the variation in the amount of electro-welded wire mesh as main reinforcement in high-rise buildings. The probabilities of light, moderate, and severe damage are similar for both buildings. The probability of collapse slightly increases when the amount of main reinforcement is reduced. The buildings behave in light and no damage under the occasional earthquake (almost 90%). In case of the rare earthquake, the buildings behave in moderate, light and no damage (almost 90%). The probability of collapse is around 2.6% for both buildings.

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

The results show that the use of electrowelded wire mesh in walls for high-rise buildings produce a low probability of collapse and a weighted mean damage acceptable for the rare earthquake.
REFERENCES


Structural Engineers Association of California (SEAOC) (1999), “Recommended Lateral Forces Requirements and Commentary (the Blue Book),” California