FRAGILITY FUNCTIONS FOR THIN-WALLED REINFORCED CONCRETE DWELLINGS IN PERU

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ABSTRACT

Peru is constantly subjected to severe earthquakes due to its close location to the Ring of Fire. Analysis of past earthquakes allowed the improvement of the seismic design code. The growth of population is increasing the housing demand in Peru. Thin-walled reinforced concrete dwellings are preferred by builders due to their short execution time and low construction costs. These walls have very limited ductility due to the properties of the one-layer steel arrangement. There is a 50-year seismic gap in the northern region of Peru, where most of these structures are built. Hence, these dwellings have not been subjected to strong accelerations yet. Therefore, we cannot predict their performance. Some tests conducted under the provisions of FEMA 461 were carried out to assess the mechanical properties of natural scale walls.

This paper focuses on the development of fragility curves for thin-walled buildings. These curves are generated by a Monte Carlo simulation technique and for 2 types of buildings. Type 1 consists of three 2-story blocks which share dividing walls and Type 2 considers a 3-story independent block. These are the most common layouts offered by the housing market since the 90s. These dwellings are usually built on an intermediate between soft soil and rock. This probabilistic approach considers randomness and uncertainty of properties such as the compressive concrete strength and the yield stress of the steel. This is applied to the seismic demand through the generation of synthetic signals. Both types of structures show excellent performance for the design earthquake.

Keywords: Thin-walled Buildings, Fragility Curves, Concrete Dwellings, Monte Carlo simulation.

1. INTRODUCTION

Thin-walled reinforced concrete dwellings have been built in Peru very commonly since the 90s because of its ease of construction and the short execution times. Also, because reinforced concrete is sufficiently stiff with its thin walls the rooms in the buildings are larger than those made with conventional frame or masonry structures. However, many problems have been reported during the construction process such as the congestion of pipelines, the scarce concrete cover in the walls, the cutting of reinforcement to allow pipelines installation and the repairing of damaged walls during the construction. Therefore, it is mandatory to predict its performance during next important seismic event.

This paper shows a probabilistic methodology for the development of fragility curves in single-family dwellings. The structural systems are composed by reinforced concrete thin walls. Two types of building layouts were assessed. The type 1 layout considers a 2-story building composed by three attached blocks and the type 2 layout consists of a single 3-story independent block, as shown in Figure 1. The fragility curves were generated analytically through simulation considering uncertainties in the seismic demand and the structural parameters. To decrease the uncertainty in the nonlinear dynamic analysis, the mechanical properties of the shear walls were calibrated from data obtained by

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tests at the laboratory of the Pontifical Catholic University of Peru (PUCP).

![Plan layouts of (a) type 1: 2-story 3-block building and (b) 3-story 1-block building.](image)

**Figure 1.** Plan layouts of (a) type 1: 2-story 3-block building and (b) 3-story 1-block building.

### 2. DUCTILITY-LIMITED SHEAR WALL BUILDINGS (DLSWB) IN PERU

The lack of housing in Peru and the growth of population are the reasons for permanent housing demand, especially for poor people with the lowest salary income. Thin-walled reinforced concrete dwellings are mostly preferred by contractors and builders because these facilities are very productive in terms of execution time, simplification of construction procedures and reduction of construction costs. The limited ductility is one of the special features for these shear walls for which its design and construction are not entirely predicted among the international seismic design codes. For this reason, there is still a disagreement about its expected seismic performance. A typical building of this type is shown in Figure 2.

![Typical Ductility-Limited Shear Wall Building (DLSWB) in Peru.](image)

**Figure 2.** Typical Ductility-Limited Shear Wall Building (DLSWB) in Peru.

The DLSWB consist of load-carrying shear walls with a thickness of 10 to 12 cm for gravity and seismic actions. Generally, beams are nonexistent, and the walls directly carry the loads from the concrete slabs. A typical reinforcement arrangement is shown in Figure 3.

![Typical reinforcement arrangement of a DLSWB.](image)

### 3. PRELIMINARY RESULTS FROM THE WALL TESTS

#### 3.1 Description of the Tests

At the laboratory of the PUCP, a total of 9 tests were carried out for natural scale ductility-limited shear walls (2.00 x 0.10 x 2.40 m) with conventional reinforcement with the purpose of studying their seismic performance. These tests were developed under the guidelines of FEMA 461. Walls were
tested under lateral cyclic loading as follows: a) 3 walls with an axial load of 300 kN, and b) 3 walls without axial load, until the failure of the element related to a maximum displacement $\Delta_u$. And, c) 3 walls were tested with cyclic loading until reaching a controlled level of damage related to a repairable interstory drift of 0.50%. The geometry of these walls is shown in Figure 4.

![Figure 3. Typical reinforcement of the tested walls (diameter in inches and spacing in meters).](image)

![Figure 4. Dimensions (in m) and features of the tested walls.](image)

### 3.2 Definition of the Analytical Model

An accurate representation of the cyclic behavior of the shear walls is a complex task, due to critical parameters such as the loss of strength, degradation of stiffness and most importantly the dissipated energy during the loading and unloading hysteretic loops.

The specified compressive concrete strength was 17 MPa and it has been idealized with the Hognestad model (1951) which represents accurately the behavior of unconfined concrete. A model of fibers based on the geometry and loading conditions of the wall has been used. The fiber model consists of 5 concrete fibers and 11 steel fibers.

The macro element utilized here corresponds to the General Wall-Compound Component defined in the program PERFORM-3D (2006). This element involves all the effects shown in Figure 5. The vertical fibers are used to model flexure and axial effects. The shear concrete fibers take into account the concrete contribution to shear strength and the diagonal compression fibers are able to transmit...
shear forces and considers the contribution of the horizontal reinforcement to shear strength by means of the interaction with the vertical fibers.

The shear behavior is modeled with the trilinear relationship defined by Gérin and Adebar (2004).

The geometry of the model has been defined to match the geometry of the test. Also, the wall has been meshed throughout its height, so the estimation of displacements was accurate enough. The walls were defined by Shell elements of four nodes with inelastic in-plane behavior and linearly elastic out-of-plane behavior. Once the numerical model was defined, the same load pattern from the real test was applied. From these results, the constitutive properties of materials were adjusted until reaching a behavior close enough to the real test as shown in Figure 6.

4. ESTIMATION OF THE SEISMIC VULNERABILITY OF DLSWB

4.1 Monte Carlo Simulation

The Monte Carlo method is a simulation technique that estimates the response of stochastic processes. The responses are estimated from the probability distribution functions of input random variables. To study the fragility of the building, a group of samples from variables that represent seismic and structural parameters were generated. The values for each parameter were chosen in a random way from its probability distribution functions. A large number of runs is required to obtain reliable and consistent results. However, the sample size was reduced by means of the Latin Hypercube Sampling technique (Florian 1992). This technique reduces sampling to 100 structural models that reasonably and accurately represent uncertainties in the seismic and structural parameters.
4.2 Seismic and Structural Parameters

Spectrum-compatible seismic signals were generated. These synthetic signals were generated for a Seismic Intensity Parameter (SIP). The SIP selected is the Peak Ground Acceleration (PGA) for stiff soils since the seismic risk in the entire Peruvian territory is defined for this parameter. Even though previous research has shown that the PGA has great dispersion and that it is not entirely correlated with nonlinear displacements, it might be assumed as valid for short period structures. The dwellings analyzed in this research have very short fundamental periods. Hence, the use of the PGA as the SIP does not represent an issue for the reliability of the results.

Synthetic signals were obtained randomly with the program SIMQKE (Vanmarke and Gasparini 1976), considering that their response spectrum is compatible with the elastic design spectrum of the Seismic Peruvian Code. Typical synthetic signals are shown in Figure 7. Also, their corresponding response spectra are shown in Figure 8. It should be noticed that the response spectra generated for these synthetic signals match reasonably well the elastic design spectrum of the Seismic Peruvian Code along the short period range.

Figure 7. Synthetic signals for PGA of 0.45g on rigid soil.

Figure 8. Response spectra generated by SIMQKE for synthetic signals for PGA of 0.45g.
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 obtained. This might be due to the frequency content of these real earthquakes. Also, not enough seismic records are available in this region. This is why synthetic signal were preferred.

The structural parameters considered for the DLSWB are the compressive strength of the concrete $f'_c$ and the yield stress of the reinforcing steel $f_y$, as shown in Figure 9. According to standard tests, $f_y$ is better adjusted to a normal distribution. A mean value of 480 MPa and a standard deviation of 5.5% were used for $f_y$ of 420 MPa (Gonzáles 2005). It is also typical to use a normal distribution for the concrete. In this case, for a concrete with $f'_c$ of 17.5 MPa, a mean value of 22.5 MPa and a standard deviation of 15% were used.

![Figure 9. Probability distribution functions for the compressive strength of the concrete (Left) and the yield stress of the reinforcing steel (Right).](image)

### 4.3 Damage States

The damage states were defined under the guidelines of FEMA 356 (2000) and ASCE 41-06 (2007), and were complemented with experimental results from previous research that reflect the characteristics of the construction process, the quality of the employed manpower and the typical wall dimensions. Table 1 shows the approach defined by Gonzáles (2005), who gathers the results obtained by Medina (2005), wherein 3 limit values are defined for each reference parameter to determine 4 damage states. Since structural elements are sensitive to the relative interstory displacement, in this method the use of the maximum interstory drift ratio $\delta_{max}$ was used as the Damage Parameter (DP).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strain in coupling beams</td>
<td>0.005</td>
<td>0.010</td>
<td>0.020</td>
</tr>
<tr>
<td>Rotation of the plastic hinge region</td>
<td>0.002</td>
<td>0.004</td>
<td>0.008</td>
</tr>
<tr>
<td>located at the base of the walls (rad)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum interstory drift ratio</td>
<td>0.25%</td>
<td>0.375%</td>
<td>0.50%</td>
</tr>
</tbody>
</table>

IO: Immediate Occupancy    LS: Life Safety    CP: Collapse Prevention

### 4.4 Nonlinear Dynamic Analysis

A widely accepted method to reduce uncertainty in a structural model is the calibration of the properties of the structural elements with tests. This was performed by cyclic tests of the main elements of the structure. The program parameters that best represent the model of the representative elements were replicated in the analytical model. The model of the structure was considered as a group
of these calibrated elements, properly modelled in the tests. Therefore, the responses obtained against the seismic demand should be acceptable.

The structural model considers shell elements with distributed nonlinearity. The model also considers the stiffness degradation and the strength loss for the elements of the structure. The constitutive model of Hognestad was used for unconfined concrete, the model of Kent and Park (1971) was used for the reinforcing steel, and the model of Gérin and Adeb ́ar (2004) was used for the shear behavior. The simulated models were subjected to synthetic signals that represented the seismic demand. To perform the nonlinear analysis, the program PERFORMD-3D was used. The maximum interstory drift ratio was computed for each sample model analyzed. A typical time-history response for the maximum interstory drift ratio is shown in Figure 10.

![Figure 10. Sample of time-history response of the maximum interstory drift ratio computed by PERFORMD-3D for building Type 2.](image)

5. DEVELOPMENT OF FRAGILITY CURVES

One of the most used methods to represent the performance of a structure is the fragility curves analysis. These curves represent the probability of exceedance for a structural response, given a parameter that defines the ground motion intensity. In other words, these curves are a measure of the seismic fragility in probabilistic terms. The fragility curves are generated by methods such as field survey, expert opinions, and with analytic simulation methods (Bonnet 2003). When field data is scarce, fragility curves are built analytically with simulation (Hwang and Huo 1994).

The analytical procedure required to generate fragility curves considers a very large number of structural models subjected to a wide range of levels of seismic demand. These fragility curves can be obtained from a statistical approach for the seismic demand and the damage states represented by a damage parameter (DP). These results are first obtained by performing dynamic nonlinear analysis, and then generating cumulative distribution functions for the DP. Finally, from these cumulative functions the vulnerability is represented in the fragility curves (Bonnet 2003). Details of the whole procedure is shown in Figure 11.

5.1 Damage Cumulative Distribution Function

The Cumulative Distribution Functions (CDF) of the DP are retrieved for each level of seismic intensity from the results obtained in the simulation process. Probability distribution functions of DP are retrieved for each level of PGA, which is the Seismic Intensity Parameter (SIP). Figure 12 shows an example for the maximum interstory drift ratio as the DP of a Ductility-Limited Shear Wall Building of Type 2. These functions have been adjusted using log-normal polynomials (Hong and Lind 1996).
5.2 Probability of Exceedance

Using the CDFs of Figure 12 and the limit values defined by the discrete damage states from Table 1, the fragility curves can be obtained. These curves relate the SIP (i.e. PGA or pseudo-acceleration $S_a$) for a given damage state (Bonnet 2003). Fragility curves for both Types of buildings are shown in Figures 13 and 14, respectively.

![Methodology for the development of fragility curves](image)

Figure 11. Methodology for the development of fragility curves.

![Cumulative Distribution Function (CDF) of the maximum interstory drift ratio for Type 2 building (single 3-story block).](image)

Figure 12. Cumulative Distribution Function (CDF) of the maximum interstory drift ratio for Type 2 building (single 3-story block).
5.3 Expected Performance of DLSWB

In the structural design, a force reduction factor of 4 was employed for the Type 1 building (2-story 3-block) due to the lack of irregularities. The base shear design force was 822 kN. In the weak direction, the shear strength of the wall in the first floor was 3236 KN, which means that a strength of 3.9 times the base shear design force was used. This value is close to the reduction factor of 4. Therefore, in the case of a design earthquake, the structure will behave elastically and should not exhibit any damage. This statement is related to the fragility curves found here. For this building, a pseudo-acceleration of 1.13g (0.45gx2.5=1.125g) is considered, and the probability that the structure exhibits any type of damage is insignificant. For a pseudo-acceleration of 3.75g (PGA of 1.5g related to 0.5% probability of exceedance in 50 years) there is a probability of 2% for Severe Damage, 75% for Moderate Damage, 18% for Slight Damage and a 3% for No Damage at all.

For the structural design of Type 2 building (3-story 1-block), due to the soft story and corner irregularity, a force reduction factor of 2.7 was used. This value was used with the purpose of compensating for irregularities. However, the final building strength is overcompensated. The base
shear design force was 331 kN. In the weak direction the strength of walls in the first floor was 840 kN, which means that a strength of 2.5 times the base shear design force was provided. This value is close to the reduction factor selected. Therefore, the structure should exhibit an elastic behavior in the case of a design earthquake without attaining any type of damage. This is related to fragility curves found here, since in this type of building, a pseudo-acceleration of 1.125g shows a probability of 1% for slight damage. For a PGA value of 0.8g (related to an extreme event with a probability of exceedance of 2% in 50 years) there is a pseudo-acceleration of 2.0g and the fragility curves show a probability of 0.5% for moderate damage, 69.5% for Slight Damage and 30% for No Damage. For pseudo-accelerations of 3.75g (PGA of 1.5g related to 0.5% of probability of exceedance in 50 years), the probability that a Severe Damage occurs is 10%, for Moderate Damage is 89% and for Slight Damage is 1%.

6. CONCLUSIONS

This paper showed that it is possible to model with enough precision the behavior of reinforced concrete thin walls with the use of numerical fiber models, the constitutive laws of material and the shear behavior law. For a probability of exceedance of 50%, fragility curves for Type 1 building (2-story 3-block) show a very wide range of pseudo-accelerations between Slight Damage and Extensive Damage (1.9g to 4.6g). This is due to the fact that these buildings have only 2 stories and the 3 blocks attached to each other might cause the delay the stiffness degradation and strength loss due to the assembly of the building. These buildings are considered as “flat buildings”. The fragility curves for the Type 2 building (3-story 1-block), for a probability of exceedance of 50%, show a smaller range of pseudo-accelerations between Slight Damage and Extensive Damage (3.2g to 4.4g). These walls are slender because the ratio between height and length is around 2. This might be the reason of the delay of the flexure failure. However, once this failure begins stiffness degradation and strength loss rapidly accelerates.

In summary, it is expected that both types of buildings will show excellent performance during the design earthquake. Future research is recommended for slender ratios greater than 2 in order to determine the failure mode and the related strength.

7. REFERENCES

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