PERFORMANCE-BASED ASSESMENT OF REINFORCED CONCRETE FRAMES WITH UNREINFORCED MASONRY INFILL WALLS

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Abstract

Reinforced Concrete (RC) frames with unreinforced masonry (URM) infill walls are widely used in earthquake resistant construction in seismically active regions around the world. The URM infill walls are primarily used as elements that delimit spaces within the building structures. These heavy and rigid elements typically interact with the more ductile RC frames during an earthquake in an undesired manner. Observations from past earthquakes that took place in Turkey, Italy, Ecuador, and Mexico, amongst other countries, have shown that many structures of this type often behave poorly and even completely or partially collapse.

A new seismic assessment trend has been motivated by observations after past earthquakes where even when many collapses do not actually occur and the life safety objective of traditional design philosophies are indeed met, the socio-economic losses due to damage, financial losses, and downtime have been significantly high and unacceptable to the society. To overcome the shortcomings of the traditional code design philosophy, probabilistic methods that consider different sources of uncertainties and their propagation are gaining popularity. The Pacific Earthquake Engineering Research (PEER) Center developed a Performance Based Earthquake Engineering (PBEE) methodology to assess the performance of structures subject to earthquakes in terms of decision variables of direct interest to stakeholders and decision makers, instead of the usual engineering parameters. The PEER PBEE methodology decomposes the problem into four analyses: hazard, structural, damage and loss where uncertainties in all four stages are explicitly considered [1].

The present study focuses on applying the performance-based seismic assessment approach to an archetype RC frame building with URM infill walls, designed according to the Colombian seismic code without considering the infill walls in the design. The selection of this structural typology is motivated by its widespread presence in many countries around the world, especially in those with developing economies and because of its lack of representation under the applications of the current PEER-PBEE methodology. The present study highlights the benefits of going beyond the traditional design philosophy while pointing out the limitations in the application of the methodology applied to this structural typology. Furthermore, some ideas to overcome these limitations are herein presented.

Keywords: *Performance-Based Earthquake Engineering; Reinforced Concrete frame; Unreinforced Masonry (URM) Infill Walls.*

1. Introduction

Colombia is one of the many seismically active countries where Reinforced Concrete (RC) frames with Unreinforced Masonry (URM) infill walls are still widely used as a lateral and gravity force resisting system. Observations from previous earthquakes have shown that this type of structures presents moderate to severe damage under seismic loading. Such poor performance can be attributed to the fact that the interaction between the URM infill walls and the surrounding RC frames is a very complex problem that has not been yet fully understood, due to its dependency on many variables such as: constitutive theory of the used materials (brick and mortar mechanical properties), the masonry unit geometry, labor quality, the stiffness ratio of the frame and the infills, the presence of openings in the wall, aspect ratio of the panels, frame/wall interface conditions, In-Plane/Out-of-Plane interaction, etc. [2-4]. Furthermore, it is common practice to only include the effects of the masonry infills as dead loads and as part of the mass of the structure, but these are rarely considered as elements that contribute to the lateral resisting system of the structure, which leads to an inconsistency between the mathematical models and the real structures.

As previously stated, recent seismic events have demonstrated the poor structural behavior of RC frames with URM infill walls. The consequences of this go beyond structural damage and usually lead to injuries, fatalities, downtime and economic losses. In fact, the engineering community and society have realized that the socio-economic losses of structures were unacceptably high, even for structures that were designed to comply with the latest building codes based on traditional design philosophy [5]. Herein, traditional design philosophy, refers to the prescriptive and often deterministic methods mandated by building codes to prevent structural and non-structural elements of buildings from any damage in low-intensity earthquakes, limit the damage in these elements to repairable levels in medium-intensity earthquakes, and prevent the overall or partial collapses of buildings in high-intensity earthquakes [1].

To overcome the shortcomings of such traditional design procedures, the Pacific Earthquake Engineering Research (PEER) Center developed a robust methodology to assess the performance of structures subject to earthquakes in terms performance measures of interest of stakeholders instead of engineering parameters. The framework usually referred to as PEER Performance Based Earthquake Engineering (PBEE) methodology decomposes the problem into four different analyses: hazard, structural, damage and loss. Furthermore, uncertainties in all four stages are explicitly considered [1]. Since its origins in the early 2000s, the PEER PBEE methodology has been widely used and validated by the research community in many applications. Unfortunately, the studies using the PEER PBEE methodology in structures comprised of RC frames with URM infill walls are limited or usually don't cover the full extent of the framework.

This article presents a performance-based application for the seismic assessment of a 4-story archetype RC frame building with URM infill walls, designed according to the latest Colombian design provisions: Reglamento Colombiano de Construccion Sismo Resistente, NSR-10 [6]. Concluding remarks are presented.

2. PEER-PBEE Methodology Overview

The PEER-PBEE methodology allows to assess the expected performance of a structure in terms of variables that are of direct interest to the different stakeholders [1, 7]. This differs from the traditional design philology, which outcome is usually engineer demand parameters such as forces, displacement, drift, etc. Another key aspect of the framework is the explicit consideration of all the sources of uncertainties in a rigorous probabilistic manner. Furthermore, the methodology breaks down the problem into four steps, as explained in [8]:

1. Hazard Analysis estimates the seismic hazard using probabilistic methods such as Probabilistic Seismic Hazard Analysis (PSHA). PSHA considers the epistemic and aleatory uncertainties in many variables like: nearby faults, rate of events, source to site distance, soil conditions, etc. [1]. Hazard analysis is performed to estimate the mean annual rate of exceedance of an Intensity Measure (IM), e.g.: Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) or the Spectral Acceleration at

a given period T_0 of interest ($S_a(T_0)$). Furthermore, PSHA is also used to select ground motions that are consistent with the hazard to be run in the following analysis.

2. Structural Analysis is performed to estimate the structural response of a building subjected to suites of ground motions previously selected to represent a wide spectrum of hazard levels in a probabilistic manner. The outcome of this stage is to determine the probability of exceedance (POE) of one or more Engineering Demand Parameters (EDP) given different IM levels, i.e.: $POE(EDP > edp|IM = im)$. Among the most common EDPs are Interstory Drift Ratio (IDR), floor acceleration, element forces or deformations, etc. In this stage probability of collapse $p(C|IM)$ and non-collapse $p(NC|IM)$ are also obtained for each hazard level.

3. Damage Analysis is performed to determine physical damage in the different structural and non-structural components. Components are usually grouped in what are known as damageable groups that are affected by the same EDP in a similar manner. Damage Measures (DM) need to be defined for each damageable group. The outcome of this analysis is the determination of the POE of a given DM for different levels of EDPs, i.e.: $POE(DM > dm|EDP = edp)$, such relations are often referred in the literature as fragility functions.

4. Loss Analysis is performed to convert the results obtained in the previous stage into Decision Variables (DVs) that are of direct interest to the stakeholders. Common DVs used are number of fatalities, injures, economic losses and downtime. The outcome of this analysis is the determination of the POE a certain DV for different DMs i.e.: $POE(DV > dv|DM = dm)$.

After all analyses have been completed, they are combined in a consistent manner using the total probability theorem to obtain a Loss curve, which is used to determine the POE of the DV of interest for the problem. The previous can be done according to Eq. (1).

$$\lambda(DV) = \iiint G\langle DV|DM\rangle\, dG\langle DM|EDP\rangle\, dG\langle EDP|IM\rangle\, d\lambda(IM) \tag{1}$$

3. Case Study: 4-Story Colombian Archetype Building

3.1 Elastic Analysis and Code-Based Design

A 4-story RC building with special moment frames and URM infills on its perimeter frames, with 4 moment resisting frames along both the longitudinal and transverse direction was design to comply with NSR-10 [6], which are the current regulations in Colombia for seismic design of structures. The building was assumed to be in Armenia, a city of high seismicity within Colombia. Furthermore, the building was assumed to be located on a soil class D. A response-spectrum analysis was performed using the software ETABS [9], to obtain the demands on the structural elements to then design the structure.

The typical story height is 2.8 m, with exception of the first floor which was 3.2 m, to consider the local practice of using first floor of building as commercial or parking spaces. The vertical elements of the moment frames are connected through a two-way beam supported slab of 0.15 m in thickness, which is considered in the design process as a rigid diaphragm. The beams span 6.0 m and 5.0 m, in the longitudinal and transverse directions, respectively. The assigned response modification factor was $R = 7.0$.

All the beam cross sections were 45 cm × 45 cm with No. 3 square hoops as transverse steel reinforcement spaced every 9.5 cm at the end sections and 19.0 cm everywhere else. Similarly, columns were 55 cm × 55 cm with No. 4 square hoops as transverse reinforcement spaced every 10 cm everywhere along their height. During the design stage, concrete was assumed normal weight with compressive concrete strength (f'_c) as 28 MPa and 35 MPa for the floor systems and columns, respectively. The concrete elastic modulus was computed as 4700 $\sqrt{f'_c}$, as per the code. All the steel reinforcement used in the design was assumed

Grade 60 with yielding strength (f_y) of 420 MPa and an elastic modulus (E_S) of 200,000 MPa . The analysis included reduced stiffness of the structural elements due to cracked sections. The infill walls were not considered as structural elements in the design process. Fig. 1a-c present different views of the idealized structure, the beams and columns reinforcement detailing can be found in Fig. 1d.

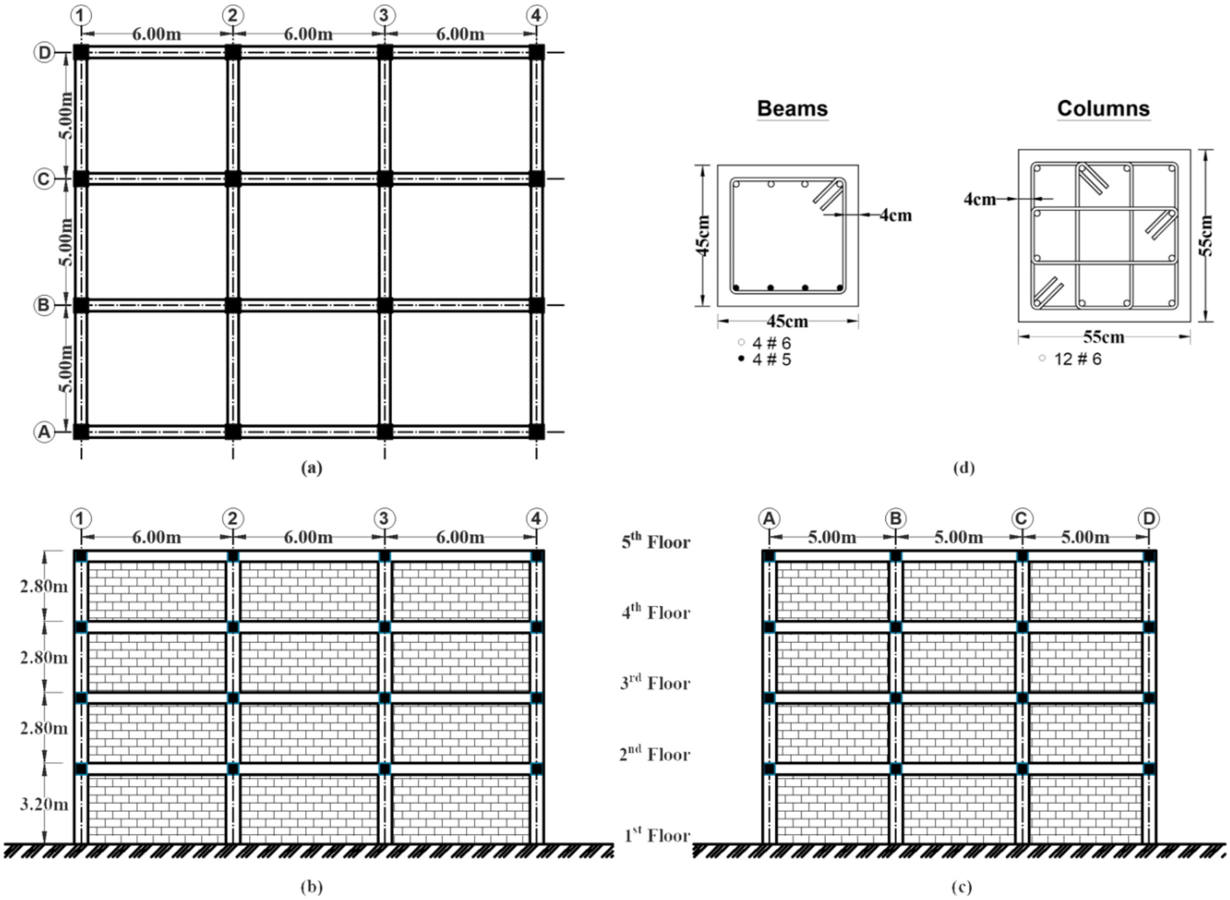


Fig. 1 – (a) Structural plan view. (b) Elevation view of one longitudinal perimeter frame. (c) Elevation view of one transverse perimeter frame (d) Beams and columns reinforcement detailing.

3.2 Inelastic Model

The behavior of the archetype building under realistic suites of ground motions that represent the hazard at the site was assessed by means of Nonlinear Time History Analyses (NLTH). The numerical model was developed using the software package OpenSEES [10]. In the formulation, Rayleigh damping was assumed, the damping coefficient were stablished to achieve a damping ratio $\xi = 2.5\%$ at periods corresponding with the first and third vibration mode of the model.

Beams and columns were modeled using nonlinear elements with distributed plasticity and fiber sections at the integration points [11]. Second order P-Δ effects were considered in columns. Column shear damage due to forces transferred from the URM infill walls was modeled using bilinear shear springs in OpenSEES by means of zero-length elements. Concrete was modeled as either confined or unconfined, depending on the location within the cross section. Tensile concrete strength was assumed to be $0.62\sqrt{f'_c}$ (MPa), with a strain softening branch adjusted to a fracture energy of 0.1 N/mm [12]. The tensile and compressive behavior of reinforcing steel was modeled using the uniaxial Hysteretic material implemented in OpenSEES.

The URM infill walls are modeled using the formulation previously proposed by [13, 14]. In their formulation, each infill wall is modeled by a single diagonal strut. Each diagonal implements two beam-column elements, these are model in OpenSEES using the force-based elements called "beamWithHinges", their cross section is modeled using fiber sections. The two beam-column elements are connected by an Out-of-Plane (OOP) lumped mass at midpoint of the diagonal strut. Two key aspects were included in the modeling of the infill panels: (i) In-Plane (IP) / Out-of-Plane (OOP) interaction and (ii) implementation of an algorithm to account for the progressive collapse of the infill panels due to seismic actions.

3.3 IP/OOP Interaction

For the case where the URM infill walls are subjected to only IP or only OOP actions, the model will follow the provision given in Section 7.5 of FEMA-356 [15]. The IP/OOP interaction curve is given by Eq. (2) [13]:

$$\left(\frac{\Delta_{IP}}{\Delta_{IP_0}}\right)^{3/2} + \left(\frac{\Delta_{OOP}}{\Delta_{OOP_0}}\right)^{3/2} \leq 1.0 \tag{2}$$

where Δ_{IP} and Δ_{OOP} are the IP and OOP displacement considering interaction between them, respectively. Similarly, Δ_{IP_0} and Δ_{OOP_0} are the IP and OOP displacement capacity without considering interaction between the two actions, i.e.: pure IP or pure OOP behavior.

3.4 Infill Wall Removal

An algorithm of progressive collapse developed by [16] and implemented by [14] in OpenSEES was used to explicitly account for the removal of the collapsed URM infill walls. For the current study, the removal criteria use was the failure curve given by Eq. (2).

4. PBEE Assessment for Archetype Building

4.1 Hazard Analysis

The seismic hazard can be estimated within the PEER-PBEE methodology using Probabilistic Seismic Hazard Analysis (PSHA). According to PSHA, the hazard at a site can be estimated with Eq. (3):

$$Haz(Sa(T) > z) = \sum_{i=1}^{NFLT} N_i(M_{min}) \int_{R=0}^{\infty} \int_{M_{min}}^{M_{max_i}} f_{M_i}(M) f_{R_i}(r)\, P(Sa > z|M, r)\, dM\, dr \tag{3}$$

The left-hand side of the equation represents the rate of earthquakes with $Sa(T) > z$ happening at the site of interest. $N_i(M_{min})$ is the rate of ground motions with magnitudes greater than the minimum magnitude of interest M_{min} at the given fault i. $f_{M_i}(M)$dM and $f_{R_i}(r)dr$ are the relative rate of ground motions of different magnitudes and at different distances occurring at fault i, respectively. Finally, $P(Sa > z|M, r)$ is the probability of a ground motion with $Sa(T) > z$ happening at the site of interest given that an earthquake of magnitude M at a distance R has occurred.

As previously stated, the archetype building is assumed to be in the Armenia, Colombia. Which is a city located in an area of high seismicity action according to the NSR-10 [6]. The NERPH soil class was assumed to be D, which is the predominant soil class classification for Armenia. The PSHA is performed using the toolbox "Seismic Hazard" developed by [17], the seismic source parameters for studied site were manually input into the platform, i.e.: fault file with slip rate, moment rate, recurrence interval, etc. per source.

Uniform Hazard Spectra (UHS) are shown in Fig. 2 for 10 hazard levels or Return Periods (TR). These UHS along with the de-aggregation of the hazard are then used to select scaled ground motions from the Colombian Geological Survey (Servicio Geologico Colombiano) database, that are consistent with the hazard at site of interest, by means of running a Conditional Scenario Spectra (CSS). The CSS are a set of realistic ground motions with assigned rates of occurrence that reproduce the hazard at a site over a range of hazard levels and periods [8]. It is worth mention that the due lack of a large set of consistent ground motions, not many them have been selected per hazard level. In the near future, more records might become available and can be easily included into the framework.

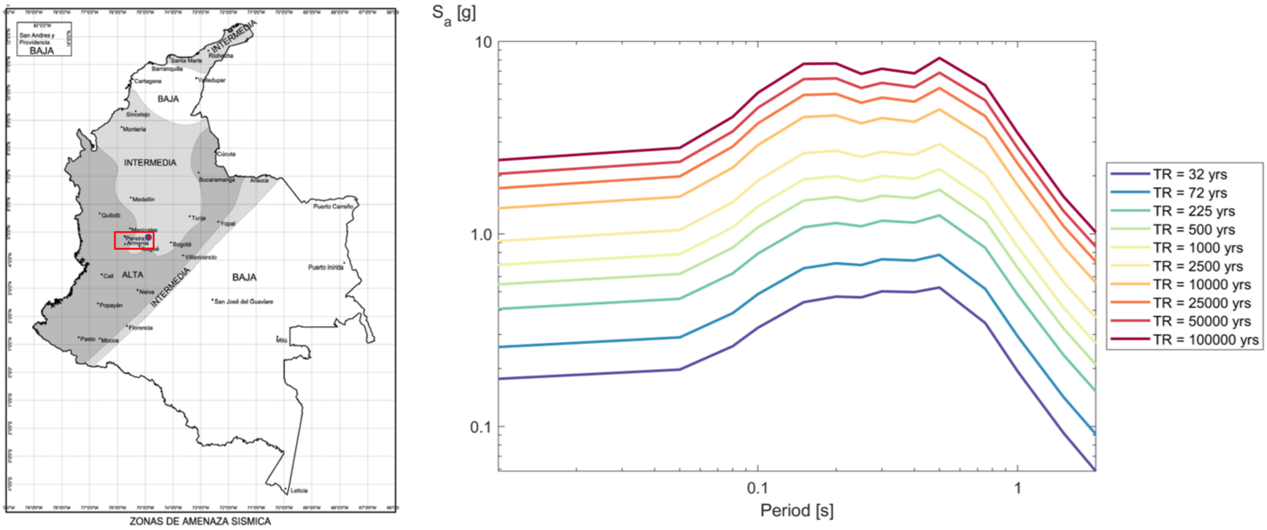


Fig. 2 – Uniform hazard spectra (5% damped).

4.2 Structural Analysis

Nonlinear time history analyses are performed using OpenSEES [10] for the all the hazard levels presented in Fig. 2. Maximum inter-story drift ratio (IDR) and peak roof acceleration (RA) are selected as the EDPs of interest. Three damageable groups are identified: structural components, non-structural components and URM infill walls. IDR is used later to determine the damage state of the structural components and URM infill walls. Whereas, RA is related to the performance of non-structural components.

As discussed in Section 2, three probabilities quantities are calculated: $POE(EDP > edp|IM = im)$, $p(C|IM)$ and $p(NC|IM)$ for each hazard level. Probability of collapse is calculated, simply, as the number of ground motions that lead the structure to collapse divided by the total number of ground motions run at that hazard level. Probability of non-collapse is just the complementary probability of the previously defined $p(C|IM)$, namely, $p(NC|IM) = 1 - p(C|IM)$. For this case-study the collapse state is assumed to be reached when the structure's peak IDR, in any of the two orthogonal directions, surpasses 2%, this threshold is obtained from a pushover analysis. Probabilities of collapse and non-collapse are reported in Table 1.

Table 1. Probabilities of collapse and non-collapse for different IMs.

Hazard Level	Number of Ground Motions	Number of Collapses	$p(C\|IM)$	Number of Non-Collapses	$p(NC\|IM)$
1	12	1	0.0833	11	0.9167
2	7	2	0.2857	5	0.7143
3	11	6	0.5455	5	0.4545
4	6	5	0.8333	1	0.1667
5	6	5	0.8333	1	0.1667
6	6	6	1.0	0	0.0

7	6	6	1.0	0	0.0
8	4	4	1.0	0	0.0
9	7	7	1.0	0	0.0

The probabilities of the EDPs, namely, $POE(EDP > edp|IM = im)$ are calculated only from the non-collapsed cases. It is worth noting, that due to the small number of ground motions available per hazard level, it is only possible to get statistics for the first 3 hazard levels. A lognormal distribution is used to fit the probability distributions of interest, as this distribution has been shown to fit EDPs adequately according to the literature [18]. Probability and POE of peak IDR_x, and RA_x are shown in Fig. 3.

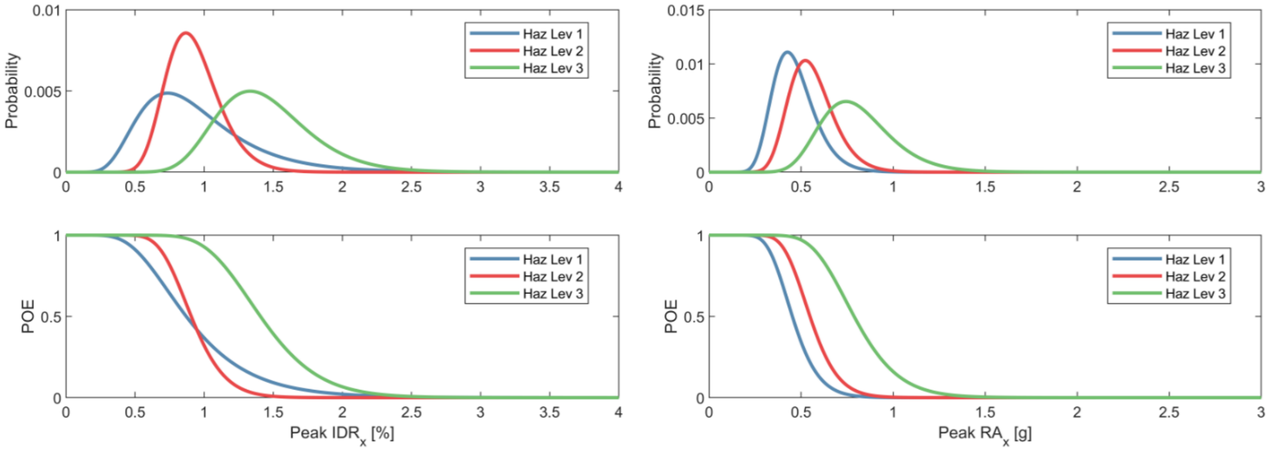


Fig. 3 – Probability and POE of IDR_x and RA_x.

4.3 Damage Analysis

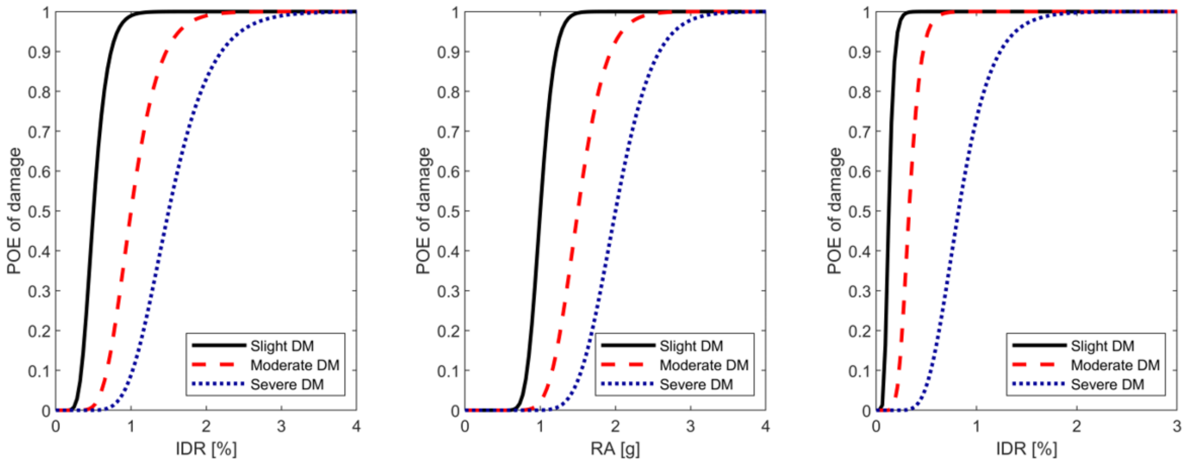
Fragility functions use in this section are obtained from literature for the three damageable groups previously discussed, i.e.: structural components, non-structural components and URM infill walls. For all cases three damage stages are used slight, moderate and severe damage. IDR is used to determine the damage state of the structural components and URM infill walls. Whereas, RA is related to the performance of non-structural components.

A lognormal distribution is used to fit the POE of damage. Fragility functions for the structural components are defined following the work of [1]. Values to determine the fragility of the non-structural components are based on damage information reported by [19] in Table 6.9 of his PhD dissertation for the "Acceleration sensitive non-structural component".

The literature is lacking reliable fragility functions for URM infill walls, and further development needs to be done in this area. Furthermore, those that do exist had been developed for in-plane loading only. Herein, values reported by [20] are used to determine drift-based fragility functions for URM infill walls. Table 2 presents the median and COV values for the damage levels of all three damageable groups. Fragility curves for the three damageable groups are presented in Fig. 4.

Table 2. Median and COV of EDPs for different damage levels.

Component	Damage Level	EDP	Median	COV
Structural	Slight	IDR	0.5%	0.3
	Moderate		1.0%	0.3
	Severe		1.5%	0.3
Non-structural	Slight	RA	1.0 g	0.15
	Moderate		1.5 g	0.20
	Severe		2.0 g	0.20
URM infill wall	Slight	IDR	0.125%	0.325
	Moderate		0.327%	0.278
	Severe		0.820%	0.320



(a) Structural Components (b) Non-Structural Components (c) URM Infill walls

Fig. 4 – Fragility functions.

4.4 Loss Analysis

Economic loss is used for this case study as DV. Values were assumed based on a conversation with a local expert contractor. Because of the lack of knowledge, a large Coefficient of Variation (COV) of 0.4 will be used for all the loss functions. Furthermore, all functions are assumed to follow a lognormal distribution.

The mean total cost of construction is assumed to be \$1.0 million. 35% and 10% of the total cost of construction are assigned to the mean cost of the structural components and URM infill walls, respectively. For the non-structural component, the remaining 55% of the construction cost in addition to 50% of the total cost of construction (i.e.: \$0.5 millions) to consider the content of the building are allocated to the mean cost of non-structural components. For all three groups, the mean cost assigned for each DM correspond to 16.66%, 50% and 100% (for slight, moderate and severe DM, respectively) of the total mean cost for that given component. Additionally, a loss curve for the POE of economic losses for the case of global collapse is also developed. For this curve, the same vale of COV = 0.4 is used, and the mean economic losses due collapse is assumed to be $1.5 million which correspond to the sum of the total value of the three components, previously reported. Economic loss functions are shown in Fig. 5.

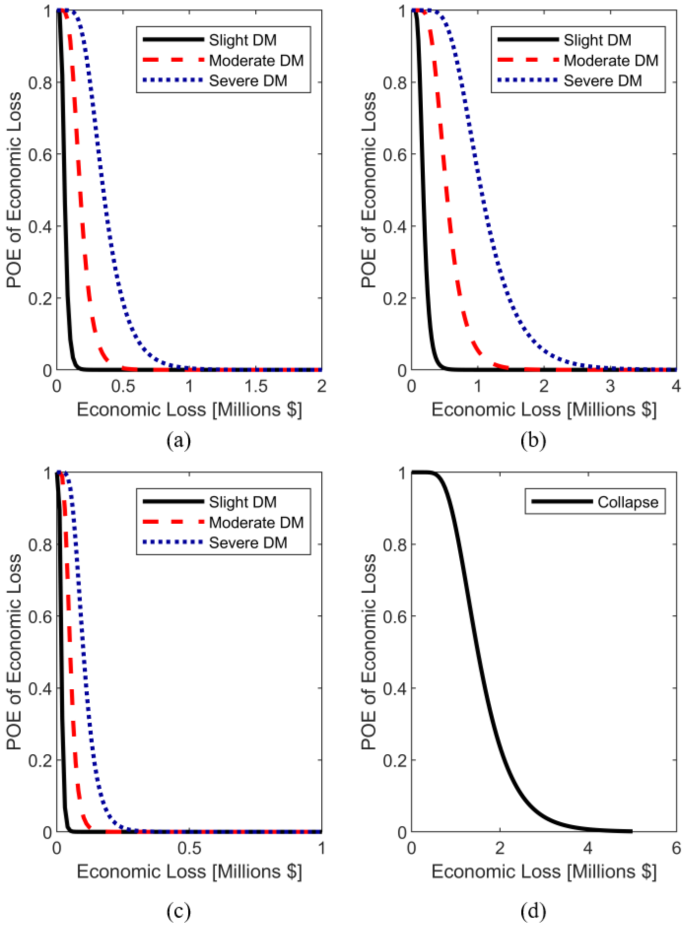


Fig. 5 – Loss functions for (a) Structural components. (b) Non-structural components. (c) URM infill walls. (d) Total collapse.

4.5 Loss Curve

After all four analyses have been performed, the final stage is to combine them using the total probability theorem discussed in Eq. (1) of Section 2. Since only three hazard scenarios were considered, the loss curve presented in Fig. 6 shows the result for each hazard levels separated.

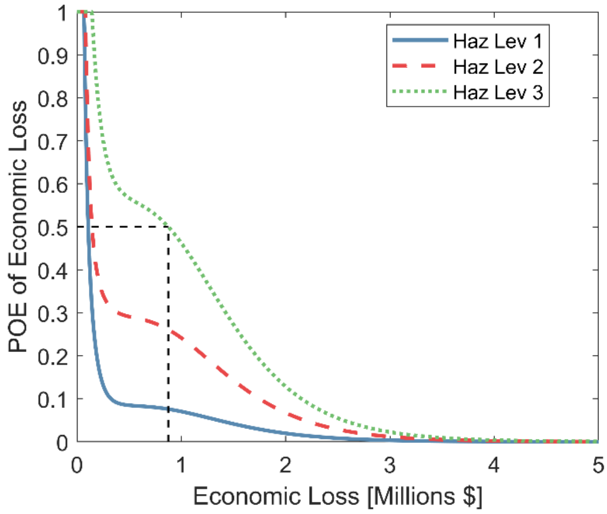


Fig. 6 – Loss curve.

5. Conclusions

RC frames with URM infill walls are widely used in earthquake resistant construction in seismically active regions around the world. Assessing the structural behavior of this type of structures is a very complex problem that depends many variables, such as the constitutive theory of the used materials (brick and mortar mechanical properties), the masonry unit geometry, labor quality, the stiffness ratio of the frame and the infills, the presence of openings in the wall, aspect ratio of the panels, frame/wall interface conditions.

Finally, an application of a PBEE assessment for the archetype building was performed, using Economic Loss as decision variable. Overall, it was a useful task to go over all the stages of the methodology for this structural typology, since it has been lacking in the literature. It allowed to identify areas of improvement and to create a framework that can be improved in the future once some new developments arise regarding better structural models, fragility functions and loss functions for the different components of the structure. Moreover, even with the simplifications made, some insightful result can be obtained: the economic losses even for a Design Basis Earthquake (DBE) level the economic losses are approximately 60% of the total cost of the building and its content.

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7. Copyrights

8. References

1. Günay, S. and K.M. Mosalam, PEER Performance-Based Earthquake Engineering Methodology, Revisited. Journal of Earthquake Engineering, 2013. 17(6): p. 829-858.
2. Mosalam, K.M., 1996 EERI Student Paper Award Modeling of the Nonlinear Seismic Behavior of Gravity Load Designed Frames. Earthquake Spectra, 1996. 12(3): p. 479-492.
3. Nicola, T., et al., Masonry Infilled frame structures: state-of-the-art review of numerical modeling. Earthquakes and Structures, 2015. 8.
4. Peralta, N.R., Computational Strategies for Multi-Scale Modeling of Masonry Components and Structures. 2016, University of California, Berkeley: Ann Arbor. p. 247.
5. Lee, T.-H. and K.M. Mosalam, Probabilistic Seismic Evaluation of Reinforced Concrete Structural Components and Systems. 2006: Pacific Earthquake Engineering Research (PEER) Center.
6. AIS, Reglamento Colombiano de Construccion Sismo Resistente NSR-10. 2010, Asociacion Colombiana de Ingenieria Sismica.
7. Moehle, J. and G.G. Deierlein. A framework methodology for performance-based earthquake engineering. in 13th world conference on earthquake engineering. 2004.
8. Arteta, C.A. and N.A. Abrahamson, Conditional Scenario Spectra (CSS) for Hazard-Consistent Analysis of Engineering Systems. Earthquake Spectra, 2019. 35(2): p. 737-757.
9. CSI, ETABS. 2018: Computers and Structures, Inc.
10. McKenna, F., M.H. Scott, and G.L. Fenves, Nonlinear Finite-Element Analysis Software Architecture Using Object Composition. Journal of Computing in Civil Engineering, 2010. 24(1): p. 95-107.
11. Spacone, E., F.C. Filippou, and F.F. Taucer, Fibre Beam-Column Model for Non-Linear Analysis of R/C Frame: Part I. Formulation. Earthquake Engineering and Structural Dynamics, 1996. 25: p. 711-725.
12. Wittmann, F., et al., Fracture energy and strain softening of concrete as determined by means of compact tension specimens. Materials and Structures, 1988. 21(1): p. 21-32.
13. Kadysiewski, S. and K.M. Mosalam, Modeling of Unreinforced Masonry Infill Walls Considering In-Plane and Out-of-Plane Interaction. 2009, PEER Center.
14. Mosalam, K.M. and S. Günay, Progressive Collapse Analysis of Reinforced Concrete Frames with Unreinforced Masonry Infill Walls Considering In-Plane/Out-of-Plane Interaction. 2015. 31(2): p. 921-943.
15. ASCE, PRESTANDARD AND COMMENTARY FOR THE SEISMIC REHABILITATION OF BUILDINGS, in Report No. FEMA-356. 2000, American Society of Civil Engineers: Washington, D.C.
16. Talaat, M.M. and K.M. Mosalam, Computational Modeling of Progressive Collapse in Reinforced Concrete Frame Structures. 2008, Pacific Earthquake Engineering Research Center (PEER)
17. Candia, G., et al., A New State ‐ of ‐ the ‐ Art Platform for Probabilistic and Deterministic Seismic Hazard Assessment. Seismological Research Letters, 2019. 90(6): p. 2262-2275.
18. Baker, J.W., Efficient Analytical Fragility Function Fitting Using Dynamic Structural Analysis. Earthquake Spectra, 2015. 31(1): p. 579-599.
19. Bohl, A., Comparison of performance based engineering approaches. 2009.
20. Chiozzi, A. and E. Miranda, Fragility functions for masonry infill walls with in-plane loading. Earthquake Engineering & Structural Dynamics, 2017. 46(15): p. 2831-2850.

