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Performance-Based Seismic Assessment of Reinforced Concrete Frame Buildings with
Unreinforced Masonry Infill Walls

By

Jorge Luis Archbold Monterrosa

A dissertation submitted in partial satisfaction of the
requirements for the degree of

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in

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in the

Graduate Division

of the

University of California, Berkeley

Committee in charge:

Professor Khalid M. Mosalam, Chair

Professor Shaofan Li

Professor Tarek I. Zohdi

Spring 2022

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Unreinforced Masonry Infill Walls

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Jorge Luis Archbold Monterrosa

Abstract

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Jorge Luis Archbold Monterrosa

Doctor of Philosophy in Civil and Environmental Engineering

University of California, Berkeley

Professor Khalid M. Mosalam, Chair

During the last two decades, the structural engineering profession has realized the need for a change in the traditional code design philosophy to assess existing structural systems and the design of new ones. Traditionally, researchers use deterministic methods, and the outcomes are typically presented in terms of variables only relevant to the structural engineers, such as forces or displacements. Instead, modern methods follow a Performance-Based Earthquake Engineering (PBEE) approach. This trend of PBEE has been motivated by observations after past earthquakes. Even with a small number of or no collapse cases and when buildings met the life safety objective of the traditional design philosophy, the socio-economic losses due to damage, monetary losses, repair costs, or downtime, could be significantly high and unacceptable to the owners of the structures and the community as a whole.

To overcome the shortcomings of the traditional code design philosophy, probabilistic methods that consider all sources of uncertainties that affect the seismic performance of structures are gaining popularity not only in academia, but also in the practicing engineering community. The Pacific Earthquake Engineering Research (PEER) Center developed the PBEE methodology to evaluate and design structures using performance measures which are in the direct interest of stakeholders, instead of engineering parameters. The PEER PBEE methodology decomposes the problem into four different phases of analyses: hazard, structural, damage, and loss. Furthermore, it considers uncertainties in all the four phases, along with a robust formulation of their accumulation.

This study presents a Performance-Based framework for the seismic assessment of Reinforced Concrete (RC) frame buildings with Unreinforced Masonry (URM) infill walls. The selection of this structural typology is motivated by its widespread presence in countries with developing economies worldwide in high seismicity regions with high vulnerability and exposure. Case studies are presented with geometric features and reinforcement configuration of buildings compatible with the Colombian building typology in high-risk seismic zones. The performance of the archetype buildings in the case studies are estimated in terms of economic losses using the PEER-PBEE methodology.

One key finding was that, at least for the considered two buildings, there is not a significant change in the loss estimation due to the uncertainty in the damage and loss functions for the URM infill wall. The impact of including or not including the infill walls on the loss estimation process

was also assessed, for the 4-story ductile building which had high strength infill walls, there was not a significant change in the loss curves of the bare frame case compared to the infilled case. On the other hand, for the 6-story non-ductile building which had low strength infill walls, the inclusion of the infills in the loss estimation process did increase the expected losses when compared to the bare frame case. Finally, the loss estimation for both case study buildings was compared; it was noted that for both cases, infilled and bare frames, the probability of exceeding a given threshold of economic loss is much smaller for the 4-story ductile case study compared to the 6-story building, as expected, a ductile building performs better under seismic loads when compared to a non-ductile building, which translates into less economic losses.

Keywords: Earthquake Engineering, Performance-Based Engineering, Reinforced Concrete Frame, Uncertainty, Unreinforced Masonry (URM) Infill Wall.

To my parents and Camila,
for believing in me and supporting me through anything.

A mis padres y Camila,
por creer en mí y apoyarme siempre.

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Introduction

In seismically active regions worldwide, engineers use Reinforced Concrete (RC) frames with unreinforced masonry (URM) infill walls in earthquake-resistant construction. This structural typology is more common in countries with developing economies (e.g., Latin American countries). But many of these buildings still exist in developed countries, some from before the adoption of modern seismic codes and some well-engineered according to recent building codes. Architects and engineers primarily use URM infill walls as elements that partition spaces within the structures. When not separated from the frame during the design and construction, these very rigid elements typically interact with the more ductile reinforced concrete frames during an earthquake in an undesired manner. Observations from past earthquakes in countries like Colombia, Turkey, Italy, Ecuador, Mexico showed that many structures of this type often behave poorly and even collapse.

Such poor performance of structures using RC frames with URM infill walls can be attributed to the fact that the interaction between the URM infill walls and the RC frames is a complex problem that has not been yet fully understood. And these are rarely considered elements that contribute to the structure's lateral resisting system, which leads to an inconsistency between the numerical model used in the design and the actual constructed building.

As previously stated, recent seismic events have demonstrated the poor structural behavior of RC frames with URM infill walls. The consequences of this poor structural behavior extend beyond structural damage and usually lead to injuries, fatalities, downtime, and economic losses. The engineering community has realized long ago that the amount of damage, the financial loss due to downtime, and repair cost of structures could be unacceptably high, even for structures that complied with the latest building codes based on traditional design philosophy (Lee & Mosalam, 2006). 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 avoid the overall or partial collapses of buildings in high-intensity earthquakes, while allowing significant levels of damage (Günay & Mosalam, 2013).

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 subjected to earthquakes in terms of performance measures of interest to 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 (Günay & Mosalam, 2013). 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 include the framework's

full extent. Hence, this dissertation contributes to extend the PEER PBEE methodology to RC frames with URM infill walls.

Application of a probabilistic PBEE methodology, such as the PEER PBEE, on RC frames with URM infill walls is useful for several reasons. First, it allows quantifying the performance of this type of buildings in earthquakes with varying intensities using a consistent set of decision variables. Second, this quantification allows highlighting particular deficiencies that need to be addressed to improve the performance of these buildings and identifying these deficiencies in the order of importance. Finally, existing buildings of this type may need retrofit, and new design methodologies, including protective systems, need to be developed for the design of new buildings. Probabilistic PBEE methodologies can guide these designs with the objective of minimizing the losses. Therefore, this dissertation focuses on the adoption of the PEER PBEE methodology for evaluation of RC frames with infill walls. Special emphasis is placed on a hazard-consistent assessment of two archetype buildings, one designed according to the latest Colombian seismic provisions, and the second one a non-ductile building conceived to represent building practices in Latin America before the implementation of seismic provisions, pursued as case studies.

Scope

This research discusses the high risk of RC frames with URM infill walls during moderate to severe earthquakes in terms of financial losses to better understand its causes and implications. The discussion opens with recent field observations from Latin American countries of damaged RC frames buildings with URM infill walls, and ends with a Performance-Based assessment of a Colombian-code-compliant archetype building using the PEER-PBEE methodology. The discussion is fed by a comprehensive review of the state-of-the-art of all the relevant phases of the methodology focused on the typology of interest, improving the identified gaps in literature. The intent is to provide the engineering community with a roadmap for implementing each of the PEER PBEE framework phases for the aforementioned structural typology.

Objectives

The overarching goal of this research is to investigate and develop a Performance-Based Framework for estimating the seismic assessment of reinforced concrete frame buildings with unreinforced masonry infill walls. To this end, the study:

- demonstrates the seismic vulnerability of RC frame buildings with URM infill walls, using field evidence from recent earthquakes and analytical simulations.
- improves the response evaluation of RC frames with URM infill walls with several critical numerical modeling and simulation aspects.
- estimates the seismic performance of an archetype building in terms of economic losses while considering various uncertainty sources.

Organization

The dissertation comprises nine chapters highlighting the research significance of the previously described objectives. It seeks to outline the steps to carry out a performance-based seismic assessment of RC frame buildings with URM infill walls.

Chapter 1 provides a brief overview of previous field observations of the performance, often poor, of RC frames with URM infill walls, after moderate to major earthquakes in Latin America. The reported observations serve as motivation for this study.

Chapter 2 presents a literature review on major characteristics and variables that dominate the behavior of URM infill walls. It reviews the in-plane (IP) and out-of-plane (OOP) behavior and the IP/OOP interaction. The chapter closes with a summary of numerical modeling techniques for URM infill walls available in literature.

Chapter 3 describes the PEER PBEE methodology. It also discusses the evolution of performance-based earthquake engineering and gaps in the literature, particularly in relation to implementing the methodology to RC frame buildings with URM infill walls.

Chapter 4 introduces two buildings to be studied further in subsequent chapters. First building is a 4-story building archetype located in the city of Armenia, Colombia, which was designed following the latest seismic codes in Colombia. Second building is a non-ductile 6-story archetype building to represent existing buildings designed and built before the first seismic provisions were enacted.

Chapter 5 proposes a seismic hazard model for the city of Armenia, Colombia, where the archetype buildings are located. In this chapter, a brief history of the tectonic setting in Colombia and the most significant seismic events affecting Colombia is first presented. This is followed by a review on how to perform a Probabilistic Seismic Hazard Analysis (PSHA). Then the input parameters for the proposed hazard model for Armenia are discussed. Finally, a PSHA and a Conditional Scenario Spectra (CSS) are performed to obtain hazard curves and ground motions consistent with the hazard of the area of interest.

Chapter 6 studies the structural response of the two archetype buildings presented in Chapter 4 under realistic suites of ground motions selected in Chapter 5 that represent the hazard at the site. The response is assessed with Nonlinear Time History (NLTH) analyses. The chapter starts with a description of the modeling techniques used to represent reinforced concrete moment resisting frames and the URM infill walls. It's followed by a study of the accurate performance of the infill model using a 1-story 1-single bay infilled frame. Then non-linear static analyses are performed; followed by non-linear time history analyses. The chapter closes with a fragility analysis of the archetype buildings and the estimation of the rate of exceedance of certain engineering demand parameters.

Chapter 7 reviews the literature for previously developed fragility functions that allow the estimation of levels of damage in unreinforced masonry infill (URM) infill walls as a function of in-plane (IP) and compares how the use of one set of fragility or other impacts the loss estimation of the case study buildings. Moreover, this chapter describes the fragility functions used to the

other components of our case-study buildings, namely for the structural elements (i.e., beams and columns) and all other non-structural components.

Chapter 8 presents loss functions for all different components of the two case study buildings, focusing primarily on the developing of the URM loss functions. Furthermore, this chapter discusses the loss analysis and the combination of the four PEER PBEE phases.

Finally, Chapter 9 summarizes and highlights the most important aspects of the findings and discusses recommendations for future development in this line of research.

1. Field Observation and Motivation

Damage observed in reinforced concrete frame buildings with unreinforced masonry infills after previous earthquakes in Armenia, Colombia, 1999 (Mw 6.1); Kocaeli, Turkey, 1999 (Mw 7.4); L'Aquila, Italy, 2009 (Mw 6.3); Maule, Chile, 2010 (Mw 8.8); Muisne, Ecuador, 2016 (Mw 7.8); Puebla, Mexico, 2017 (Mw 7.1); and many others, have exposed vulnerabilities in the way engineers design and analyze this type of structures.

This chapter presents field observations of RC frames with URM infill walls that performed poorly and, in some cases, even collapsed during recent earthquakes. We limit the detailed discussion from previous earthquakes to three significant events that occurred within the last 25 years in Latin America and that caused enormous consequences to society. The first one occurred on January 25, 1999, in the Colombian coffee region, and it's known as the 1999 Armenia earthquake (Mw 6.1). The Armenia earthquake is amongst the most deadly and costly in the recent history of Colombia.

The second one is the 2016 Muisne earthquake, a Mw 7.8 earthquake that hit the west coast of Ecuador. It killed hundreds of people and left thousands more injured or displaced. The socio-economic impact of the event was estimated to be US \$3.3 billion, according to the Ecuadorian government. Plenty of RC frame buildings with URM infill walls suffered severe damage and even collapsed further, exposing the high risk of this structural typology when subjected to moderate to severe ground-shaking.

The third one is the Mw 7.1 Puebla earthquake that struck Mexico's central region on September 19, 2017. The author had the opportunity to be part of a post-earthquake reconnaissance mission that surveyed the affected neighborhoods of Mexico City after this earthquake. During the reconnaissance mission, the team observed a wide range of damage for the structural typology of interest for this dissertation. The following three sections provide information about these earthquakes and explain the observed damage.

1.1 The 1999 Armenia Earthquake (Mw 6.1)

On January 25, 1999, at approximately 1:19 p.m. local time, a moment magnitude Mw 6.1 struck the Colombian Coffee Region Servicio Geologico Colombiano (1999). The epicenter located 15 km, and 50 km south of the cities of Armenia, and Pereira, respectively (Restrepo and Cowan 2000), had epicentral coordinates of 4.432°N, 75.703°W, and a depth of 15 km as reported by the Servicio Geologico Colombiano (Servicio Geologico Colombiano 1999); as shown in Figure 1.1. The main shock's focal mechanism corresponded to a strike-slip rupture on a north-south trending fault plane (Restrepo and Cowan 2000). Colombia's National Seismic Network (RSNC in Spanish) recorded a total of 138 aftershocks within the month after the main event. The largest one happened on the same day, four hours after the main event, with a moment magnitude of Mw 5.5.

Although the earthquake was moderate in intensity, the proximity to highly populated areas like Armenia and Pereira and its shallow nature caused significant damage to many buildings. The

National Administrative Department of Statistics (abbreviated DANE in Spanish) reported 1185 fatalities, 8536 injuries, over 158918 people displaced from their homes, 35972 dwellings collapsed, 43476 dwellings with minor to moderate damage (DANE 1999). Although not reported, it is possible that some of the buildings that experienced moderate damage were also uninhabitable. Furthermore, the economic losses were estimated to be around US \$1.9 billion (Restrepo and Cowan 2000).

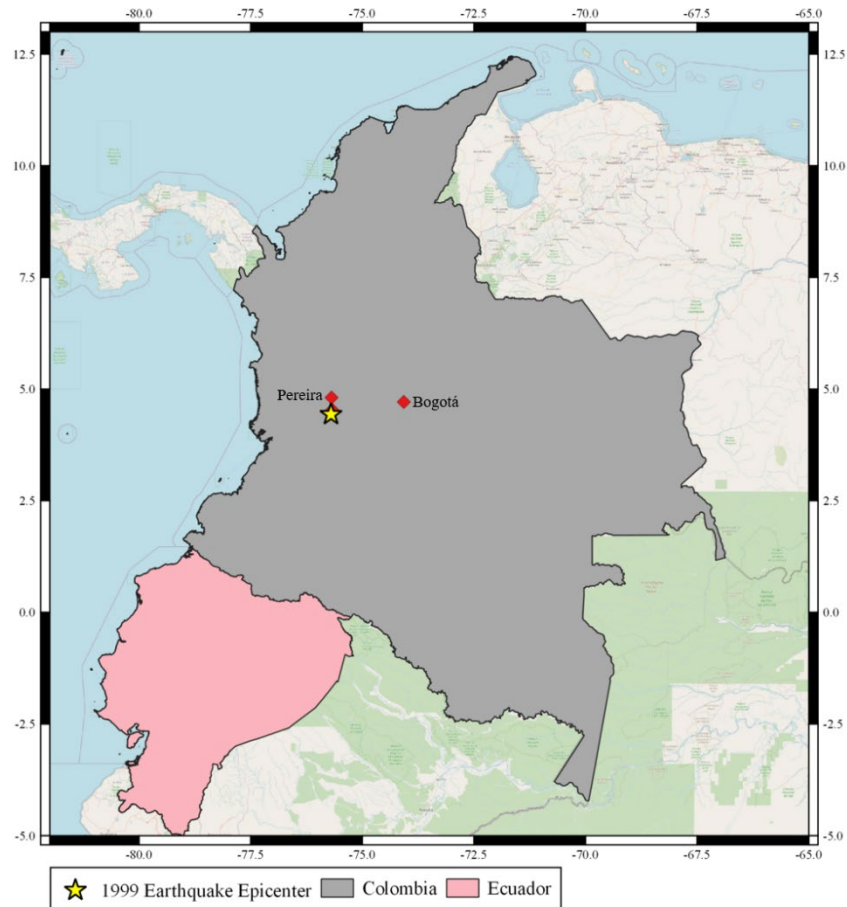


Figure 1.1 Epicenter of the 1999 Armenia Earthquake.

Many of the 79448 damaged structures predated the first seismic provision in Colombia, which was enacted in 1984. Most of the damage occurred in poor areas where non-engineered construction was typical (EEFIT 2000). The positive impact of adopting the first Colombian seismic code was evident when comparing the damage observed from previous earthquakes in the region (Garcia 2003), for example, the 1983 Popayan earthquake (Mw 5.5). However, even for structures designed according to the latest code, widespread damage to unreinforced masonry infill walls and other non-structural components was observed as reported by (EEFIT 2000, Restrepo and Cowan 2000, Garcia 2003). Figure 1.2 shows the severity of the damage across the urban areas near the epicenter. Figure 1.3 shows two examples of unreinforced masonry infill wall damage observed during the 1999 Armenia earthquake.



Figure 1.2 Damage observed after the Mw 6.1 1991 Armenia Earthquake. Adapted from El Tiempo (2019).



Figure 1.3 Examples of poor performance of RC frames with URM infill walls buildings. Adapted from Restrepo and Cowan (2000).

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Garcia (2000) reported the results of an effort made by the Colombian government to assess the extent of the damage in the city of Pereira after this earthquake. The effort sampled 12454 buildings located in the downtown area of Pereira. As part of the post-earthquake reconnaissance mission, the field team reported the type of construction, building use, and age of construction. Furthermore, they tagged buildings according to the damage level experienced by each structure. The structures visited were classified into four categories: red-tagged for severe structural damage to collapse; orange-tagged for moderate to severe structural, or severe non-structural damage that required the building to be evacuated until the repairs were completed; yellow-tagged for structures that could stay occupied while minor maintenance occurred; and green-tagged for minor structural or non-structural damage. Figure 1.4 shows some of the findings of the assessment performed in Pereira. It is observed that majority of yellow and orange tagged buildings were 4-6 stories height, more than half of which were reinforced concrete frames.

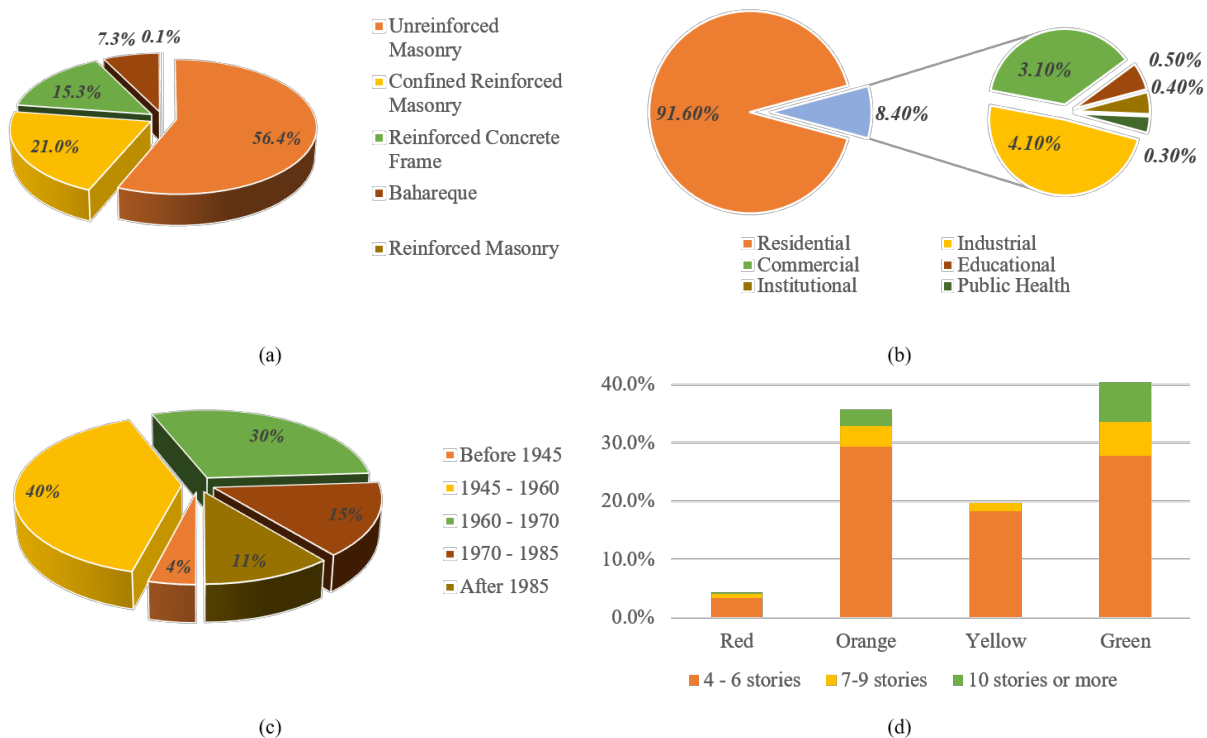


Figure 1.4 Statistics from the Pereira reconnaissance effort: (a) Type of construction. (b) Building Use. (c) Age of construction. (d) Damage assessment by building height. Adapted from Garcia (2003).

1.2 The 2016 Muisne earthquake (Mw 7.8)

The April 16, 2016, Mw 7.8 earthquake offshore of the west coast of Ecuador at 6:58 p.m. local time took place as a result of shallow thrust faulting at a depth of 20.6 km (USGS 2016). The earthquake occurred near or on the subduction zone between the Nazca and South America plates. The epicenter was located 27 km south-southeast from the coastal town of Muisne, Ecuador. The

hypocenter of the earthquake was located at 0.382°N , 79.922°W at a depth of 20.6 km, as reported by the U.S. Geological Survey (USGS 2016). Figure 1.5a shows the location of the epicenter and nearby cities that were affected by the seismic event; Figure 1.5b show Peak Ground Acceleration, PGA, maps produced by the ShakeMap program of the USGS (2016). The main event was followed by thousands of aftershocks, 11 of them with moment magnitude (M_w) greater than 6.0 (Villalobos et al. 2018).

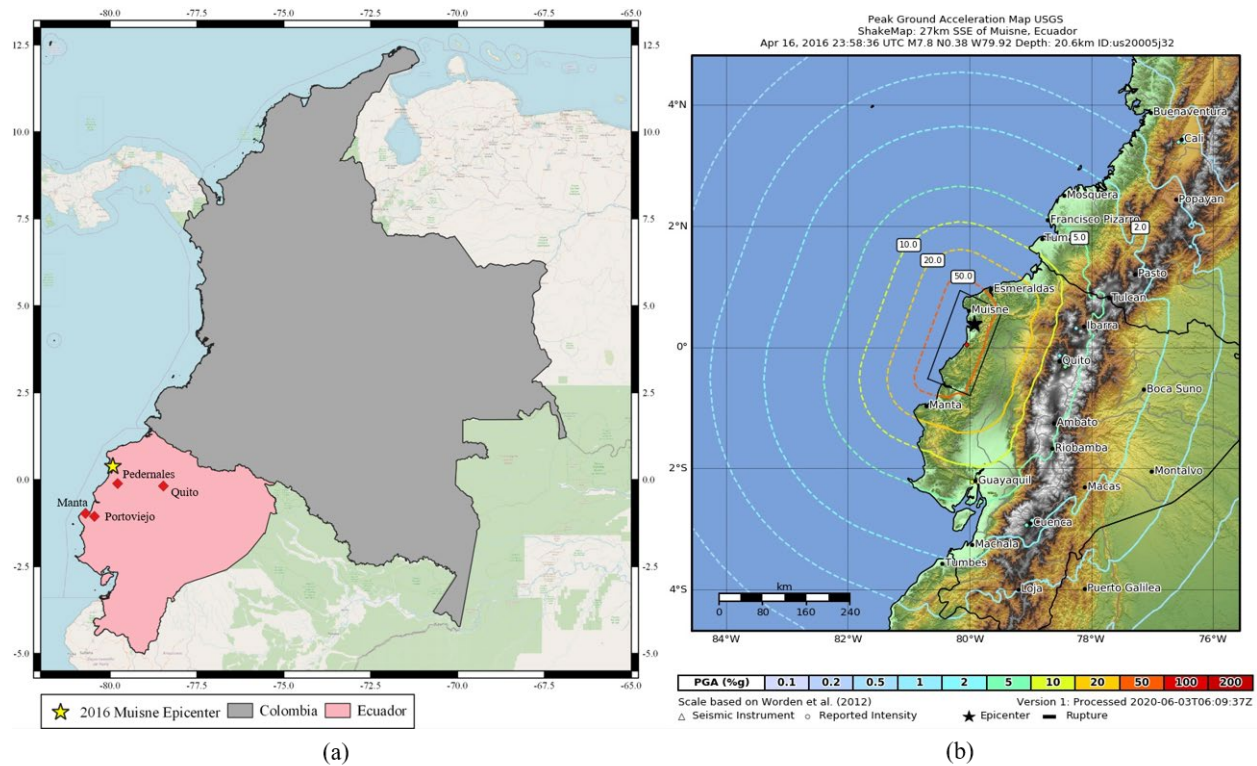


Figure 1.5 (a) Epicenter of the 2016 Muisne earthquake. (b) PGA contours estimates of ShakeMap (USGS 2016).

The earthquake of April 16, 2016 was the largest to strike Ecuador since 1979. It caused havoc in several nearby cities, among which were Muisne, Pedernales, Manta, and Puerto Viejo. The government of Ecuador reported 663 fatalities, nine people missing, 6274 injuries, and 28775 displaced and sent to shelters (Secretaria de Gestion de Riesgos 2016). The estimated economic losses from the event were around US \$3.3 billion. The total number of damaged structures is not available, but it is estimated to be thousands.

Although several structures were affected by the earthquake, the discussion herein is limited to the observed damage to the structural typology of interest for this manuscript. Reinforced concrete frame buildings with URM infill walls are common in the affected area and in the rest of the country. The main reason why URM infill walls are used as partition walls is that masonry is often locally produced, hence cheap and easy to access. Unfortunately, Ecuador lacks guidelines in design codes for these '*non-structural*' components and their attachment to the primary lateral

load resisting system. Furthermore, in Ecuador, as is common in Latin American countries, construction quality control is seldom provided by the structural designers. Figure 1.6 shows examples of observed damages in reinforced concrete frame buildings with unreinforced masonry infill walls. As observed in these examples, consequences of damage to unreinforced masonry infill walls were not limited to these walls, but affected the entire building either by the formation of soft/weak stories or introducing damage to the structural components due to the interaction between the infill walls and the frames



Figure 1.6 Examples of observed damages on RC frame buildings with URM infill walls after the 2016 Muisne earthquake (photos by C. Arteta).

Therefore, the damage observed after this earthquake reinforced the idea that poorly detailed RC frame buildings with heavy URM infill walls that are not adequately designed and built pose a high risk to the structure's health under severe lateral loading, hence an overall threat to the entire community.

1.3 The 2017 Puebla earthquake (Mw 7.1)¹

On September 19, 2017, a moment magnitude Mw 7.1 earthquake struck Mexico's central region at approximately 1:14 p.m. local time (UNAM 2017). The epicenter was located between Puebla and Morelos's states at about 120 km from Mexico City (Servicio Sismológico Nacional 2017). The epicentral coordinates reported by the United States Geological Survey (USGS) are 18.550°N, 98.489°W, with a depth of approximately 50 km (USGS 2017). The event corresponded to a normal faulting earthquake with a strike angle of 108° and a dipping angle of 47°. Such focal mechanisms are characteristic of an intraplate earthquake, which agrees with the regional seismicity, where The Cocos plate subducts under the North American plate. Figure 1.7 shows the location of the epicenter and most of the 25 stations that recorded the event.

One week after the event, the Pacific Earthquake Engineering Research (PEER) Center and the Colombian Earthquake Engineering Research (CEER) Network organized a four-day post-earthquake reconnaissance mission. Once on the ground, the teams joined forces with local researchers, practitioners, and faculty from the National Autonomous University of Mexico (UNAM in Spanish) and the Mexican School of Civil Engineers (CICM). This joint effort brought together over 20 engineers who conducted a rapid post-earthquake assessment of over 60 buildings in Mexico City neighborhoods where the damage was concentrated.

¹ This section uses figures and text from Arteta et al. (2019) in which the author is a main contributor.

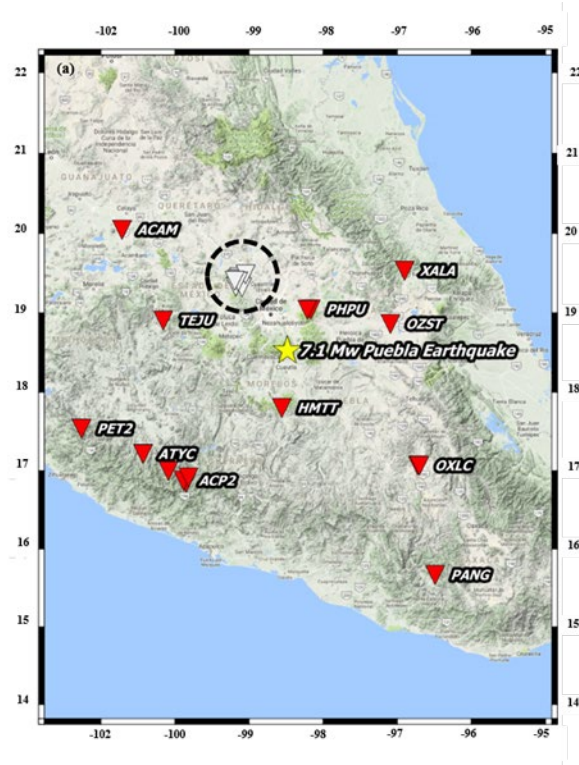


Figure 1.7 The Mw 7.1 Puebla Earthquake epicenter and locations of recording stations.

1.3.1 Post-Earthquake Reconnaissance Mission

The PEER-CEER team was on the field for four days giving support to the CICM-UNAM reconnaissance and tagging brigades. As stated before, the epicenter of the earthquake was located approximately 120 km south of Mexico City. Several smaller cities were closer to the epicenter and experienced severe damage due to the earthquake. However, the PEER-CEER team focused its reconnaissance efforts in Mexico City, where minimum safety standards were met according to the local jurisdiction. The reconnaissance mission was carried out from September 27 to 30, 2017, visiting over 60 sites tagged for apparent damage. The team received logistic help from the CICM and researchers from UNAM to focus on evaluating multistory RC structures. The main task was to determine if any building was under imminent collapse risk. Any knowledge gained on the status of apparently compromised structures was shared with the local authorities through CICM to expedite the critical decision-making process.

1.3.2 Statistics of the Damaged Buildings in Mexico City

The teams that supported the CICM reconnaissance effort inspected around 2,100 buildings. Approximately 25% of these buildings were identified with structural damage, while 61% were tagged as buildings without structural damage. The remaining 14% of the buildings could not be correctly classified for different reasons (CICM 2017). Damage associated with the soil-foundation system was observed in 51% of the buildings tagged with structural damage. Figure

1.8a shows the ratio of non-damaged to damaged buildings for different heights, including the number of visited buildings in each category. The height of about 86% of the buildings inspected varied between 1 and 8 stories. Out of the 933 buildings inspected having 1 to 3 stories, 26% were marked as damaged.

Additionally, 80 buildings with 4 to 8 stories were visited, out of which 28% were marked as damaged. The statistics of taller buildings show a similar trend, although the number of visited structures with more than nine stories was smaller. Figure 1.8b shows the distribution of different types of damage. Displayed categories are not exclusive; hence a single building may be tagged with various kinds of damage. More than 150 buildings (i.e., ~7.5% of inspected structures) were cataloged as partially or collapsed, while 322 buildings (i.e., 16% of inspected structures) had some structural damage. The damage associated with soil-structure interaction, including tilting, was assigned to 352 buildings (i.e., 18% of inspected structures).

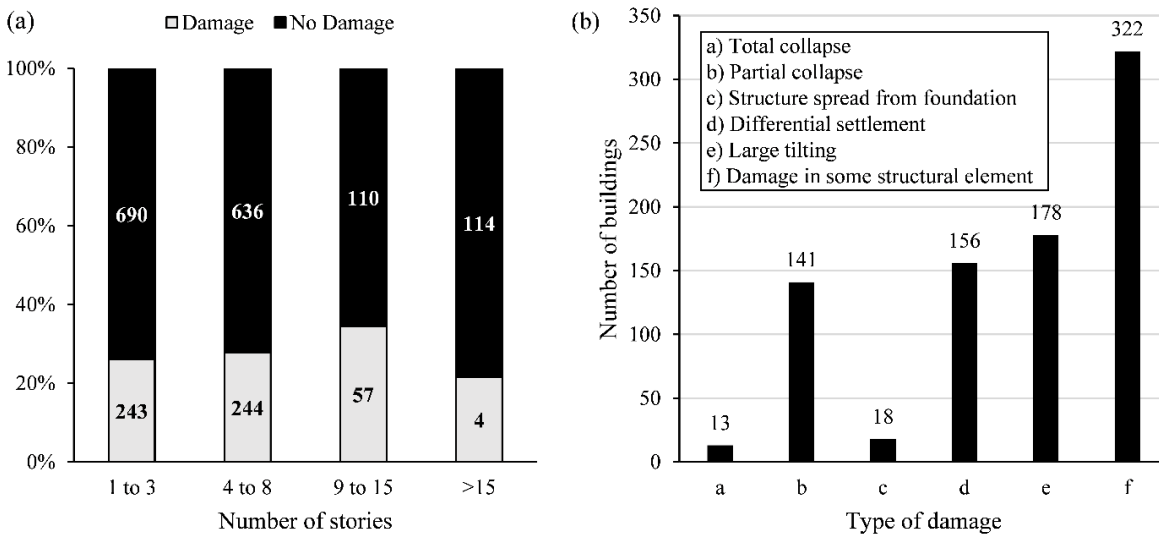


Figure 1.8 Statistics from the CICM reconnaissance team (a) ratio of non-damaged to damaged buildings in terms of their height; (b) number of buildings with different damage types.

Overall, a broad spectrum of performance levels was observed by the PEER-CEER team, from perfectly healthy and undamaged structures to partial or complete collapses. Figure 1.9 shows examples of Unreinforced Masonry infill wall damage observed during the reconnaissance mission. The visit's main focus was assessing frame-type structures with URM infill walls as this typology represents at least 13% of the engineered buildings in urban areas of several Latin-American countries (GEM 2014, Yepes-Estrada et al. 2017). A significant percentage of these structures were built using outdated design guidelines, namely regulations before the great Mexico City earthquake of 1985, that allowed relatively flexible frames, poor transverse reinforcement detailing, and did not provide specific conditions to isolate the non-structural walls from the main structure. Furthermore, the use of a flat slab supported by RC columns is still allowed, even though it has been widely documented that this type of structure does not perform well during earthquake excitation. Additionally, the extensive lack of enforcement of the building codes contributed to the poor performance of some of the structures affected by the earthquake.

Figure 1.10 summarizes the statistics of the observations made in the field by the PEER-CEER team. A total of 62 buildings were investigated. The buildings had between 2 and 12 stories, with the majority in the range of 5 to 8 stories. The structural period $C_u T_a$ ranged between 0.4 to 1.8 s and was evaluated following ASCE 7-16 (2016) recommendations (Figure 1.10a). The approximate fundamental period is $T_a = C_t h_n^x$, where h_n is the structural height in meters, and coefficients $C_t = 0.047$ and $x = 0.9$ are those of concrete moment-resisting frames. Coefficient C_u is taken as 1.4 to lessen the conservatism inherent in T_a 's estimation and to be consistent with structural periods of flexible buildings (FEMA-P1050-1 2015). Structural periods computed following ASCE 7-16 (ASCE 2016) recommendations fit in the range of periods measured in Mexico's instrumented buildings (Muriá and González 1995). As shown in Figure 1.10b, most of these buildings are described as RC frames with masonry infills (over 60% of the cases); other RC framed structures comprised masonry infills with shear walls or RC braces. Buildings with reinforced masonry composed 15% of the investigated structures, while only 4 (i.e., ~6.0% of the inspected buildings) steel buildings were investigated.



Figure 1.9 Examples of observed damages on Unreinforced Masonry infill walls (photos by J. Archbold).

The type of non-structural damage was cataloged into four categories (Figure 1.10c): (i) no damage, (ii) slight, (iii) intermediate, and (iv) severe. These four categories were adapted and modified from the European Seismological Commission's ones in their European Macroseismic Scale 1998, EMS-98 (ESC 1998). The EMS-98 developed a damage classification of concrete and masonry buildings based on qualitative attributes. The definition of the four non-structural damage categories can be found in Table 1.1.

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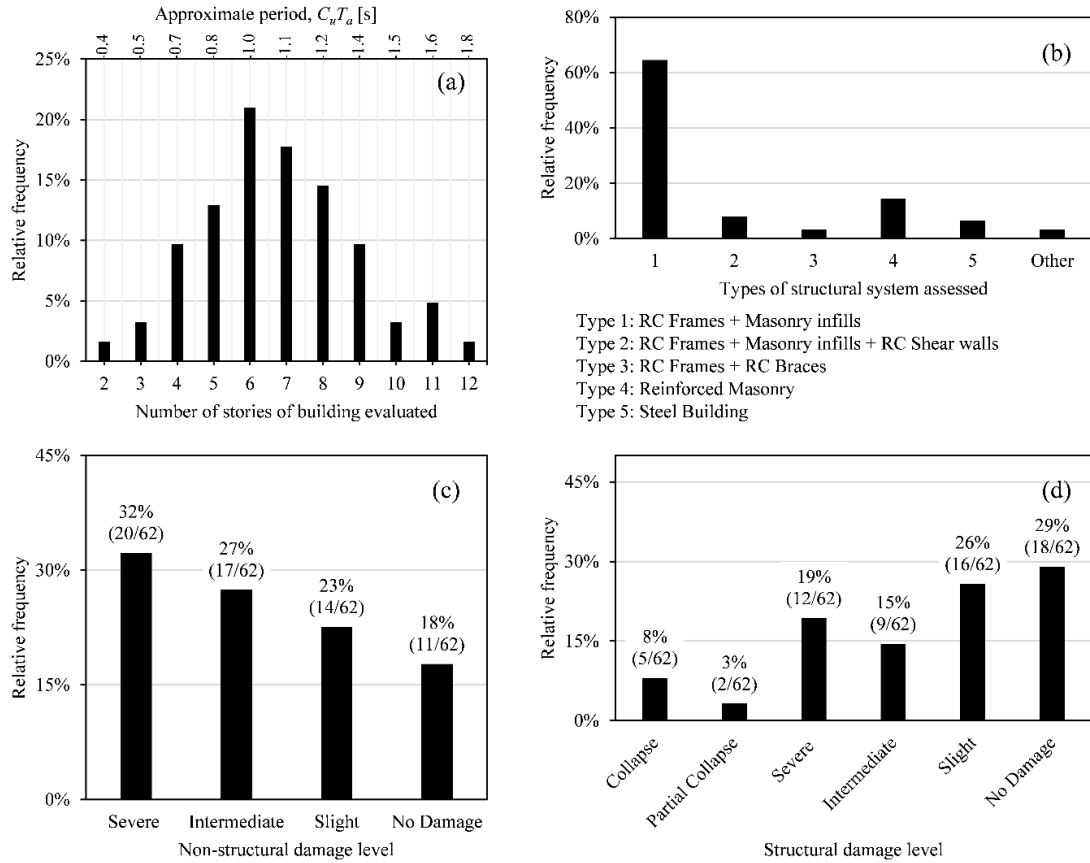


Figure 1.10 Statistics of inspected buildings by the CEER-PEER team: (a) building height; (b) type of structural system; (c) categories of non-structural damage; (d) categories of structural damage.

Table 1.1 Classification of non-structural damage.

Damage Level	Definition
No Damage	Minimal to no loss of function of the components with no impact on the performance of other elements, and building can be used after the seismic event.
Slight	Moderate loss of function of the components, no life hazard. Some minor repairs may be needed, and some areas of the building will be closed during repair time.
Intermediate	Large and extensive damage to non-structural components but no life-threatening risk. The functionality of the building will be partly affected, and repair work is required.
Severe	Serious failure of non-structural components. Severe interruption of the functionality of the building and possibly will lead to injuries and fatalities.

Approximately 60% of the buildings were cataloged as having intermediate or severe non-structural damage. The investigated RC buildings commonly used masonry as interior and exterior infill walls. According to (Meli 1994), the most popular masonry unit in Mexico is a solid clay brick with low strength. The Mexican Provision regarding the specifications and test methods for masonry, NMX-C-404 (ONNCE 2012), states that the thickness of the masonry units commonly used in Mexico ranged from 10 to 14 cm. A common practice is to exclude these walls from being part of the lateral load resisting systems. Instead, they have only considered the analytical models as gravitational loads exerted on the RC frame's beams. However, only on rare occasions, these walls are fully separated laterally from the primary structural system. Thus, once these types of structures are subjected to lateral loads such as earthquakes, the rigid walls attract a significant part of the lateral in-plane and out-of-plane loads while they are still intact. Since these walls are typically not adequately designed for such effects, they may fail. Many studies in the literature have shown the vulnerability of RC frames with masonry infills, e.g. (Mehrabi et al. 1996, Han-Seon and Sung-Woo 2002, Crisafulli et al. 2005, Hashemi and Mosalam 2006, Mosalam and Günay 2015). In general, buildings with widespread non-structural damage had several masonry infill panels with extensive cracking (cataloged as intermediate damage) and partial or total collapse of masonry units, usually on the façade planes.

The structural damage was cataloged into six categories, from no-damage to collapse (Figure 1.10d). An approximately uniform range of damage levels (average of about 22%) from undamaged to severely damaged were observed in buildings. Approximately 30% of the buildings were cataloged as having severe or worse structural damage, including five collapsed buildings. Typically, structures with intermediate damage had visible cracks with no spalling in columns and beams. Severely damaged structures exhibited concrete cover spalling in columns or shear walls, usually near the building base or within the lower third of the building height, showing signs of rebar buckling or lap splice failures. Other severely damaged columns exhibited signs of incipient development of planes of shear failure.

1.4 Concluding Remarks of Earthquake Reconnaissance

The 1999 Armenia earthquake devastated Colombia's coffee-growing region and prompted changes in the country's seismic provisions. The predominance of RC frames buildings with URM infill walls affected and the extent of the socio-economic consequences of such an event motivated the decision of choosing Armenia as a case study to study, in a hazard-consistent approach, to conduct the performance-based assessment of an archetype building designed according to the latest Colombian Building Code (i.e., Reglamento Colombiano de Construcción Sismo Resistente, or NSR-10).

The 2016 Muisne earthquake in Ecuador further helped to strengthen the importance of properly designing RC frame buildings with URM infill walls. Specifically, this earthquake highlighted the need for detailed provisions to be used for the design of non-structural components such as the URM infill walls and their proper attachment to (or detachment from) the RC frame.

Finally, a week after the 2017 Puebla earthquake struck Mexico, the PEER Center and the CEER Network organized a four-day post-earthquake reconnaissance mission to conduct a rapid post-earthquake assessment of 60 buildings in the affected areas of Mexico City. The team recorded the seismic performance of such structures; and both structural and non-structural components were assessed. The reconnaissance mission found many issues associated with reinforced concrete frames with infill walls.

These three seismic events and many others worldwide exposed RC frame buildings' vulnerability with URM infills when subjected to lateral forces. They served as a motivation for this study and as previously stated, inspired the archetype buildings selected in Section 4 for the investigations discussed. In particular, the observations and subsequent statistics from the 2017 Puebla post-earthquake reconnaissance mission, played a key role in the design of the second archetype building discussed in section 4.2, the height and configuration of the non-ductile building was one of the most common structural configurations with damage reported by the reconnaissance team.

2. Behavior of URM infilled frames

Unreinforced masonry infill walls surrounded by reinforced concrete or steel frames are still widely used as a lateral and gravitational resisting force system in seismically active countries, especially in countries with developing economies. Moreover, a large stock of these buildings predates seismic provisions and are at risk and need reliable seismic performance evaluations and retrofitting schemes to improve their behavior. In order to perform reliable evaluations and develop effective retrofit schemes, there is a need to understand and accurately model the structural response of the interaction between the surrounding frame and the URM infill walls subjected to lateral loads. However, understanding and modeling such interaction is a challenging task because it depends on many variables, such as the constitutive theory of the used materials (i.e., 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, the aspect ratio of the panels, frame/wall interface conditions, In-Plane/Out-of-Plane interaction, etc. (Mosalam 1996, Crisafulli 1997, Nicola et al. 2015, Peralta 2016). As a response to the large and wide variety of uncertainties in the problem at hand, the research community has devoted extensive efforts to minimize the epistemic uncertainties. The following subsections present a literature review on the behavior of infilled frames subjected to lateral loads, namely: in-plane (IP), out-of-plane (OOP), and their interaction. Additionally, a review of modeling techniques of infilled frames is presented.

2.1 Behavior of infilled frames

URM infilled frames are complex structures that exhibit highly non-linear behavior due to the interaction between the reinforced concrete frame and the URM infill wall. Masonry walls tend to be very rigid. On the other hand, if appropriately detailed, reinforced concrete frames tend to be more flexible and ductile, which causes the two parts of the structural system to deform very differently. This hybrid system usually exhibits a higher strength and stiffness than the strength of the bare frame (Crisafulli 1997). On the contrary, the infilled frame deformation capacity is reduced. According to Crisafulli (1997), one of the main drawbacks of using infilled frames is the degradation of stiffness, strength, and energy dissipation under seismic loading.

Unfortunately, many engineers typically choose to ignore the infill walls during the design and analysis process, neglecting all the previously discussed effects on the structure's performance. Thus, it is common only to include the masonry infills' effects as dead loads and masses during the modeling phase. Ignoring the infill's presence during the design and analysis phase could cause potential issues that could result in the poor behavior or even collapse of these structures. Some of these issues are discussed next:

- **Soft or weak stories:** Occur when a story (typically the first one) has none or considerably fewer infill walls than the levels above or below. This creates a story in the building that is substantially less stiff, allowing for concentrated deformations on such a weak story to occur, as depicted in Figure 2.1a. It is also noted that soft or weak stories can form during an earthquake after the failure of stiff infill walls in one of the stories

- Figure 2.1b shows an example of a soft-story collapsed building in Mexico City after the 2017 Puebla earthquake; in this example, the building's first story was used for parking space, resulting on a first floor with fewer infill walls than the residential floors above.

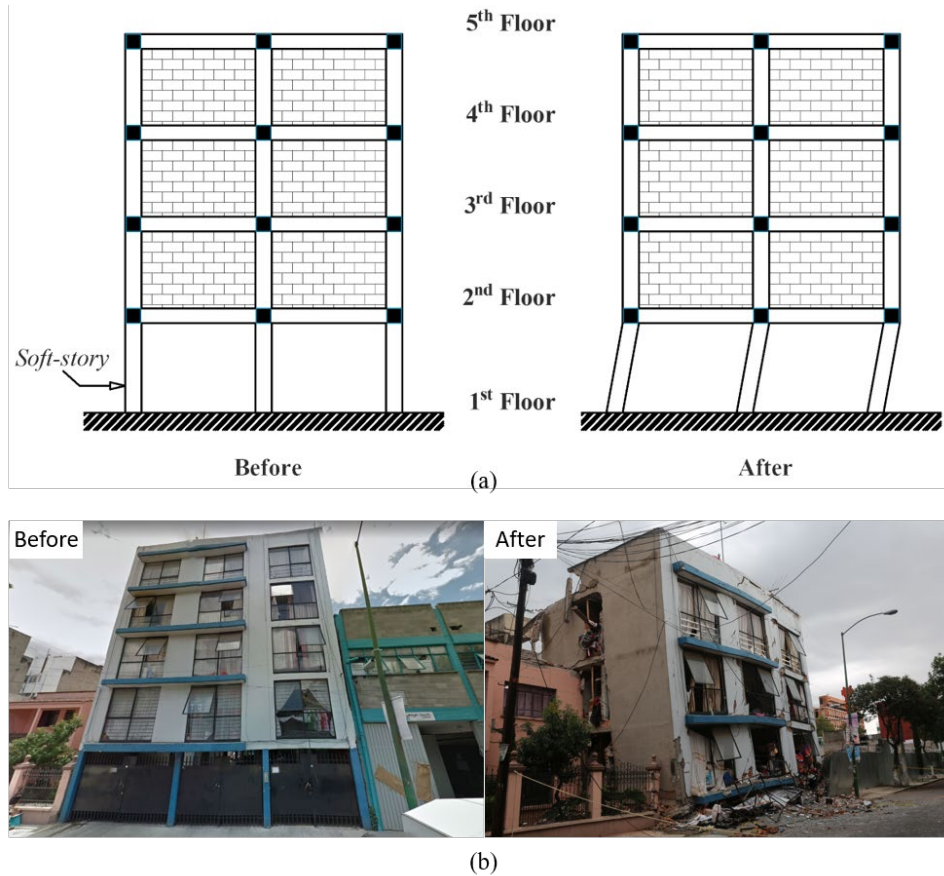


Figure 2.1 (a) Conceptual visualization of soft-story concentrated deformations. (b) A soft-story collapse of URM infilled frame building after the 2017 Puebla earthquake. (photos by J. Archbold).

- Short or captive columns: Occurs when infill walls framed into columns extend for only part of the column height or when openings like a window are next to a column. This effectively results in a shortened length of the column, leading to larger shear forces than the column is designed for. In some cases, those shear forces are such that they can exceed the designed capacity and cause severe damage and even collapse of the column. Figure 2.2 displays the difference between the assumed and actual moment distribution within a captive column; this translates into a discrepancy between the assumed and actual shear forces within the column, as previously discussed. Figure 2.3 shows two examples of observed damage in two schools in Puerto Rico after the 2019-2020 earthquake sequence that struck the island (Miranda et al. 2020).

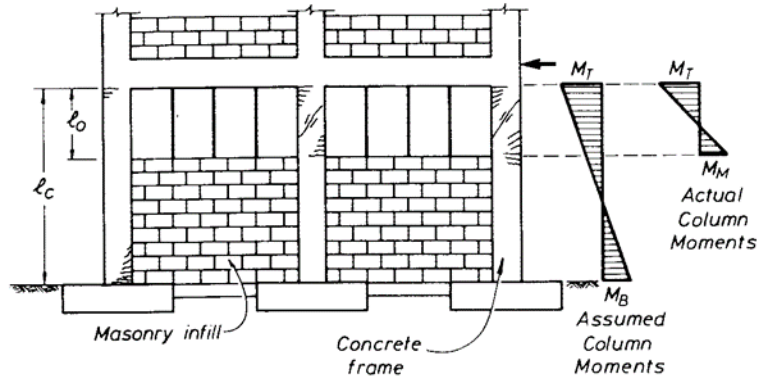


Figure 2.2 Difference between assumed and actual column moments and thus internal shear forces within.



Figure 2.3 Buildings with concentrated shear damage in columns due to captive column effect following the 2019-2020 Puerto Rico earthquake sequence. (photos by J. Archbold and A. Messina).

- Torsion due to plan irregularities: Occurs when there is a non-uniform spatial distribution of infill walls in the building plan (Semnani et al. 2014). The uneven distribution of infill walls in plan view creates a torsional demand on the structure due to the difference between the center of mass and the rigidity center (Paulay and Priestley 1992). The induced torsion causes additional deformation demands on the columns in the more flexible frames of the structure, sometimes even exceeding the frame element's capacity. Figure 2.4 presents an example of a building's structural plan with URM infill walls only on two of the four exterior frames. The non-uniform distribution of the infill walls creates eccentricities in both longitudinal and transverse direction, inducing undesired torsional demands on the columns.

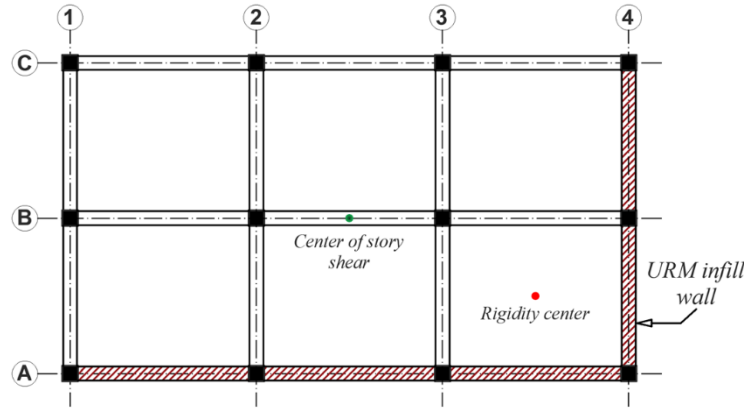


Figure 2.4 Example of a structural plan with non-uniform spatial distribution of infill walls. Adapted from Paulay and Priestley (1992).

URM infill walls undergo different loading types during their lifetime (i.e., vertical or gravity loads and lateral loads). The first kind, as expected, generates compressive stresses in the wall in the same vertical direction. The lateral loads, induced by seismic actions, generate shear and flexural stresses in the infill wall. Furthermore, lateral loads can be separated into two: in-plane and out-of-plane actions, as shown in Figure 2.5. The following sections will discuss these two types of lateral load separately (i.e., in-plane and out-of-plane). A discussion on the influence of the IP/OOP interaction on the behavior is presented later.

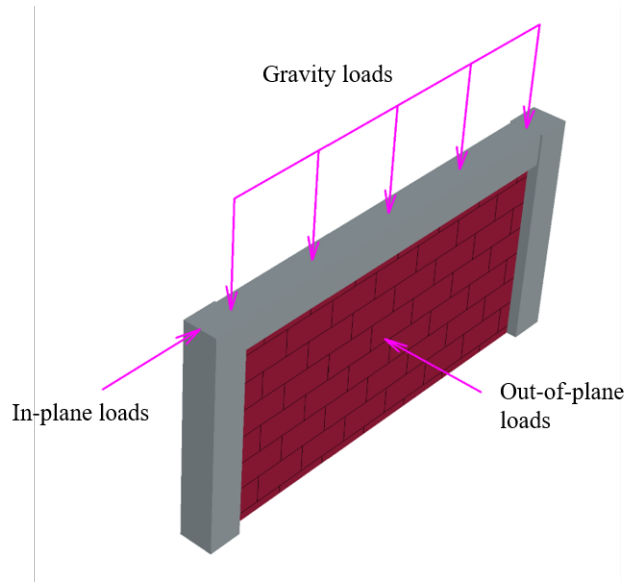


Figure 2.5 Loadings on the URM infilled frame.

2.2 In-plane (IP) behavior.

The infilled frame acts as a structural wall with boundary elements at low levels of in-plane horizontal force. As the in-plane deformation increases, the RC frame tries to deform in a flexural mode; meanwhile, the more rigid infill wall tries to deform in a shear-dominated mode. The

deformation incompatibility results in the detachment of the infill wall from the frame elements at the corners along the tension diagonal, as displayed in Figure 2.6a. Furthermore, after separation, the infill wall can be thought of as a diagonal compression strut. On the loaded corner, normal and shear stresses are created along the contact length between the infill wall and the frame elements (Crisafulli 1997).

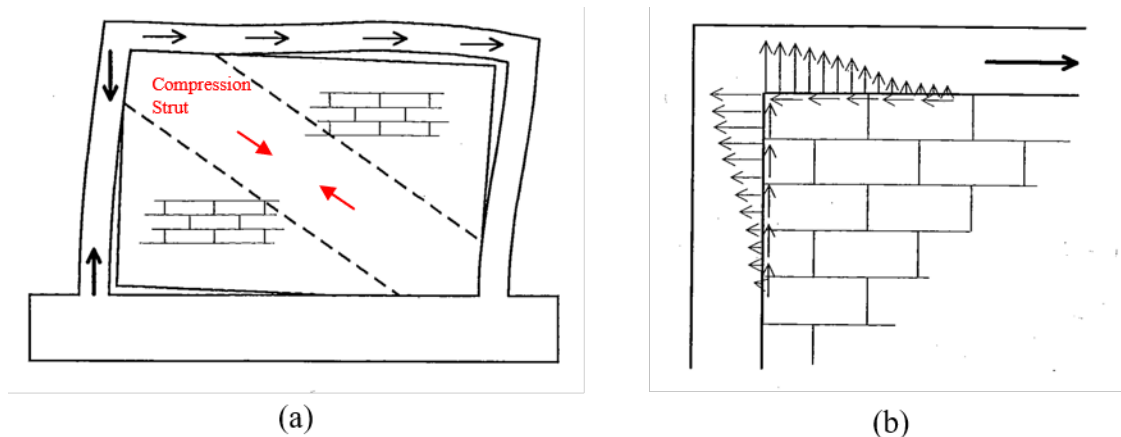


Figure 2.6 (a) Deformation and internal forces in an infilled frame after separation. (b) Normal and shear stresses acting on the loaded corner of the infill. Adapted from Crisafulli (1997).

Several in-plane failure modes for infilled RC frames undergoing seismic loads can be found in the literature based on experimental and analytical campaigns (Liauw and Kwan 1983, Liauw and Kwan 1985, Mehrabi et al. 1996, Mosalam 1996, Hashemi and Mosalam 2006). Asteris et al. (2011) summarized and classified the IP failure modes into five (see Figure 2.7):

1. Corner Crushing mode: the infill presents crushing in one or more of its loaded corners. This failure mode can occur when a weak infill wall is surrounded by a RC frame comprised of weak joints but strong beams and columns,
2. Diagonal Compression mode: the infill presents crushing in its central area. This type of failure is usually found in slender panels, where out-of-plane buckling leads to failure.
3. Sliding Shear mode: the infill presents horizontal shear slip along a plane of bed joints. Infill panels with weak mortar joints interacting with a strong surrounding RC frame present this type of failure.
4. Diagonal Cracking mode: the infill presents compressive diagonal cracks. This type of failure is usually found in strong infill panels relative to the surrounding frame elements.
5. Frame Failure mode: the surrounding RC frame presents plastic hinges in the columns or at the joints. This type of failure appears when a weak surrounding RC frame with or without weak beam-column connections interacts with a relatively strong URM infill wall.

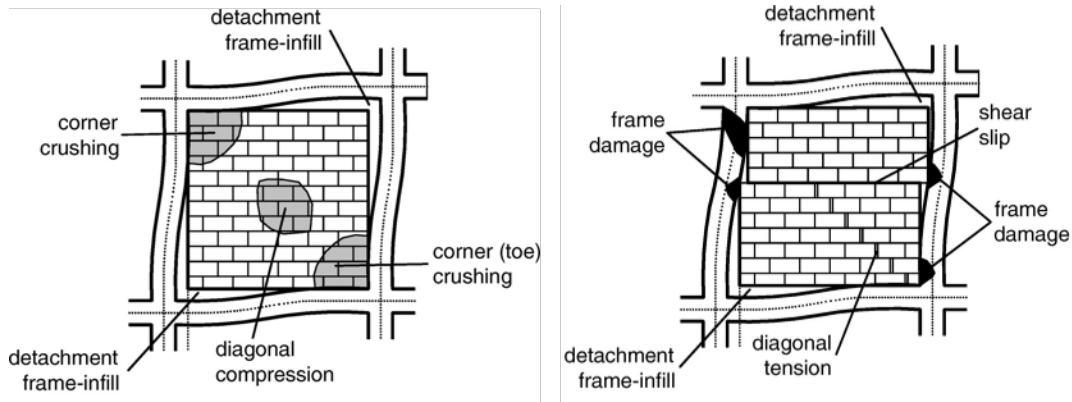


Figure 2.7 In-Plane modes of failures Adapted from Asteris et al. (2011).

2.3 Out-of-plane (OOP) behavior.

The URM infill walls are subjected to out-of-plane (OOP) actions due to seismic loading. Previous earthquakes have shown that infill walls are also prone to OOP collapses. Figure 2.8 shows two examples of infill walls that exhibited large out-of-plane displacements after the 2017 Puebla earthquake. Out-of-plane failures pose a life-safety threat since it usually results in masonry units falling. Despite the importance of the OOP behavior, the research community has not invested as many resources into its understanding relative to the extensive efforts made in the last 60 years to study the in-plane response of infilled frames.



Figure 2.8 Examples of observed out-of-plane damage of URM infill walls after the 2017 Puebla earthquake (photos by J. Archbold).

Timoshenko and Woinowsky-Krieger (1959) developed one of the first theories to estimate the OOP strength of URM infill walls. The OOP strength capacity of the panels was calculated assuming a simple bending analysis using plate theory. But as reported in Peralta (2016), the OOP

capacity of infill walls is higher than the one predicted from its flexural capacity using beam or plate theories. Other authors have attributed this higher OOP capacity to a phenomenon called *arching action*, which was first presented by McDowell et al. (1956). When cracks initiate at the end supports and midspan, two masonry blocks form. They rotate about the supported ends as rigid bodies developing high compressive forces, in a pattern that resembles an arch, hence the name. Figure 2.9 shows the compressive stress field within the infill wall after initial cracking due to OOP deformation.

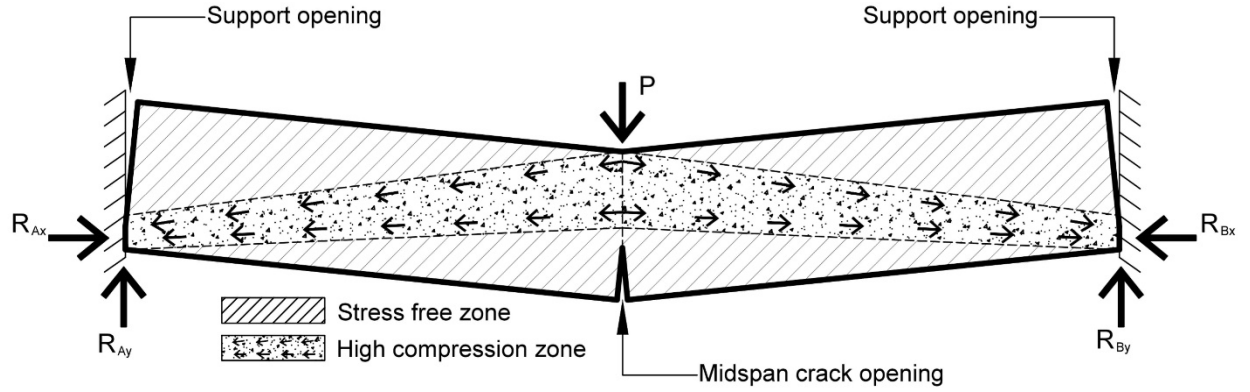


Figure 2.9. Arching action mechanism. Adapted from Peralta (2016).

Since the first attempt by McDowell et al. (1956), several researchers have tried to estimate the OOP strength of masonry infill walls, using different approaches such as numerical modeling with finite element models, empirically-based, and those based on rigid body mechanics (Liberatore et al. 2020). Di Domenico et al. (2019) presented a detailed literature review of the available formulations and models to estimate URM infill walls' OOP capacity. Table 2.1 summarizes some of the relevant equations that describe the formulation to estimate the OOP strength and the original reference.

Table 2.1. OOP formulations and models found in the literature.

Reference	Estimated OOP capacity (relevant equations)
(McDowell et al. 1956)	$q = 1.7 \frac{f_m}{(L/t)^2} \left(1 - \frac{f_m}{E\epsilon_c}\right)^2$
(ASCE 2000)	$q = \frac{0.7f'_m\lambda_2}{\left(\frac{h_{inf}}{t_{inf}}\right)} \times 144$
(ASCE 2017)	$q = \frac{0.3f'_mR_1R_2e^{-0.0985\left(\frac{h_{inf}}{t_{inf}}\right)}}{\left(\frac{h_{inf}}{t_{inf}}\right)} \times 144$

2.4 Influence of the IP/OOP interaction on the behavior.

The URM infill walls within RC frames undergo combined IP and OOP forces during seismic events. Recently, interest from the research community has focused on understanding the influence of the IP/OOP interaction on the infilled frame's behavior. The combined actions of IP and OOP forces and deformations, and the damage they cause is complicated, and it's still far from being understood, as shown in Figure 2.10.

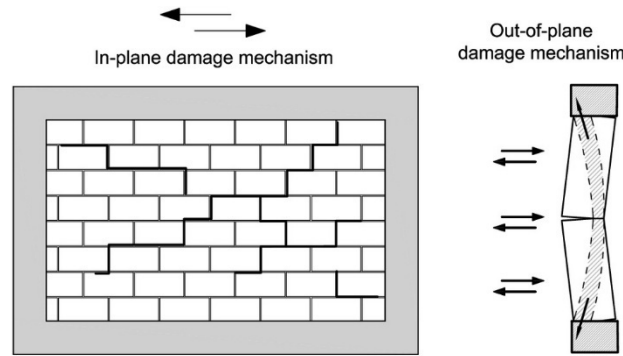


Figure 2.10 Damage mechanisms of infilled frames (Trapani et al. 2018).

Previous experimental work (Dawe and Seah 1989, Abrams et al. 1996, Flanagan and Bennett 1999, Di Domenico et al. 2021) and numerical work (Hashemi 2007) exposed the direct influence IP demands have on the OOP capacity of infill walls and vice versa. They reported that the OOP response parameters, such as the strength or deformation capacity, get reduced with IP demands and vice versa. One key finding of these studies is that the OOP strength is inversely proportional to the wall's slenderness ratio. Figure 2.11a summarizes the results of bidirectional tests of structural clay tile infilled frames performed by Flanagan and Bennett (1999). In this set of experiments, they tested a one-story one-bay infill RC frame under three loading sequence: 1) In-plane force first up to a certain level, remove the force, and then apply OOP pressure until failure. 2) OOP pressure first up to a certain level, remove the force, and then apply IP force until failure. 3) Simultaneous IP and OOP loadings. Moreover, in Figure 2.11a, it's possible to see the effect that the IP/OOP interaction has on the infill wall's response. For example, Frame 23, which was loaded simultaneously in-plane and out-of-plane direction, sees a significant reduction of its IP and OOP capacity.

Similarly, Di Domenico et al. (2021) tested three identical infilled RC frames: 1) Specimen 120S-OOP tested under pure OOP loading. 2) Specimen 120S-IPM-OOP was first tested under IP cyclic loadings up to a medium level of IP deformation and subsequently was loaded in the OOP direction. 3) Specimen 120S-IPH-OOP was first tested under IP cyclic loadings up to a high IP deformation level and later was loaded in the OOP direction. Figure 2.11b clearly shows the decrease of OOP strength and secant stiffness as the infill wall's IP demands increase.

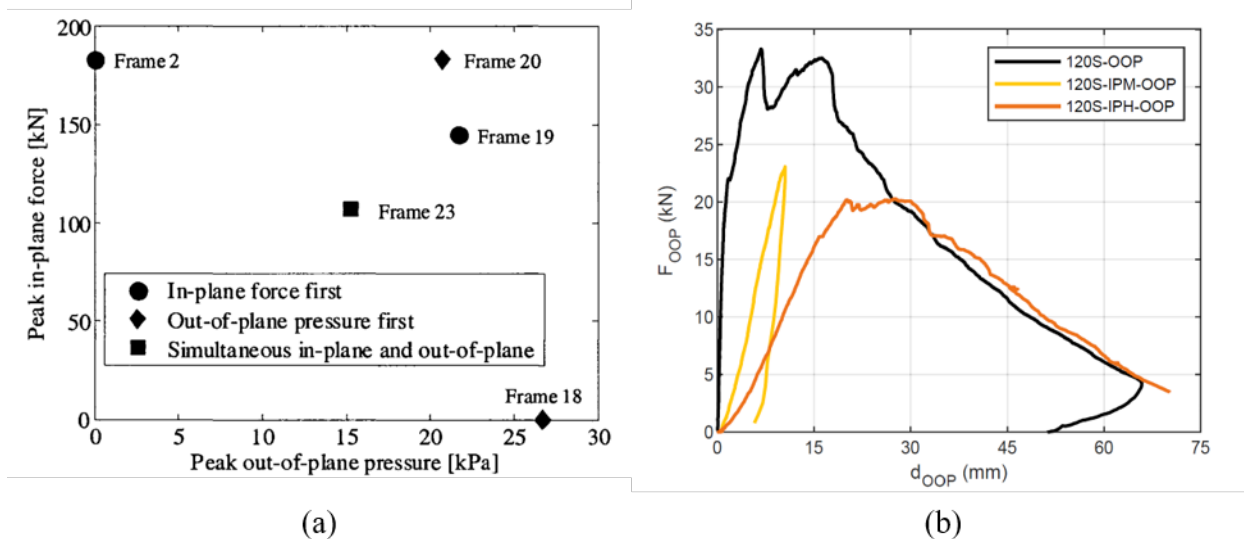


Figure 2.11 (a) Laboratory results from bidirectional tests of structural clay tile infilled frame (Flanagan and Bennett 1999). (b) Experimental results of IP/OOP interaction effects of URM infill walls (Di Domenico et al. 2021).

Limited formulations are found nowadays to account for the IP/OOP interaction. However, recently ASCE 41-17 (ASCE 2017) included Equation (2.1) to consider such interaction. The aforementioned equation is based on the work previously done by Kadysiewski and Mosalam (2009), Mosalam and Günay (2015) and is presented here:

$$\left(\frac{Q_{IP}}{Q_{IP_0}}\right)^{3/2} + \left(\frac{Q_{OOP}}{Q_{OOP_0}}\right)^{3/2} \leq 1.0 \quad (2.1)$$

Where Q_{IP} and Q_{IP_0} are the IP axial strength with and without considering the OOP actions, respectively. Similarly, Q_{OOP} and Q_{OOP_0} are the OOP strength with and without considering the IP actions.

2.5 Mathematical Modeling of URM Infill Walls

Engineers must rely on robust numerical models when experimental results are not available to perform an accurate assessment of a building's structural response to earthquake loading. Peralta (2016) suggested that these mathematical formulations should aim to represent all the physics on both the problem's demand and capacity side. However, when modeling the response of RC frames with URM infill walls, the uncertainties in all the variables that influence the response make it a cumbersome problem to solve. So, the engineering community should try to develop accurate *enough* models without compromising efficiency in computational running time.

Different modeling techniques can be implemented depending on the information at hand to model the problem and the desired computational cost. Most of the approaches in literature are based on the Finite Element Method (FEM), but other methods such as the Discrete Element Method (DEM) can also be used to assess the response of URM infill walls. This section focuses

on the former, and some approaches are herein explained in more detail. Within the continuum FEM models, three distinctions can be made based on the level of detailing used to represent the actual infill panel and its surrounding frame (Nicola et al. 2015):

1. Micromodel: masonry units and mortar joints are modeled as continuum elements. This approach usually delivers reasonable estimates of the desired quantities to be estimated. However, it is computationally expensive and requires long run times and often requires a good knowledge of the micro properties of the materials that are not always available (refer to Figure 2.12a).
2. Multi-scale models: also referred to as meso-models. This modeling technique determines the effective properties of an equivalent material to be used on a macro-scale. The effective material properties are obtained from a detailed analysis performed on a micro level or from equations that are developed based on tests or detailed micromodels. Multi-scale methods can bring down the computational cost of a model compared to those obtained when performing a more detailed micromodel. One drawback of this methodology is that it requires a good knowledge of the involved physics and consequent mathematical modeling, which often prevents practitioner engineers from moving away from more conventional modeling techniques (refer to Figure 2.12b).
3. Macromodels: the surrounding frame elements are modeled using conventional frame elements available in most FEM software packages. The infill wall response is modeled using one or more equivalent diagonal struts. This approach is the fastest and easiest to implement compared to the two previous ones. Still, with its simplicity, there is a price to be paid in the accuracy of the physics of the real problem to be considered and, therefore, the consequent structural response (refer to Figure 2.12c).

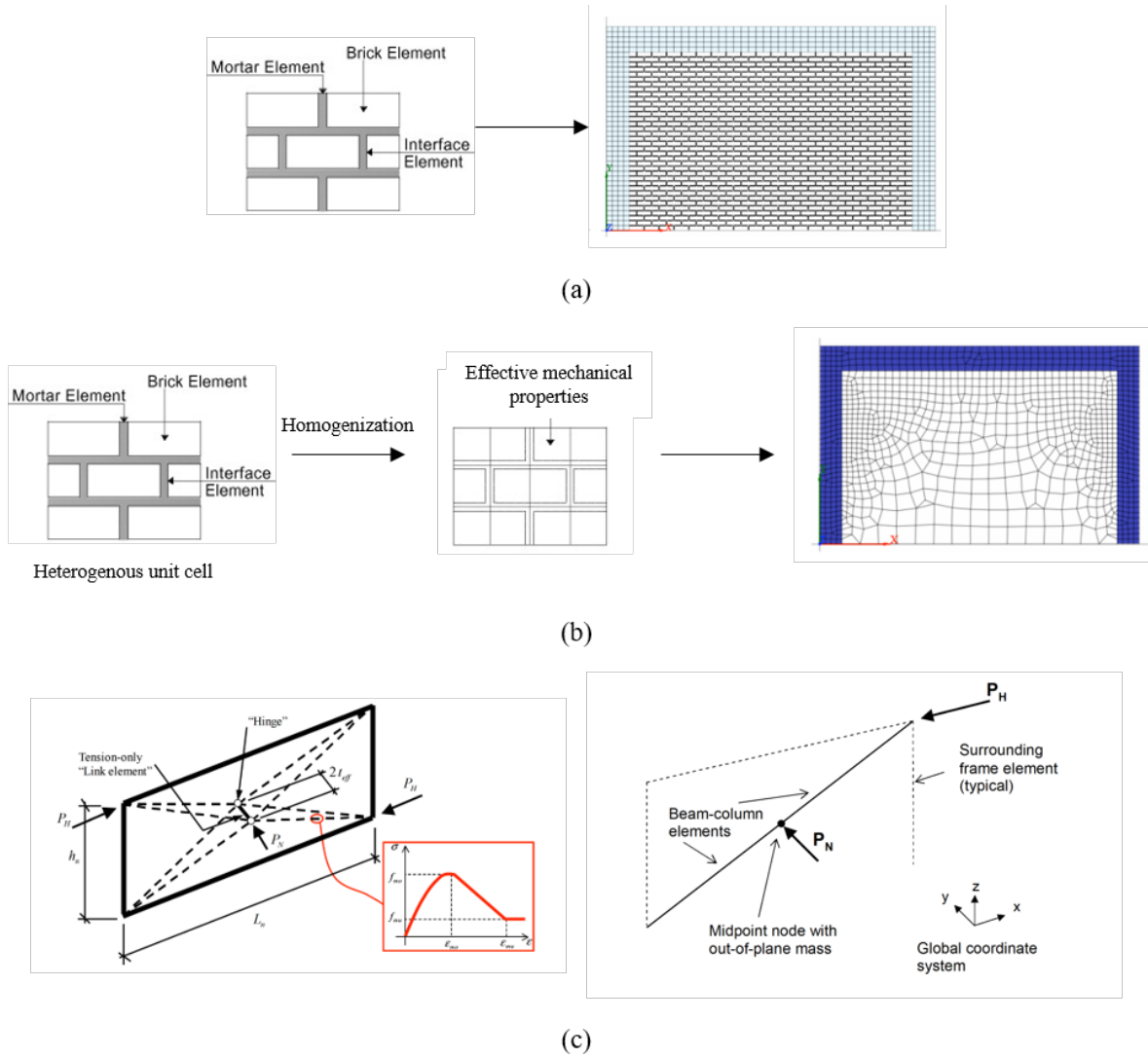


Figure 2.12. URM infill walls modeling approaches (a) Micromodel, adapted from Asteris et al. (2013), Peralta (2016) (b) Multi-scale model, adapted from Asteris et al. (2013), Peralta (2016) (c) Macro models, adapted from Hashemi and Mosalam (2006), Kadysiewski and Mosalam (2009).

3. Performance-Based Earthquake Engineering

3.1 Introduction

Towards the end of the last century, the structural engineering community learned the hard way that earthquakes moderate in magnitude could cause significant societal impacts. Even when structures designed according to the latest seismic design provisions did not collapse, the level of damage, economic losses due to repairs, and downtime were too costly (Günay and Mosalam 2013). For example, the Mw 6.7 1994 Northridge earthquake struck the suburban area of Los Angeles, and even though it was a moderate-size seismic event, it caused unprecedented losses: 57 people died, over 9000 injured, and estimated economic losses of about \$40 billion (Updike et al. 1996, Eguchi et al. 1998). A year after Northridge, on January 17, 1995, a moment magnitude 6.9 earthquake struck Kobe, Japan (USGS 1995). The 1995 Kobe earthquake resulted in more than 6000 deaths, 30000 people injured, 300000 people displaced from their homes, and economic losses greater than \$200 billion (Chung 1996). Furthermore, from 1998 to 2017, 125 million people were affected by earthquakes; and earthquakes resulted in 747234 casualties, which is 56% of the total number of deaths caused by natural disasters (Figure 3.1a). In addition, in these years earthquakes cost the global economy approximately US\$ 661 billion, becoming the second costliest type of disaster after storms, as portrayed in Figure 3.1b (UNISDR 2018).

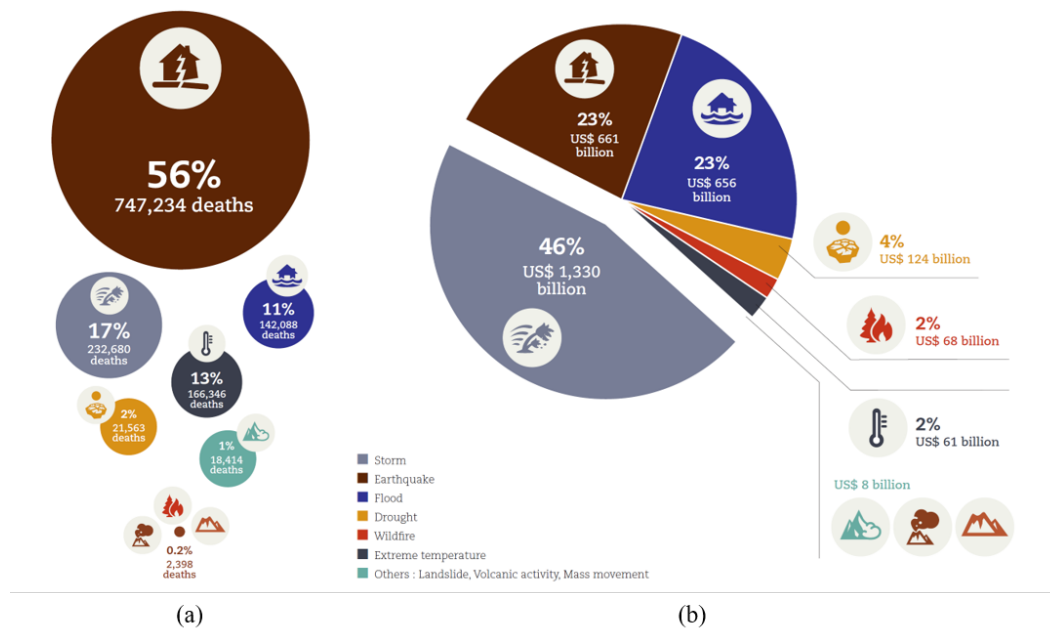


Figure 3.1 (a) Number of casualties, and (b) Economic losses by disaster type between 1998 – 2017. Adapted from UNISDR (2018).

After the 1994 Northridge and the 1995 Kobe earthquakes, the structural engineering profession realized the need for a change in the traditional code design philosophy to evaluate existing structural systems and for the design of new ones. Traditionally, building codes use a prescriptive design approach, focusing on satisfying specific design requirements, with structural

performance as a qualitative consequence of the design process. Therefore, the actual expected performance of a building designed according to current building codes does not need to be quantified. By communicating with the owner and other involved parties, an engineer may choose to do this explicit quantification, which is rarely done in practice. Additionally, the outcome of these prescriptive methods is presented in terms of variables only relevant to the structural engineers, such as forces or displacements, ignoring essential performance metrics meaningful to non-engineer decision makers (e.g., developers, building owners, policy makers) such as economic losses, downtime, deaths, or injuries.

To overcome the shortcomings of the traditional code design philosophy, the research community, with the support of local and federal agencies, developed methodologies based on a robust scientific basis to estimate quantitative levels of performance at various levels of seismic hazards (Moehle and Deierlein 2004). Due to these new efforts, the concept of performance-based earthquake engineering (PBEE) was developed and first used in the Vision 2000 report (SEAOC 1995). The interest in performance-based assessment grew with time, with the framework constantly evolving. In the early 2000s, the Pacific Earthquake Engineering Research (PEER) Center developed a second-generation PBEE methodology, referred to as the PEER PBEE framework, to assess structures subject to earthquakes in a probabilistic way and in terms of performance measures of interest to stakeholders instead of engineering parameters. The PEER PBEE methodology decomposes the seismic performance of a structure into four different analysis phases: hazard, structural, damage, and loss. Furthermore, it considers uncertainties in all the four phases and their combination, using a consistent robust formulation.

3.2 Evolution of PBEE

3.2.1 Evolution of Traditional Seismic Codes in the US

Building codes are living documents that evolve to reflect advances in technology, scientific research, and lessons learned. The evolution of building code provisions in the United States has been a slow and continuous process dated to the late 19th and early 20th centuries. In most cases, this evolution takes place as a result of a destructive earthquake. The first major earthquake that struck the US in the 20th century was *The Great 1906 San Francisco* earthquake, which caused terrible material and human losses. Although this earthquake received worldwide attention, the time was not ready for earthquake engineering to be developed (Housner 1984).

Almost twenty years later, a magnitude 6.8 earthquake occurred in the vicinity of Santa Barbara, California, on June 29, 1925. As a consequence of the Santa Barbara earthquake, in 1927, the Pacific Coast Building Official Conference, currently called the International Conference of Building Officials (ICBO), published the first Uniformed Building Code (UBC) (California Building Standards Commission 2020). Many local governments adopted the ICBO family of Uniform Codes. The UBC established uniformity of building codes in California. Among the most relevant provisions included was one that required that the structures be designed for a lateral force applied at each floor as a percentage (ranging from 7.5% to 10%) of the structure's total weight. In

1929 and 1930, the Structural Engineers of Southern California and the Structural Engineers of Northern California were created (Bozorgnia and Bertero 2004).

1933 is considered by many as the year in which Earthquake Engineering was born. On March 10, 1933, a devastating earthquake struck Long Beach's city, destroying 15 schools in the area, and damaging many more. After the Long Beach earthquake, several milestones related to earthquake engineering were achieved: 1) Two California State laws were enacted: the Field Act, which resulted in the creation of a mandatory public school retrofitting program in California, and the Riley Act, which required local governments to have building departments that issue permits for new construction and alterations to existing structures and conduct inspections (Bozorgnia and Bertero 2004, WSSPC 2010). 2) The first mandatory seismic design code used to any extent in the United States was published in 1933(Bozorgnia and Bertero 2004). 3) The city of Los Angeles required that every new building must withstand a minimum lateral force of 8% of the structure's total weight (Housner 1984).

Similarly, after the moment magnitude 6.5 San Fernando, California earthquake in 1971, 64 people died, 2543 were injured, and the estimated economic loss was \$553 million (Reich 1996). Because of this event, a new seismic provision was put in place, which required any structure to be designed taller than 160 feet to have sufficient strength to withstand the seismic effect as determined by a dynamic analysis (Housner 1984). Furthermore, the 1973 UBC added a provision that required that all concrete frames that were part of the lateral force-resisting system incorporated ductile detailing.

Many other seismic events have influenced building codes; among them, the following are worth mentioning. The M_w 6.9 Loma Prieta earthquake in 1989 led to 63 deaths, 3757 people injured, and over \$5.9 billion in damage (Borcherdt 1994). As a response to the damage observed during this seismic event, improved code provisions for wood-frame homes with crawlspace walls and wood frame soft-story buildings were included in the newer version of the UBC. The M_w 6.7 1994 Northridge earthquake, in which 57 people perished, over 9000 were injured, and losses were estimated to be \$40 billion (Utdike et al. 1996, Eguchi et al. 1998). This earthquake led to improved code provisions for welded steel frame building systems, and particularly connections.

Moreover, in 1994, the ICBO, the Building Officials and Code Administrators International (BOCA), and the Southern Building Code Congress International (SBCCI), merged into the International Code Council (ICC). The ICC was created to unify these three different agencies' efforts and develop one set of uniform standards to be used nation-wide code (International Code Council 2019). The ICC published the first edition of the International Building Code in March 2000.

3.2.2 Development of Performance-Based Guidelines

As previously mentioned, moderate-magnitude earthquakes like the ones in Northridge or Kobe in the mid-1990s led to considerable economic losses, disruption to access goods or services, a large number of casualties, injuries, and people displaced from their homes. These observations confirmed that building codes at the time were good at delivering their life-safety objective since

the number of deaths was not too high (Hamburger 1996). Still, society found the socio-economic impacts of these events too costly. This led to the initial efforts to develop performance-based design procedures, among them: Vision 2000 (SEAOC 1995), ATC-40 (ATC 1996), FEMA-273 (FEMA 1997), and FEMA-356 (ASCE 2000), often referred to as first-generation PBEE methodologies.

The Structural Engineers Association of California (SEAOC) published the report Vision 2000 (SEAOC 1995), which presented a design framework that sought to achieve the desired target system performance at different hazard levels. The report defined five performance levels for buildings: fully operational, operational, life safety, near collapse, and collapse. The performance objectives were accompanied by qualitative and quantitative damage stage descriptions for structural and non-structural components (Bozorgnia and Bertero 2004). Additionally, four hazard levels with decreasing probability were introduced: frequent, occasional, rare, and very rare. Finally, it presented four performance objectives to be decided by the owner and designer, as a desired combination of performance level and hazard level (Günay and Mosalam 2013), as shown in Figure 3.2.

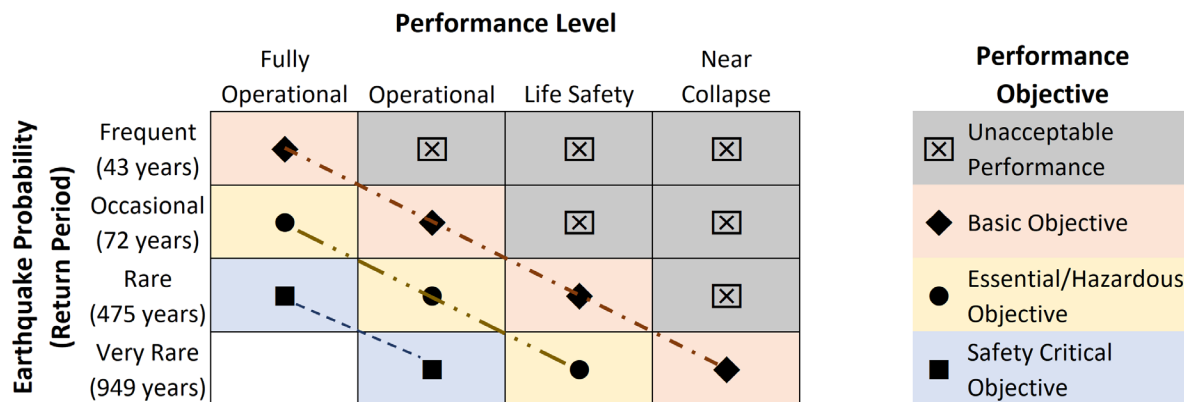


Figure 3.2 Recommended performance objectives for buildings according to Vision 2000 (SEAOC 1995).

A year later, the Applied Technology Council (ATC) published a two-volume report, ATC-40, on the Seismic Evaluation and Retrofit of Concrete Buildings (ATC 1996) based on performance-based design. As Comartin et al. (2000) stated, ATC-40 sought to speak to an audience beyond engineers. It aimed its content to building owners, building officials, architects, engineers, and any other stakeholder directly or indirectly involved with the project at hand. It introduced a simplified non-linear analysis technique based on the capacity spectrum method. Moreover, similarly to Vision 2000, ATC-40 defined performance objectives as a desired building performance level for a given earthquake ground motion. In this report, building performance levels are explicitly defined as a combination of structural and non-structural performance levels, as shown in Figure 3.3. Regarding levels of earthquake ground motion, three are presented: the serviceability earthquake (SE), the design earthquake (DE), and the maximum earthquake (ME), with return periods and probability of occurrence shown in Table 3.1.

Table 3.1 Hazard levels according to ATC-40 (ATC 1996)

Earthquake Level	Return Period (years)	Probability of Exceedance in 50 years
SE	72	50%
DE	475	10%
ME	975	5%

Building Performance Levels						
Nonstructural Performance Levels	Structural Performance Levels					
	SP-1 Immediate Occupancy	SP-2 Damage Control (Range)	SP-3 Life Safety	SP-4 Limited Safety (Range)	SP-5 Structural Stability	SP-6 Not Considered
	↓	↓	↓	↓	↓	↓
NP-A Operational →	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy →	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety →	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Hazards Reduced →	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered →	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

Legend

	Commonly referenced Building Performance Levels (SP-NP)
	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

Figure 3.3 Combination of Structural and Non-structural performance levels to form building performance levels. Adapted from ATC (1996), Comartin et al. (2000).

In parallel to the development of ATC-40, the Federal Emergency Management Agency (FEMA) published in October 1997 the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, also referred to as FEMA 273 (FEMA 1997). The performance-based framework of FEMA 273 is not conceptually different from the ones found in Vision 2000 or ATC-40. Similarly, FEMA 273 defined several levels of earthquake ground motions, performance levels, and performance objectives. FEMA 273 laid the foundations for the current seismic evaluation and rehabilitation standards ASCE 31 and ASCE 41 (Hamburger and Hooper 2011).

The American Society of Civil Engineers (ASCE) prepared the report, known as FEMA 356 (ASCE 2000), for the Federal Emergency Management Agency (FEMA) in November 2000. This report's main objective was to widen the adoption and application of the former FEMA 273 report, and it hoped to do so by converting it into a mandatory language (ASCE 2000).

As expected with any new significant endeavors, these so-called first-generation PBEE documents had a few shortcomings worth highlighting, not intending to diminish their

achievements but underscoring the need for more advanced and rigorous performance-based methodologies. (Deierlein 2004), Moehle and Deierlein (2004), Günay and Mosalam (2013) described some of these limitations: 1) Engineering demands were most based on simplified analysis techniques, which are not compatible with inelastic time-history analyses. 2) Lack of consistent associations between engineering demands and component performance; such associations were based on unreliable relations between experimental tests, numerical modeling, and expert judgment. 3) No clear connection between element performance evaluation and global system performance was mentioned. 4) Framework lacks the inclusion of uncertainties in the performance assessment. 5) Performance assessment is conducted using engineering demand parameters, which is not meaningful to a wide range of stakeholders.

3.3 PEER-PBEE Methodology Overview

To overcome the deficiencies of the traditional code design philosophy and those of the first-generation PBEE efforts, in the early 2000s, the Pacific Earthquake Engineering Research (PEER) Center developed a rigorous framework for performance-based earthquake engineering to assess performance with decision variables meaningful to all stakeholders instead of engineering parameters. This differs from the traditional design philosophy, in which the outcome is usually engineer demand parameters such as forces, displacement, drift, etc. Another critical aspect of the framework is the explicit consideration of all the sources of uncertainties in a rigorous probabilistic manner. The PEER PBEE methodology decomposes the problem into four different analyses: hazard, structural, damage, and loss. Furthermore, the result of each of the four stages is associated with an underlying random variable: Intensity Measure (IM), Engineering Demand Parameter (EDP), Damage Measures (DM), and Decision Variables (DV) respectively. Figure 3.4 depicts the underlying probabilistic framework of the PEER PBEE methodology, divided into the four phases mentioned above.

The PEER-PBEE methodology has been summarized in numerous academic publications (Porter 2003, Bozorgnia and Bertero 2004, Deierlein 2004, Moehle and Deierlein 2004, Günay and Mosalam 2013). As previously mentioned, the PEER-PBEE framework allows the assessment of a structure's expected performance in terms of variables that are of direct interest to the different stakeholders, such as the number of fatalities, injuries, economic losses, and downtime (Moehle and Deierlein 2004, Günay and Mosalam 2013). Furthermore, the methodology breaks down the problem into four phases, as explained in (Arteta and Abrahamson 2019):

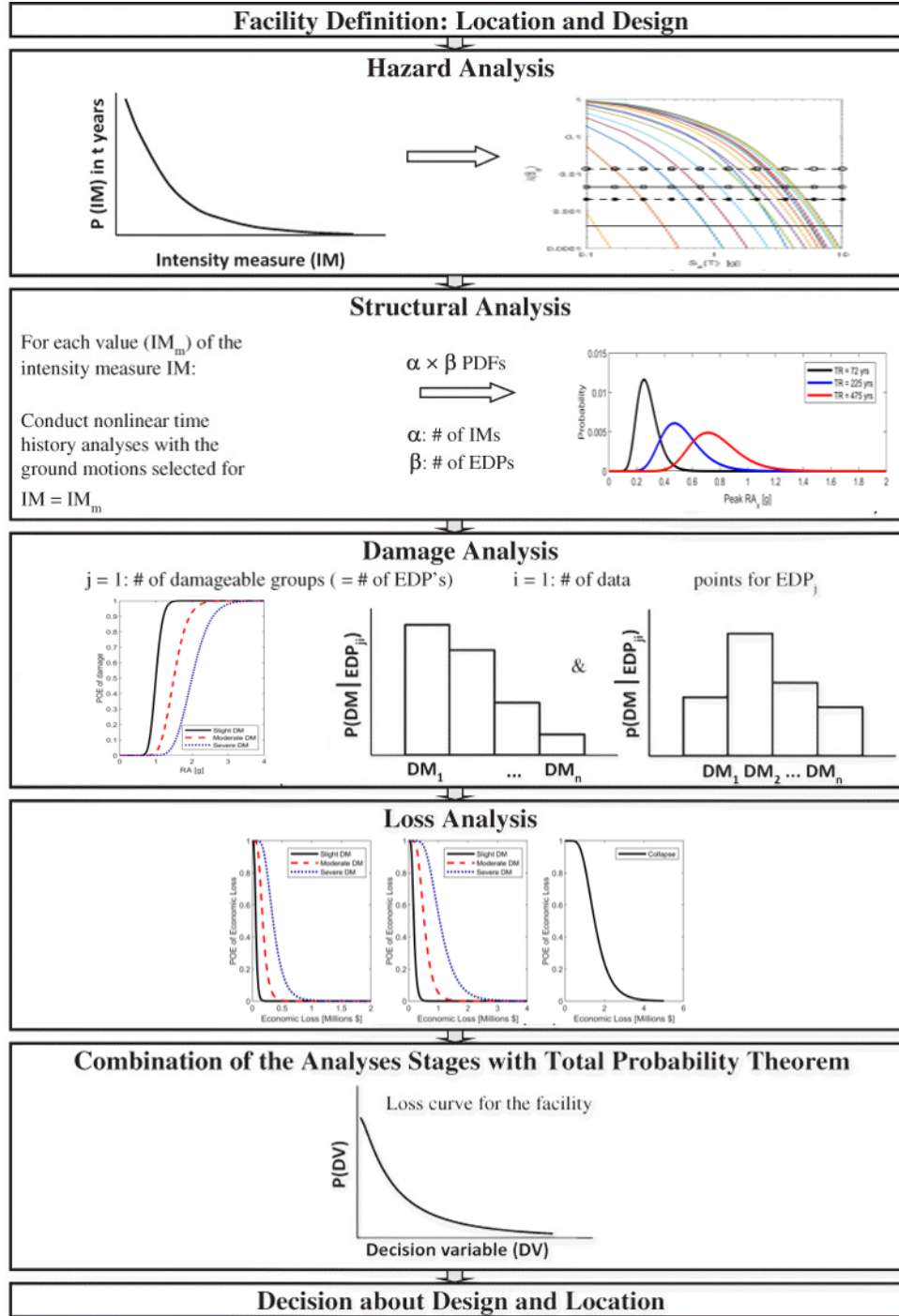
1. Hazard Analysis estimates the seismic hazard using probabilistic methods such as Probabilistic Seismic Hazard Analysis (PSHA). PSHA considers the epistemic and aleatory uncertainties in many variables like nearby faults, rate of events, source to site distance, soil conditions, etc. (Günay and Mosalam 2013). 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 structural 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 (P) of one or more *Engineering Demand Parameters (EDP)* given different IM levels, i.e., $P(EDP > edp|IM = im)$. Among the most common EDPs are Interstory Drift Ratio (IDR), floor acceleration, element forces or deformations. 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 is 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 to in the literature as fragility functions.
4. Loss Analysis is performed to establish the relationship between damage and corresponding losses defined in the form of *Decision Variables (DVs)* that are of direct interest to the stakeholders. Common DVs used are the number of fatalities, injuries, 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 consistently 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. These calculations can be done using Equation (3.1), commonly referred to as the PEER PBEE triple integral.

$$\lambda(DV) = \iiint G(DV|DM) dG(DM|EDP) dG(EDP|IM) d\lambda(IM) \quad (3.1)$$

Where G is used here to denote the conditional probability, d is the usual differentiation operator, and λ is the Mean Annual Frequency (MAF) of exceedance. This methodology assumes that the four random variables and the four analysis stages can be treated as a discrete Markov process (Heresi 2019). In other words, each conditional probability between the parameters are independent (Deierlein 2004).



$P(X|Y)$: Probability of exceedance of X given Y , $P(X)$: probability of exceedance of X , $p(X)$: probability of X

Figure 3.4 PEER PBEE framework. Adapted from Günay and Mosalam (2013).

3.3.1 Beyond the PEER PBEE framework

Since its inception, the PEER PBEE methodology has continuously grown in popularity and acceptance into academics and practicing engineers alike. In 2012, the Applied Technology Council (ATC) included this framework in their FEMA P-58: Seismic Performance Assessment of Buildings report (ATC 2012, ATC 2018). The ATC project team adopted and expanded this approach based on an updated version of the original PEER PBEE version, developed by Yang et al. (2009). This updated version uses a modified Monte Carlo simulation approach to compute the triple integral presented in Equation (3.1). Moreover, it uses simulated correlated statistical distributions of *edps* derived from a small number of non-linear time-history analyses (Yang et al. 2009, Hamburger et al. 2012).

3.4 Concluding remarks: Gap in the literature.

As previously mentioned, performance-based earthquake engineering allows engineers to go beyond structural performance. It can help us get essential insights regarding structures, such as (Heresi 2019): 1) The probability of collapse given a specific earthquake scenario or considering all possible earthquakes scenarios for a particular site of interest; 2) The probability of exceeding different levels of economic losses given an earthquake scenario or weighing all possible earthquakes scenarios for a specific location of interest; 3) The expected annual loss considering all possible earthquake scenarios for a particular site.

This dissertation seeks to use the PEER PBEE methodology to assess RC frame buildings' performance with URM infill walls in terms of economic losses. However, many gaps need to be filled when applying the PEER-PBEE methodology to RC frame buildings with URM infill walls. Because of the framework's probabilistic nature, many non-linear time-history analyses need to be conducted to get statistically meaningful results and determine the probability distribution of EDPs given different IM levels. To run many analyses, the numerical models used in the simulations need to represent the physics of the problem and capture the structure's response in an accurate manner. Yet, they also need to be efficient when it comes to computational cost. For this purpose, the macro models previously discussed in Section 2.5 seem to be the most promising modeling technique. However, experimental validation and further improvements to increase their current available model's accuracy are still lacking (Asteris et al. 2017). This gap is addressed in Chapter 6.

Regarding the methodology's damage analysis stage, reliable fragility functions for URM infill walls need to be developed. Similarly, loss functions for URM infill walls need to be determined in terms of the decision variables of interest, which are addressed in Chapters 7 and 8. Future work of this study will focus on using downtime as decision variables since these two are key to extending the concept of this methodology to city-scale problems in the frame of achieving a resilient society.

4. Case Study: Colombian Building Archetypes

This chapter describes the design of two buildings archetypes, one according to the latest Colombian seismic provisions and another one intended to represent an existing building designed and constructed around 1985, one year after the first seismic building code was implemented in Colombia. The subsequent chapters will use these archetypes as case studies of a hazard-consistent methodology to evaluate economic losses and performance through the PEER-PBEE methodology. The design of the buildings follows typical Colombian engineering practices.

4.1 4-story Ductile Archetype Building Designed to the Recent Colombian Building Code

4.1.1 Description of the Building Archetype

The case-study building is an idealized 4-story building whose lateral load-resisting system comprises reinforced concrete special moment frames and unreinforced masonry infill walls in the exterior frames. The typical story height is 2.8 m, except the first floor, which was 3.2 m, to consider the local practice of using the first floor of the 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. Figure 4.1 presents different views of the idealized structure. Two types of beams were used, one section of 40 cm x 50 cm for the perimeter beams for the second to the fourth floor, which are the ones supporting the infills, and all the other beams have a cross-section of 40 cm x 45 cm. All columns were 55 cm x 55 cm.

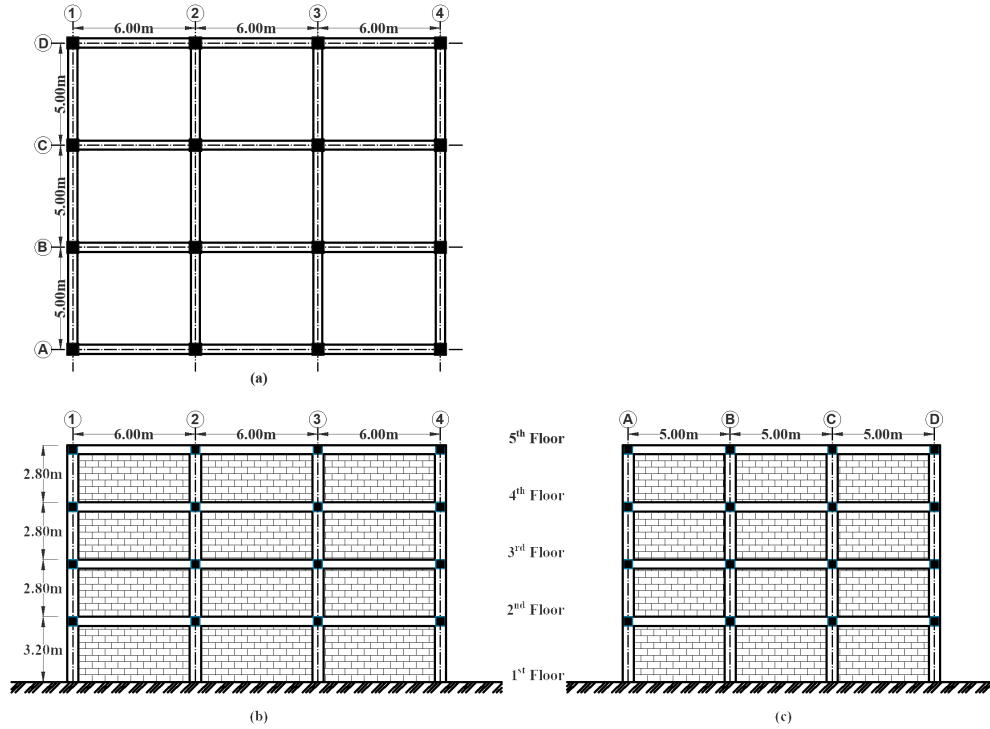


Figure 4.1 (a) Plan view of typical floor showing structural elements of the floor system and columns. (b) Elevation view of one longitudinal perimeter frame. (c) Elevation view of one transverse frame.

4.1.2 Elastic Analysis and Code-Based Design

A 4-story RC building with special moment frames and URM infills on its perimeter frames, with four moment-resisting frames along both the longitudinal and transverse direction, was designed to comply with NSR-10 (AIS 2010), which is 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 located on soil class D. The building was deemed Risk Category I, representing a normal occupancy structure. This classification allows selecting an importance factor of the structure of one (*i.e.*, $I_c = 1.0$). Moreover, the assigned response modification factor was $R = 7.0$. The obtained elastic design spectrum for the city of Armenia, Colombia, is shown in Figure 4.2.

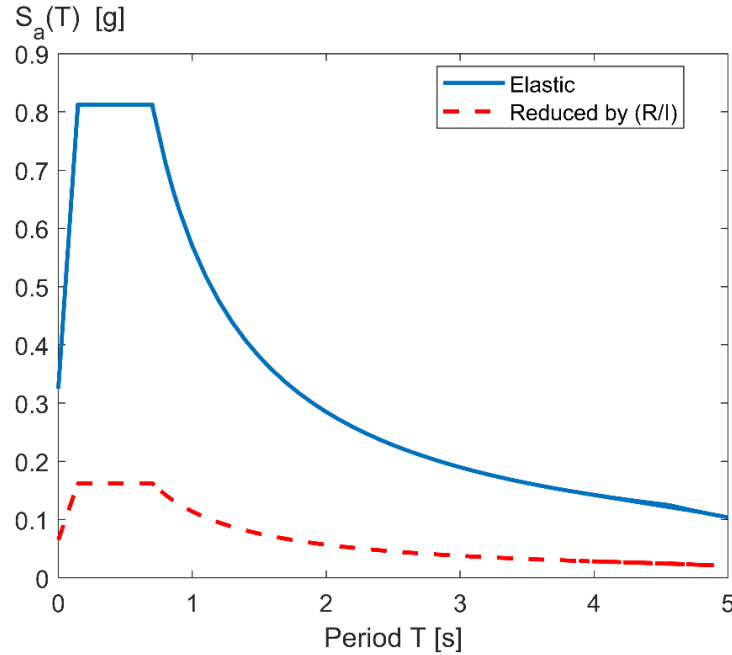


Figure 4.2 Elastic design spectrum for Armenia, Colombia.

A response-spectrum analysis was performed using the software ETABS (CSI 2019) to obtain the demands on the structural elements to design the structure. Fundamental periods based on these assumptions were 0.91 and 0.87 seconds for the longitudinal and transverse direction, respectively. These periods are longer than that which could be expected for a typical four story structure and the main reason is the long bay lengths of 5.0 m and 6.0m described previously and the cracked sections of beams and columns.

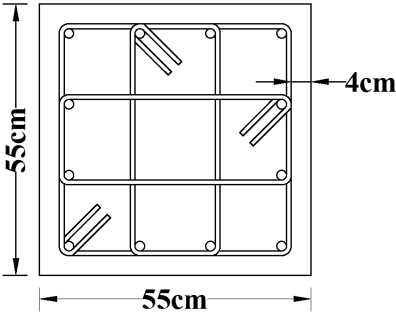
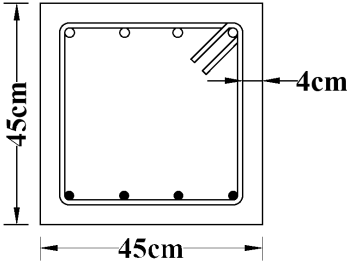
During the design stage, concrete was assumed normal weight with a unit weight (γ_c) of 24 kN/m^3 and compressive concrete strength (f'_c) of 28 MPa was used for the floor systems (beams and slab) and columns. The concrete elastic modulus was computed as $4700 \sqrt{f'_c}$ as per the code. All the steel reinforcement used in the design was assumed Grade 60 with yielding strength (f_y) of 420 MPa and an elastic modulus (E_s) of $200,000 \text{ MPa}$. To account for reduced stiffness of the structural elements due to cracked sections, the effective moments of inertia of beams and columns were set to $0.35I_g$ and $0.70I_g$, respectively. The infill walls were not considered as structural elements in the design process. Following standard practice, it was decided to only include the effects of the masonry infills as dead loads and as part of the structure's mass.

Following the latest Colombian design guidelines, the superimposed dead load and live load on the slab are assumed as 3.0 kN/m^2 and 1.8 kN/m^2 respectively. Moreover, the superimposed dead load and the mass from the structural members were lumped at every floor level.

The beam and columns cross-sections were chosen to ensure that the maximum inter-story drift ratio would not exceed 1.43% ($1.0\%/0.7$) considering cracked sections. Beams and columns were designed and detailed as special moment frames. The design process resulted in all the beams with cross-sections of $45 \text{ cm} \times 45 \text{ cm}$ with No. 3 square hoops as transverse steel reinforcement

spaced every 9.5 cm at the member ends and 19.0 cm everywhere else. Similarly, columns were 55 cm $\times$ 55 cm with No. 4 square hoops as transverse reinforcement spaced every 10 cm everywhere along their height. The beams and columns reinforcement detailing can be found in Table 4.1.

Table 4.1 Beams and columns reinforcement detailing.

Elements	Cross-section	Longitudinal reinforcement	Transverse Reinforcement
All columns		12 No. 6	No. 4 square hoops + rectangular ties @ 10 cm everywhere
All beams		Top: Four No. 6 Bottom: Four No. 5	No. 3 square hoops @ 9.5 cm near joints @ 19 cm elsewhere

4.2 6-Story Non-ductile Archetype Building

As discussed in Section 1.1, the first seismic provision in Colombia was enacted and implemented in 1984. To this day, there is still a considerable percentage of the building stock in the country that predates such first provisions. For this reason, a second archetype building is used to study the performance of non-ductile reinforced concrete frame buildings with masonry infills.

4.2.1 Description of the Building Archetype

To study the performance of midrise non-ductile buildings, a 6-story moment frame RC building with masonry infills is proposed and designed. The building was designed for low ductility following standard design practices in Colombia of the early 1980s, predating the first seismic provisions as previously discussed. The obtained elastic design spectrum, according to the 1981 standards, named Norma AIS-100-81, for the city of Armenia, Colombia, is shown in Figure 4.3.

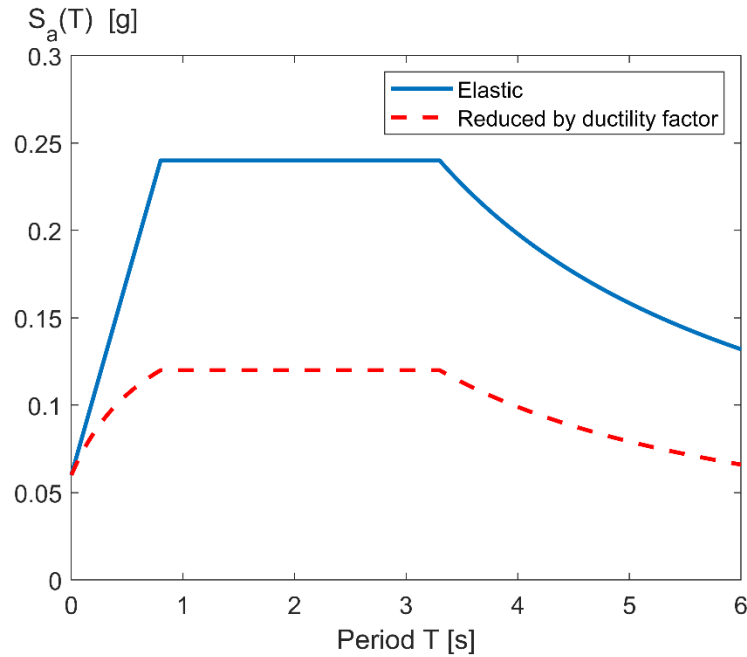


Figure 4.3 Elastic design spectrum for Armenia, Colombia according to NORMA AIS-100-81

The structural plan of the case-study buildings is depicted in Figure 4.4. It comprises four 4-bay frames in the longitudinal direction and five 3-bay frames in the transverse direction, with a span length of 6.0 m in both directions. The first story is slightly taller than the other typical stories, having a height of 3.7 m, and all the other stories are 3.2 m tall. The columns of the moment frames are connected through a two-way beam-supported slab of 0.10 m in thickness, which is considered a rigid diaphragm. For design, the concrete compressive strength was defined as $f'_c = 28$ MPa. All reinforcement was assumed Grade 60 ($f_y = 420$ MPa). All columns have a cross-section of 50 cm x 60 cm, while all beams are considered to have a cross-section of 40 cm x 40 cm.

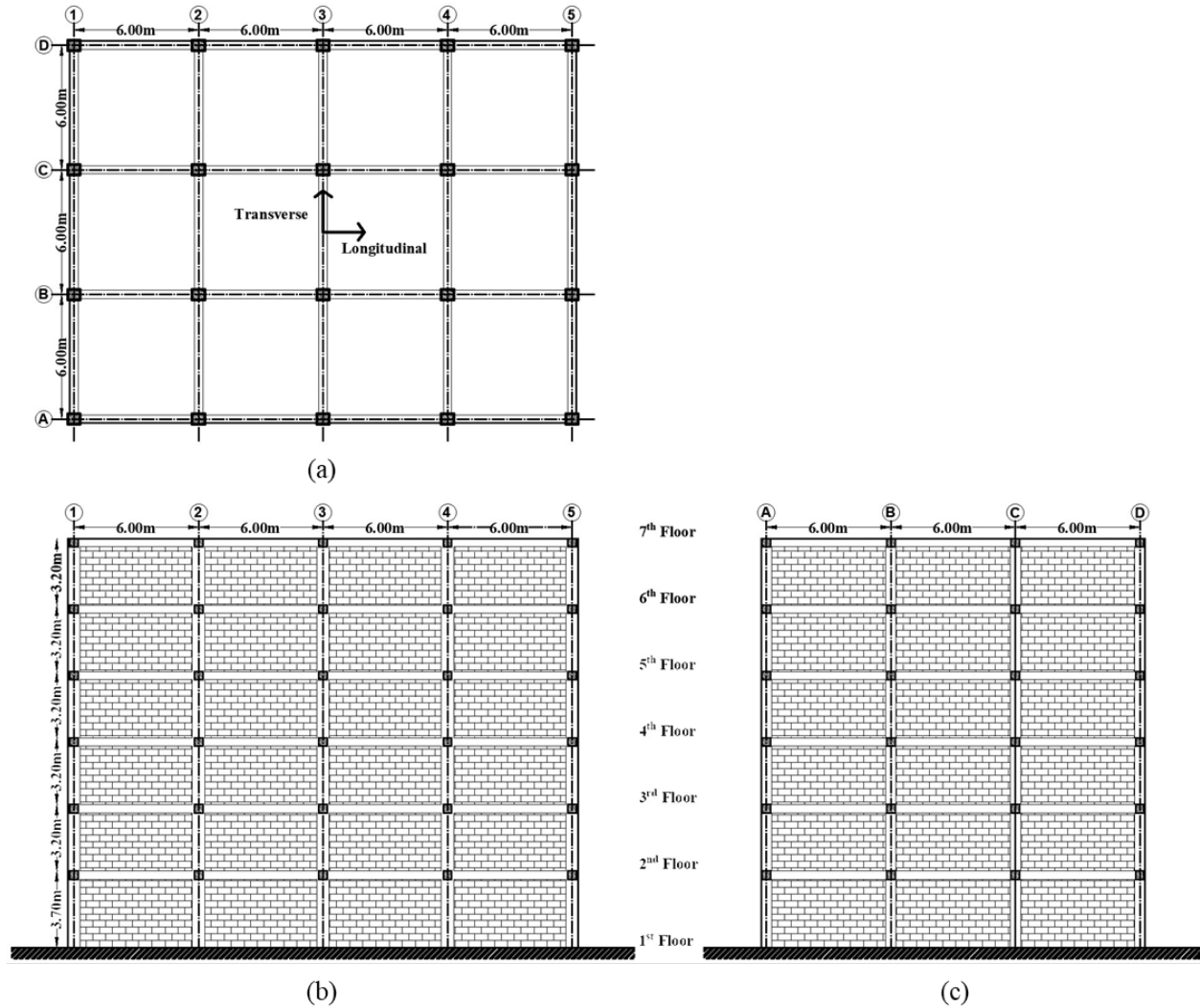


Figure 4.4 (a) Plan view of typical floor showing structural elements of the floor system and columns. (b) Elevation view of one longitudinal perimeter frame. (c) Elevation view of one transverse frame.

4.2.2 Elastic Analysis and Design

The regulations back then and now allowed engineers to use either a linear static analysis or a linear dynamic analysis to obtain the seismic forces to be used for designing the structural elements. A linear static analysis was used for the second case-study building because of their symmetry and regularity. The elastic model did not account for reduction in stiffness due to concrete cracking and reinforcement slip from connections, nor for the existence of infills. As before, the effects of the masonry infills were only considered in the model as dead loads and as part of the mass of the structure.

The total building weight per unit area for each story, including superimposed dead loads which account for the presence of infills, is 7.7 kN/m^2 , with total seismic weight $W = 24222 \text{ kN}$.

The seismic base shear V_s is estimated as the product of the seismic coefficient, c and W . Values of c range from 0.16 (for rocks and firm soils) to 0.24 (for soft soils). For the case-study, a value of $c = 0.24$ was selected. The elastic (uncracked) fundamental vibration periods for design of the bare frame building were 1.46 s and 1.35 s for the longitudinal and transverse directions, respectively. Reason of having such long periods the long bay length of 6.0 m and tall story height of 6.0m described previously and the cracked sections of beams and columns.

Concrete sections were sized and detailed to represent standard practices of the early 1980s in Colombia. The beams and columns reinforcement detailing can be found in Figure 4.5

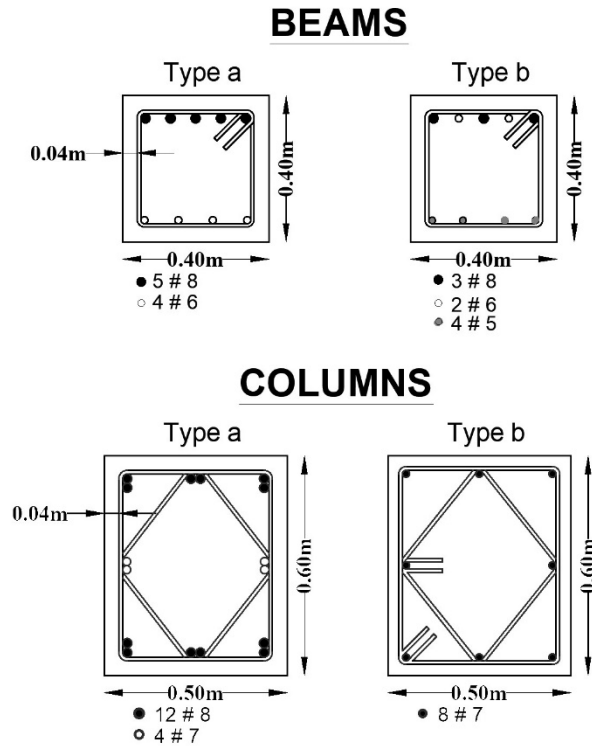


Figure 4.5 Beams and columns reinforcement detailing.

5. Hazard Analysis

This chapter describes the seismic hazard for Armenia, Colombia. Armenia was picked as the site where the case study archetype buildings are located. Armenia is situated in a high seismicity area and has suffered considerable damage due to previous moderate earthquakes.

This chapter starts with an overview of the tectonic setting of the area, and continues with a review of the main earthquakes that have struck the region. A simplified seismic hazard model is proposed and implemented using the SeismicHazard 2.0 platform developed by Candia et al. (2019). The platform is used to obtain the seismic hazard at our site of interest through classical probabilistic seismic hazard analysis (PSHA). Then, a methodology referred to as the Conditional Scenario Spectra (CSS) is used to select a set of realistic ground-motion records with rates of occurrence that reproduce the site's hazard (Arteta and Abrahamson 2019). The chapter ends with a proposed way to estimate loss curves for a given structure and site of interest (Equation 3.1) with the selected suite of ground motions and their assigned rates of occurrence that resulted from the CSS.

5.1 Tectonic summary

Colombia is the most northwestern country in South America. It's located in a complex tectonic setting resulting from three tectonic plates converging: the Nazca, the Caribbean, and South American plates (Pulido 2003). The Nazca plate converges eastward at 6 cm/year relative to the northwestern South America plate; the Caribbean plate moves 1-2 cm/year to the east-southeast relative to the South American plate, as shown in Figure 5.1 (Taboada et al. 2000, Pulido 2003). Two other critical tectonic dynamics at play are the displacement of the Panama-Choco Block towards the South American plate in the west-east direction and the displacement of the Andean block in a SSW-NNE direction (Arcila et al. 2020). The interaction between these plates shaped the three cordilleras in the Colombian Andes (i.e., Western, Central, and Eastern). Furthermore, it has led to the formation of an intricate system of active faults across the country.

According to Taboada et al. (1998), the seismic hazard in Colombia comprises earthquake scenarios with a wide range of depth, magnitude, and focal mechanisms. Moreover, earthquakes in the region occur mainly in four seismic environments: 1) Shallow crustal earthquakes in active areas of the Andean region; 2) Interface subduction earthquakes in the Pacific coast; 3) Intraslab subduction earthquakes in the Pacific from Benioff Zone; and 4) Deep subduction zone from the so-called Bucaramanga Seismic Nest (Taboada et al. 1998, Arcila et al. 2020).

One of the most seismically active regions in Colombia is the Andean Region, where Armenia is located. For the case of Armenia, the sources that contribute the most to the hazard are: 1) active crustal faults, with the Romeral Fault System (RFS) being the most important one due to its proximity to the city (Restrepo and Cowan 2000, Bonett Díaz 2003); and 2) subduction faults in the Pacific of Colombia, even though these sources are farther away compared to others like the RFS, they are still capable of generating stronger earthquakes (Bonett Díaz 2003).

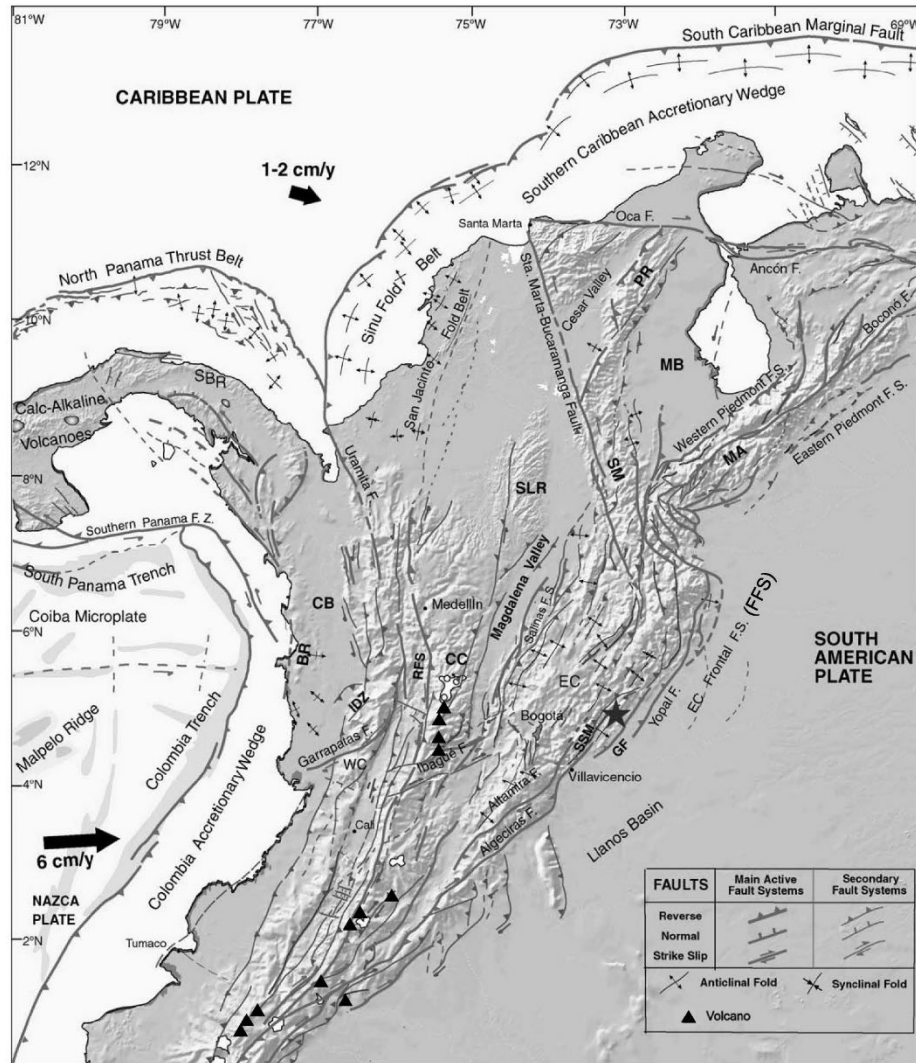


Figure 5.1 Tectonic setting of Colombia.

5.2 Historical Context of Colombian Seismicity

Colombia has witnessed numerous moderate to large-sized earthquakes in the last 400 years, including at least two earthquakes with moment magnitudes over 8.0 and many with magnitudes ranging from 6.0 – 8.0. Figure 5.2 shows the major seismic events recorded that have affected the Colombian territory. This figure's metadata was retrieved from the historical earthquake catalog maintained by the Colombian Geological Survey (or Servicio Geológico Colombiano in Spanish).

As previously mentioned, the seismicity of the Colombian coffee-growing region, where Armenia is located, is relatively high. Table 5.1 summarizes information on significant earthquakes that affected the coffee region of Colombia. The seismic events that have affected the area come from different tectonic settings, as discussed in the following section of this chapter. The seismic hazard in the region is dominated by nearby active shallow crust faults like the Romeral Faults System, followed by the contribution of earthquakes originated due to the

subduction that occurs in the Pacific region (Ramos and Monsalve 2008). These historical records suggest that subduction earthquakes with hypocenters located at intermediate depth are typically larger in magnitude. Furthermore, it shows that shallow crustal earthquakes occur less frequently and at times at lower magnitudes than those in the subduction zone. Still, due to the proximity of some of these crustal faults to urban areas, they can cause significant damage once they occurred.

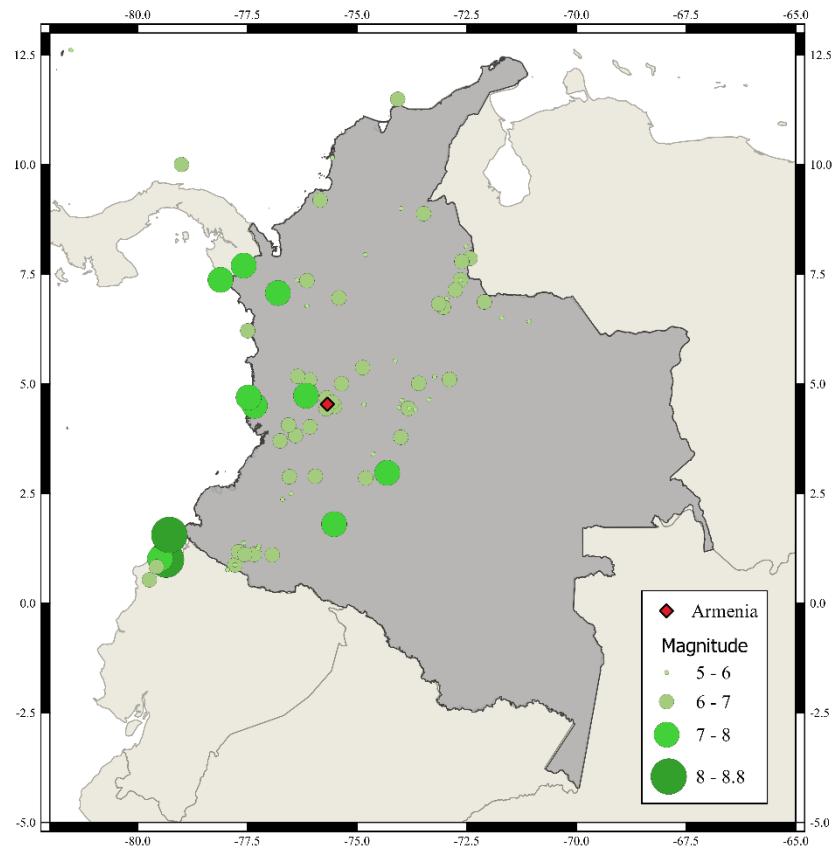


Figure 5.2 Historical Seismicity in Colombia, created with data from Servicio Geológico Colombiano (2021).

Table 5.1 Significant seismic events affecting Armenia, adapted from CIMOC and CEDERI (2002)

Date			Epicenter	Latitude	Longitude	Depth	MSK	Magnitude
Month	Day	Year		[N]	[W]	(km)	Intensity	
July	12	1785	Chingaza National Park (Cundinamarca)	4.7	73.8	30	IX	7
November	16	1827	Quebrada Santa Ana (Caqueta)	2	75.9	10	X	7.7
May	19	1875	Cucuta (Norte de Santander)	7.9	72.5	20	X	7.7
February	9	1878	Los Nevados National Park	4.8	75.5	25	VII	-
September	9	1878	Los Nevados National Park	4.8	75.5	-	VIII	-
May	25	1885	Popayan (Cauca)	2.5	76.5	20	IX	-
January	31	1906	Pacific Ocean	2.4	79.3	25	X	8.6
August	31	1917	Sumapaz Paramo (Cundinamarca)	4.3	74.2	40	IX	7.3
December	22	1923	Miraflores (Boyaca)	5.2	73.2	20	IX	-
January	10	1924	Cartago	4.7	76.1	-	VI	-
June	7	1925	Yotoco (Valle)	3.9	76.4	170	VII-VIII	-
February	4	1938	Neira - Aranzazu - Filadelfia	5.4	75.6	90	VIII	-
December	20	1961	Ansermanuevo - Balboa	4.8	76	40	VII	-
July	30	1962	Pereira - Cartago	4.8	75.7	69	VIII	-
February	9	1967	Vegalara (Huila)	2.9	74.8	26	IX	-
April	3	1973	Finlandia	4.7	75.7	146	VII	-
April	24	1973	Santa Rosa de Cabal	4.9	73.6	19	VII	-
November	23	1979	El Cairo (Valle)	4.8	76.2	105	VIII	5.5
November	29	1986	San Jose del Palmar (Choco)	5.1	76.7	90	V-VI	5.5
November	23	1990	Pijao - Roncesvalles	4.3	75.4	129	V	-
November	19	1991	San Juan River's mouth	4.5	77.5	104	IX	-
October	18	1992	Murindo (Choco)	7.1	76.8	33	X	7.3
February	8	1995	Calima (Valle)	4.1	76.7	100	VII	6.6
August	19	1995	Apia	4.6	75.4	118	VI	4.7
January	25	1999	Cordoba (Quindio)	4.3	75.6	35	VIII	6.2

5.3 Probabilistic Seismic Hazard Analysis

The primary reason for performing a seismic hazard analysis is to obtain the probabilities of various shaking intensities and to select corresponding ground motions. 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 calculated with Equation (5.1):

$$Haz(Sa(T) > z) = \sum_{i=1}^{NFLT} N_i(M_{min}) \int_{R=0}^{\infty} \int_{M_{min}}^{M_{max_i}} f_{M_i}(M) f_{R_i}(r) P(Sa > z | M, r) dM dr \quad (5.1)$$

The left-hand side of the equation represents the rate of ground motions 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. In PSHA, we estimate the possibility of strong shaking occurring at the site of interest regardless of which fault led to the ground motion or earthquake scenario on a given fault (magnitude and distance). Thus, we add the contribution from all possible scenarios on a given fault; we also sum the chances of getting a ground motion from all sources. PSHA explicitly considers uncertainties in the process, such as fault geometry and location, magnitude-recurrence rates, and ground-motion models (GMMs). Typically, a probabilistic seismic hazard analysis comprises four stages: 1) definition of the seismic sources that contribute to the hazard at the site of interest. 2) seismic source characterization. 3) selection of ground-motion models. 4) computing the seismic hazard through a probabilistic analysis. The following subsections present each of these stages for the estimation of the hazard in Armenia, Colombia.

5.4 2020 Seismic Hazard Model for Colombia

The seismic hazard estimation at a regional level in Colombia is an ongoing effort that dates back 40 years. Among the first seismic hazard maps for Colombia are the ones developed by Ramírez and Uribe (1977). In more recent history, the development of seismic hazard assessment has gone hand in hand with the development and updating of seismic building codes, led by the Seismic Engineering Association of Colombia (abbreviated AIS in Spanish). Some of these hazard model updates can be found in various reports (AIS-300 1983, 1996, 2010) published by committee 300 of AIS. These seismic hazard studies served to establish seismic design coefficients in subsequent building standards (AIS 1983, 1998, 2010). Salgado et al. (2016) proposed a revised model based on the hazard model developed for the latest seismic design standard NSR-10 (AIS 2010), and the Colombian Seismic Resistant Design Code later adopted it for Bridges (CC-14) (Arcila et al. 2019, Araujo 2021).

Last year, the Colombian Geological Survey (abbreviated SGC in Spanish) released the latest and most sophisticated seismic hazard model for Colombia yet. It was developed in partnership with the Global Earthquake Model (GEM) Foundation (Arcila et al. 2020). This model is used herein to estimate the hazard model in the site of interest, namely, the city of Armenia. The 2020 seismic hazard model for Colombia includes improvement in the seismic catalog for the region, the definition of the geometry of the sources, and their seismic characteristics; it also consists of the inherent epistemic uncertainty in the selection of GMMs by defining logic trees to be used in a respective tectonic setting (i.e., crustal, interface, intraslab) (Araujo 2021). The results of the seismic hazard study for rock sites in all major Colombian urban centers using the Arcila et al. (2020) model can be found online (Servicio Geológico Colombiano 2020).

Unfortunately, the Arcila et al. (2020) model is not yet openly available for users to perform a hazard analysis in any seismic hazard platform. For that reason, this study proposes a simplified version of it, implemented in a recently developed platform for Probabilistic Seismic Hazard Analysis, called SeismicHazard 2.0 (Candia et al. 2019). The author tries to emulate the input parameters as much as possible based on the available documentation for the model (Arcila et al.

2020, Servicio Geologico Colombiano 2020). The input parameters for the proposed model are discussed next.

5.4.1 Source Geometry Model

The initial step in the seismic source characterization process is to establish a model of the geometry of the sources (Abrahamson 2019). The Arcila et al. (2020) model proposes to separate the seismic sources that contribute to the hazard in Colombia into four tectonic environments: 1) Shallow crustal; 2) Interface subduction in the pacific coast; 3) Intralab intermediate subduction in the pacific (Benioff Zone); 4) Deep subduction zone from the so-called Bucaramanga Seismic Nest. To consider the epistemic uncertainty in the definition of the geometry of the sources and their seismicity parameters, the 2020 hazard model introduces a logic tree for the source model, as shown in Figure 5.3.

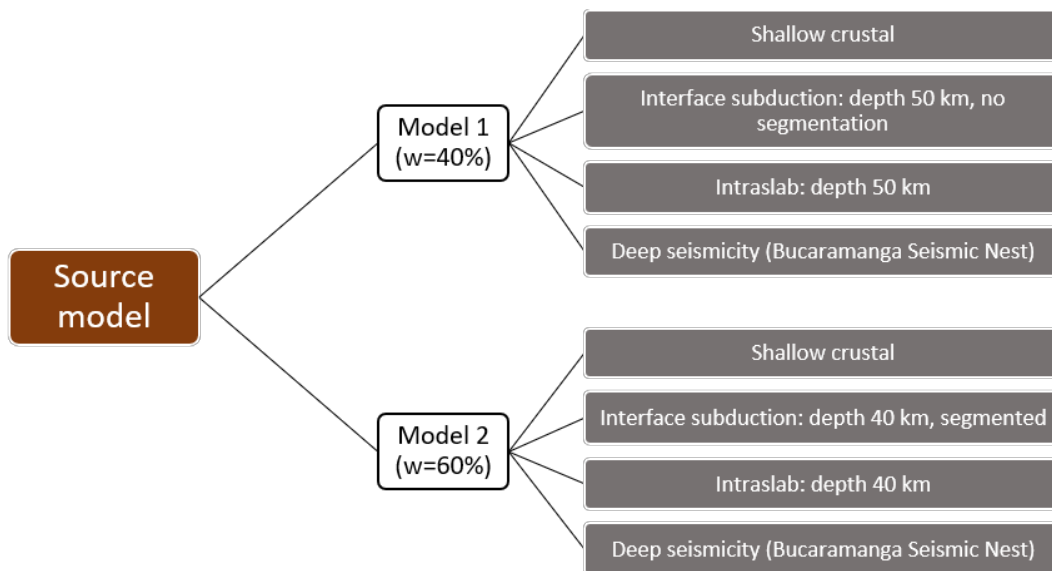


Figure 5.3 Seismic source logic tree, adapted from Arcila et al. (2020).

Starting from the information obtained from previous seismic hazard models for Colombia (AIS-300 1983, 1996, 2010), the 2020 seismic hazard model developed by Arcila et al. (2020) models the sources from the different tectonic environment, as areal sources, described by 3D polygons that account for its plan location, depth and dip angle (Araujo 2021).

The simplified version of the seismic hazard model implemented in this dissertation only uses one branch of the seismic source logic tree, namely Model 2 in Figure 5.3. Furthermore, it does not include the deep seismicity sources from the Bucaramanga Seismic Nest. Their contribution was negligible to the hazard at the site of interest, so the author decided not to model it altogether. Figure 5.4 presents the geometry of the different seismic sources considered in the model separated by the tectonic environment. A more detailed discussion on the modeling of these seismic sources can be found in the original documentation of the model (Arcila et al. 2020, Servicio Geologico Colombiano 2020).

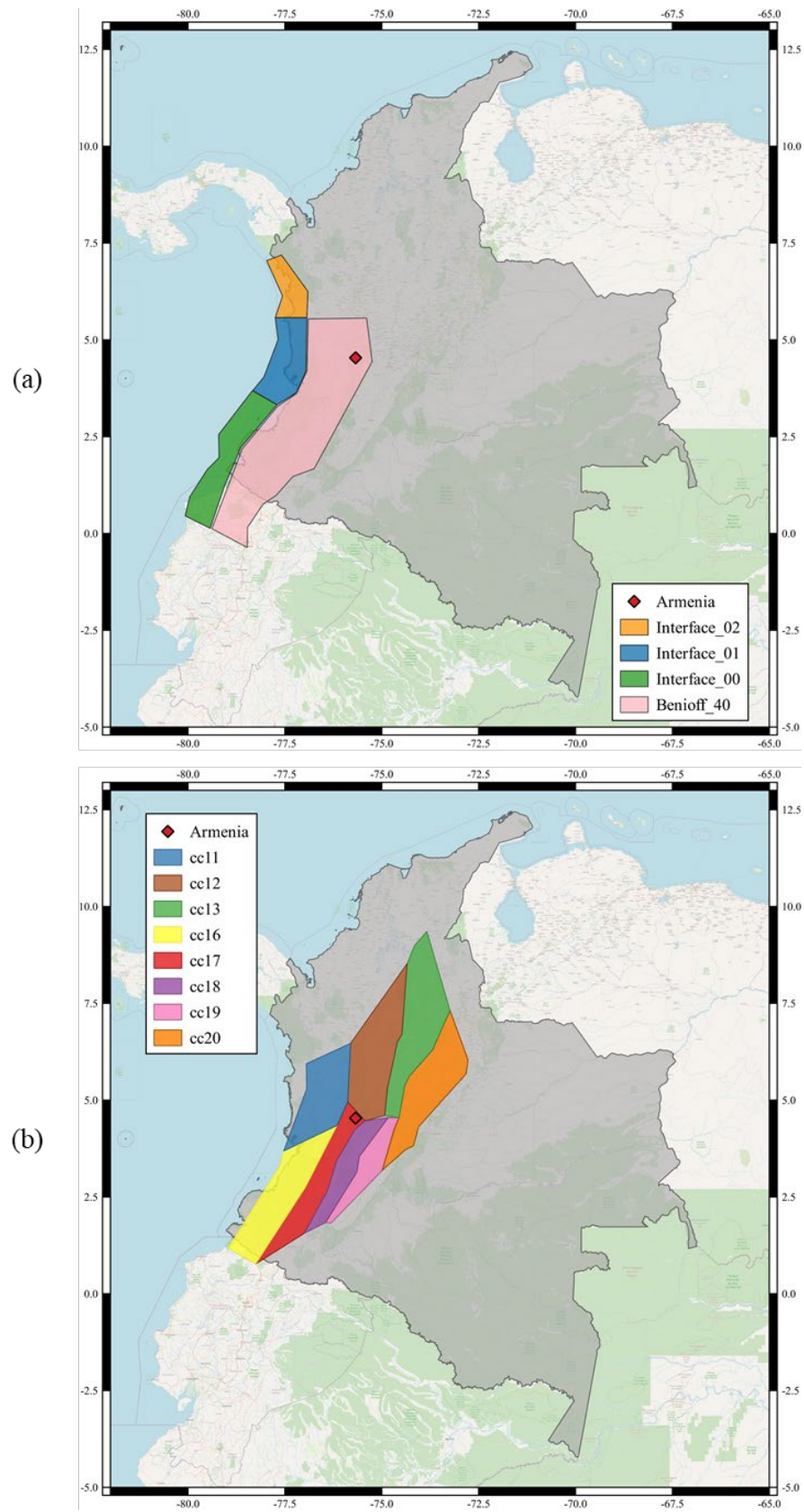


Figure 5.4 Seismic sources: (a) Subduction: interface and intraslab. (b) Shallow crustal.

5.4.2 Source Parameters

According to Abrahamson (2019), seismic source characterization aims to obtain models that describe the rate at which earthquakes of a given magnitude and dimension occur at a given site. Namely, it will produce a set of credible and relevant earthquake scenarios for each seismic source and their respective rate at which they occur. This stage of the process includes models for the probability distribution of earthquake magnitude, rupture dimensions, locations of the earthquakes for a given rupture dimension, and their corresponding rates at which they occur on the source (Abrahamson 2019).

Arcila et al. (2020) modeled the magnitude distribution, which describes the relative number of large, moderate, and small magnitude earthquakes that a source can generate, using a truncated exponential model. This model is based on the Gutenberg-Richter magnitude relation. According to the Gutenberg-Richter relation, the total rate of earthquakes with a magnitude greater than M is given in Equation (5.2). The parameters a and b are estimated using the maximum-likelihood method using available data related to the earthquake sources (Table 5.2).

$$\log_{10} N_m(M) = a - bM \quad (5.2)$$

Furthermore, the annual rate of exceedance of earthquakes with magnitude greater than M $\lambda(M > m)$ is given by Equation (5.3), where m_{min} and m_{max} are the magnitude values where the distribution is truncated at.

$$\lambda(M > m) = \lambda(M > m_{min}) \frac{10^{-b m} - 10^{-b m_{max}}}{10^{-b m_{min}} - 10^{-b m_{max}}} \quad (5.3)$$

In this model, an earthquake's occurrence is considered to follow a Poisson process, which means that the probability of an earthquake happening in a given year does not depend on previous events. Table 5.2 presents the seismicity parameters used for the different sources contributing to the hazard at the site of interest in Armenia, Colombia.

Table 5.2 Seismic Source Parameters.

<i>Source</i>		m_{min}	m_{max}	a_{value}	b_{value}
<i>Mechanism</i>	<i>Tag</i>				
Interface	Interface_02	6.0	7.8	2.91	0.689399
	Interface_01	6.0	8.0	2.78	0.71739
	Interface_00	6.0	8.7	2.68	0.635628
Intraslab	Benioff_40	6.0	7.5	3.13	0.65
Crustal	cc11	5.0	6.5	4.23	1.02
	cc12	5.0	6.5	3.66	0.92
	cc13	5.0	6.5	3.35	0.78
	cc16	5.0	6.8	2.82	0.74
	cc17	5.0	6.8	2.05	0.61
	cc18	5.0	7.1	2.55	0.7
	cc19	5.0	6.5	3.65	1.09
	cc20	5.0	6.8	3.14	0.83

5.4.3 Ground Motion Models (GMMs)

A ground motion model (GMM) allows estimating, at a given site, the level of ground shaking and the associated variability in terms of an intensity measure (e.g., Sa, PGA) as a function of variables like earthquake magnitude, source-to-site distance, soil effects, fault mechanism, and many others (PEER 2021).

In the 2020 Seismic Hazard Model for Colombia developed by Arcila et al. (2020), the SGC (Colombian Geological Survey) defined logic trees for the GMMs used in the three different tectonic environments (i.e., crustal, interface, intraslab). The selected GMMs and their respective weights are based on an extensive earthquake catalog recorded in the region of interest (Araujo 2021). Table 5.3, Table 5.4, and Table 5.5 summarize the logic trees used for the GMMs for the interface, intraslab, and crustal sources, respectively.

Table 5.3 GMMs logic tree for interface sources.

GMMs	Weight	Reference
Zhao2006	0.348	(Zhao et al. 2006)
BCHydro2012	0.437	(Abrahamson et al. 2016)
MontalvaBastias2017	0.215	(Montalva et al. 2017)

Table 5.4 GMMs logic tree for intraslab sources.

GMMs	Weight	Reference
Zhao2006	0.211	(Zhao et al. 2006)
BCHydro2012	0.365	(Abrahamson et al. 2016)
MontalvaBastias2017	0.424	(Montalva et al. 2017)

Table 5.5 GMMs logic tree for crustal sources.

GMMs	Weight	Reference
Cauzzi2014	0.389	(Cauzzi et al. 2015)
ASK2014	0.212	(Abrahamson et al. 2014)
I2014	0.399	(Idriss 2014)

5.4.4 PSHA Results

The seismic hazard in Armenia was calculated using the SeismicHazard 2.0 platform (Candia et al. 2019), which allows us to efficiently compute Equation (5.1) provided that all the seismic source properties, including the geometry and the associated GMMs, are given as input parameters, as discussed in the previous sections. Spectral acceleration (S_a) was chosen as the IM of interest. As previously mentioned, the only results available online for the Arcila et al. (2020) model are for rock sites. Thus, initially, the simplified seismic source model for Armenia is used, assuming the site of interest to be on firm rock (i.e., $V_{s30} = 760$ m/s) to determine whether the proposed model yields reliable results compared to the Arcila et al. (2020) model. Hazard curves for four different periods are presented in Figure 5.5. Uniform Hazard Spectra (UHS) are constructed from hazard curves for different return periods and shown in Figure 5.6, along with the UHS obtained by the Arcila et al. (2020) team. In both Figure 5.5 and Figure 5.6, the solid lines are used for the

simplified model developed herein, and the dashed lines are used for the results in the 2020 SGC hazard model.

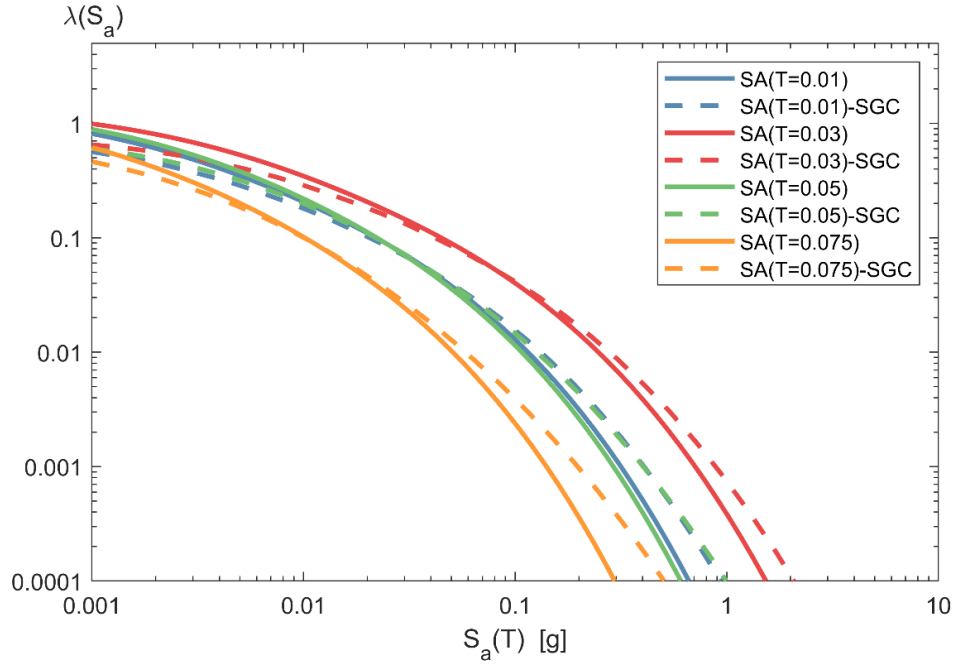


Figure 5.5 Comparison between the hazard curves predicted by the simplified model and the Arcila et al. (2020) model assuming firm rock site conditions for the city of Armenia.

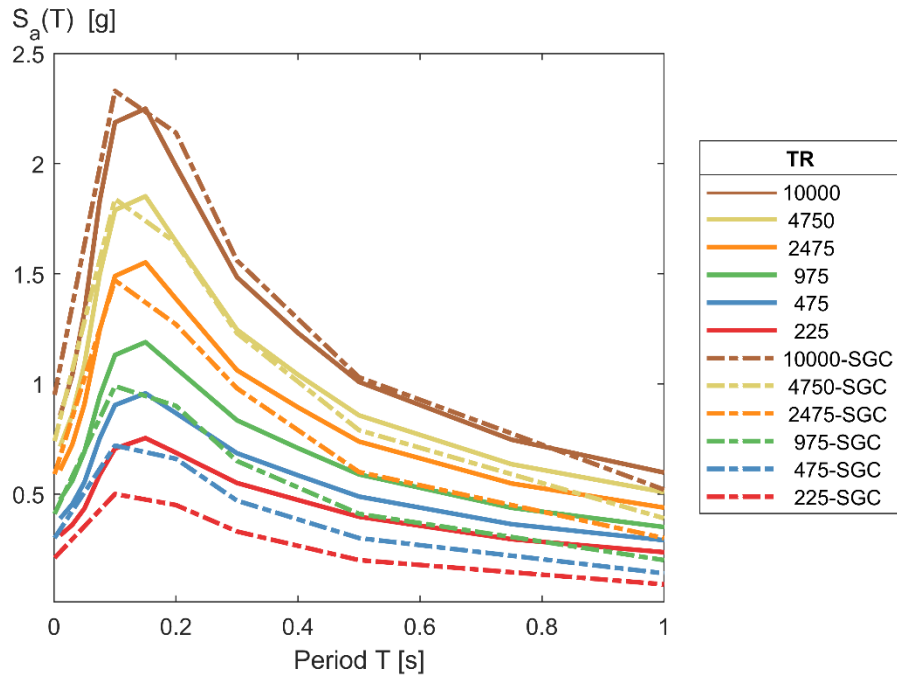


Figure 5.6 Comparison between the UHS predicted by the simplified model and the Arcila et al. (2020) model assuming firm rock site conditions for the city of Armenia (TR: return period).

The simplified model doesn't yield a perfect match to the SGC model, especially at lower hazard levels. Still, it does do a good job recovering the UHS at higher hazard levels (i.e., above $TR > 2475$ years), as shown in Figure 5.6; furthermore, even for the lower hazard levels, the results are in the same order of magnitude. The results herein are considered satisfactory, keeping in mind that the ultimate objective of the current dissertation is to develop a Performance-Based Framework for estimating the seismic assessment of reinforced concrete frame buildings with unreinforced masonry infill walls and not the development of better hazard models for Colombia. The author is interested in improving the implementation of the simplified hazard model as part of his future work.

Once the simplified hazard model has been deemed acceptable, a second calculation is run to estimate the hazard at the site of interest using the soil condition for which the archetype building was designed, namely for soil class D, with V_{s30} in the range $180 - 360 \text{ m/s}$. Hazard curves for structural periods ranging from 0.01 s to 5.0 s are presented in Figure 5.7 for the city of Armenia, Colombia, considering stiff soil conditions (i.e., $V_{s30} = 300 \text{ m/s}$). Uniform Hazard Spectra (UHS) are constructed from these hazard curves by selecting a hazard level (see horizontal dashed black lines in Figure 5.7), the intersecting pair of (T, S_a) with a given horizontal line are then plotted in a T vs S_a graph. Nine different UHS are shown in Figure 5.8 for Return Periods (TR) ranging from 100 to 100,000 years. An additional outcome of a PSHA is what's known as hazard deaggregation, which allows determining the earthquake scenarios that dominate the hazard at a given return period. Figure 5.9 shows the hazard deaggregation for Armenia at $T = 0.5 \text{ s}$, which is close to the first mode period of the 4-story building as explained in Chapter 6, for two different hazard levels. Finally, the UHS, along with the deaggregation information of the hazard, is used to select ground motions that are consistent with the seismic hazard of the site of interest. The following section will discuss a methodology to choose a realistic set of ground-motions spectra that reproduce the hazard at a site.

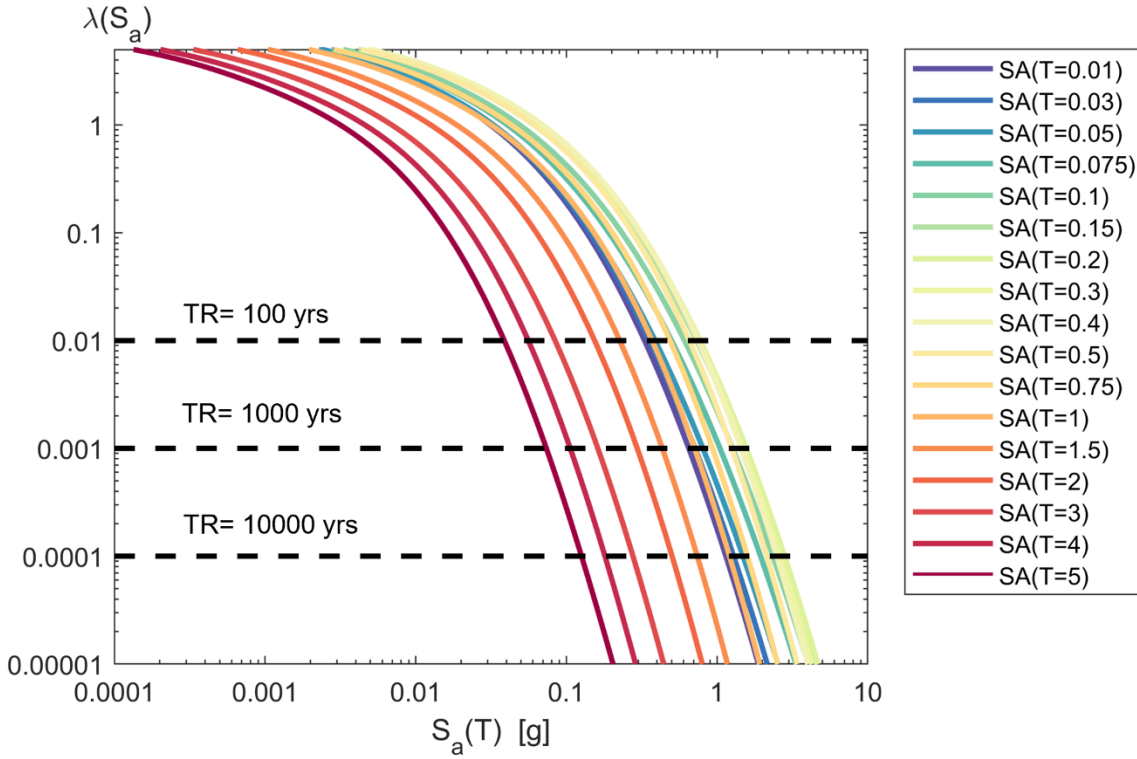


Figure 5.7 Hazard Curves for Armenia, Colombia ($V_s = 300$ m/s)

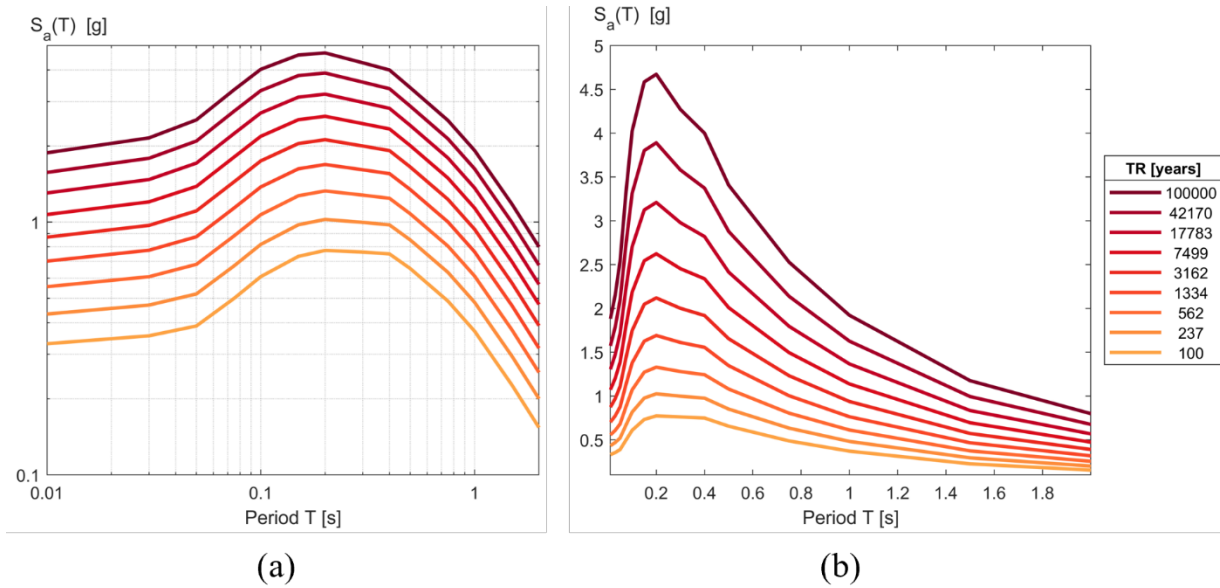


Figure 5.8 UHS for Armenia Colombia for multiple return periods (TR) for soil type class D ($V_s = 300$ m/s) in (a) a logarithmic scale and (b) in a linear scale plot.

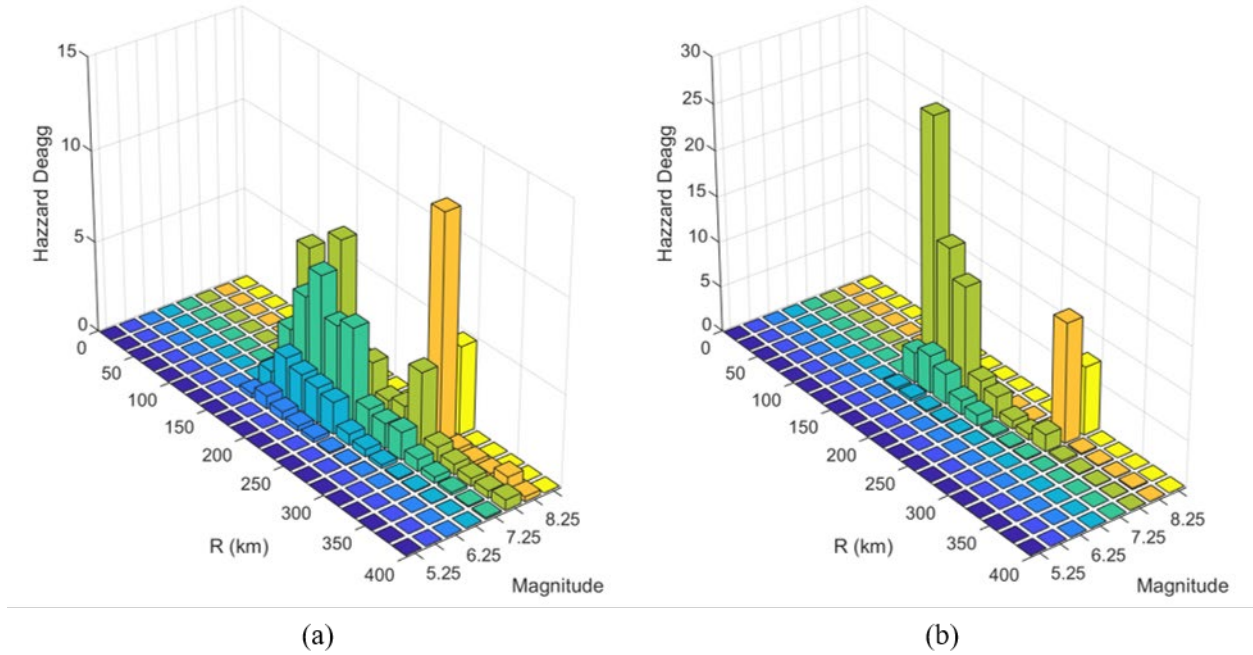


Figure 5.9 Deaggregation of the seismic hazard for $T = 0.5$ s at Armenia, Colombia for return periods: (a) 475 years. (b) 2475 years.

5.4.5 Conditional Scenario Spectra (CSS)

The CSS methodology allows selecting ground motions and subsequent assessment of the seismic performance of a given structure in a hazard-consistent approach. Moreover, the Conditional Scenario Spectra (CSS) are a set of realistic ground-motion spectra with assigned rates of occurrence that reproduce the hazard at a site of interest over a range of hazard levels (return periods) and a range of structural periods (Arteta and Abrahamson 2019). With this methodology, one can link the simulated seismic performance with the expected occurrence rate of a given ground motion to get a hazard-consistent estimate of the mean annual frequency (MAF) of exceedance of a given decision variable (DV). As Araujo (2021) explained, this MAF of exceedance is estimated using a large set of ground motion records chosen to cover a range of intensities. Then, rates of occurrences are numerically optimized and assigned to each time history based on their spectral shape and intensity.

Arteta and Abrahamson (2019) give a clear explanation of how to calculate the Conditional Scenario Spectra. A summary of the procedure for estimating the CSS is as follows: first, candidate time histories are selected based on the hazard deaggregation at the site. Next, UHS are obtained for a range of hazard levels of interest. Then, using the midpoint between consecutive UHS, Conditional Mean Spectra (CMS) (Baker 2011) are estimated; additionally, in this step, the variability of the CMS is computed as described by Jayaram et al. (2011). To this end, the correlation coefficients ρ need to be estimated. The correlation coefficients from Abrahamson et al. (2016) and Baker and Jayaram (2008) were used for subduction and crustal earthquakes, respectively. Afterward, subsets of time histories are selected for each hazard level based on their

spectral shapes consistent with both the CMS and their variability. Next, the chosen ground motion records are scaled to the CMS at the given hazard level and assigned an initial rate of occurrence. Ultimately, the rates of the time histories are adjusted to best fit the hazard in the full range of spectral periods and hazard levels considered.

Once the set of time histories and their respective optimized rates of occurrence ($Rate_{CSS,i}$) are estimated the hazard recovered with the CSS can be computed with Equation (5.4), where $S_{a,i}(T)$ is the spectral acceleration of the i -th time history, and H is the Heaviside function.

$$Haz(Sa(T) > z) = \sum_{i=1}^n Rate_{CSS,i} H(S_{a,i}(T) - z) \quad (5.4)$$

The CSS estimated herein for the city of Armenia was calculated using the SeismicHazard 2.0 platform (Candia et al. 2019). Nine hazard levels were specified to estimate target UHS with rates of exceedance ranging from 10^{-2} to 10^{-5} . The hazard levels were discretized using log spacing, as suggested by Arteta and Abrahamson (2019). These hazard levels represent the hazard levels of interest for the performance-based calucluations in future chapters. Table 5.6 shows the hazard levels used to estimate the CSS and their rate of exceedances.

Table 5.6 Selected hazard levels to estimate the CSS.

Haz Level	Rate of Exceedance	TR (yrs)
1	1.00E-02	100
2	4.22E-03	237
3	1.78E-03	562
4	7.50E-04	1334
5	3.16E-04	3162
6	1.33E-04	7499
7	5.62E-05	17783
8	2.37E-05	42170
9	1.00E-05	100000

The resulting CSS is a set of 350 two-component time histories, composed of 59 time histories from interface subduction sources, 97 from intraslab subduction sources, and 194 from crustal sources. The 350 scenario geo-mean spectra (5% damped) are presented in Figure 5.10, grouped by the tectonic environment. Subduction time-histories were selected using the NGA Subduction Database Preliminary Ground Motion Suite (Kishida et al. 2020), and Crustal time-histories were chosen from the PEER NGA-West2 Database (Ancheta et al. 2014). The metadata for the Conditional Scenario Spectra obtained can be found in Appendix A.

Furthermore, the time-histories selected are assigned rates of occurrences that reproduce the hazard previously obtained using PSHA. Figure 5.11 shows the hazard curves recovered from the CSS by source and total hazard. It compares it with the hazard curves for the site of interest using traditional PSHA for four different structural periods. Comparing the recovered hazard curves from the CSS and the target hazard curves yields good agreement, as expected.

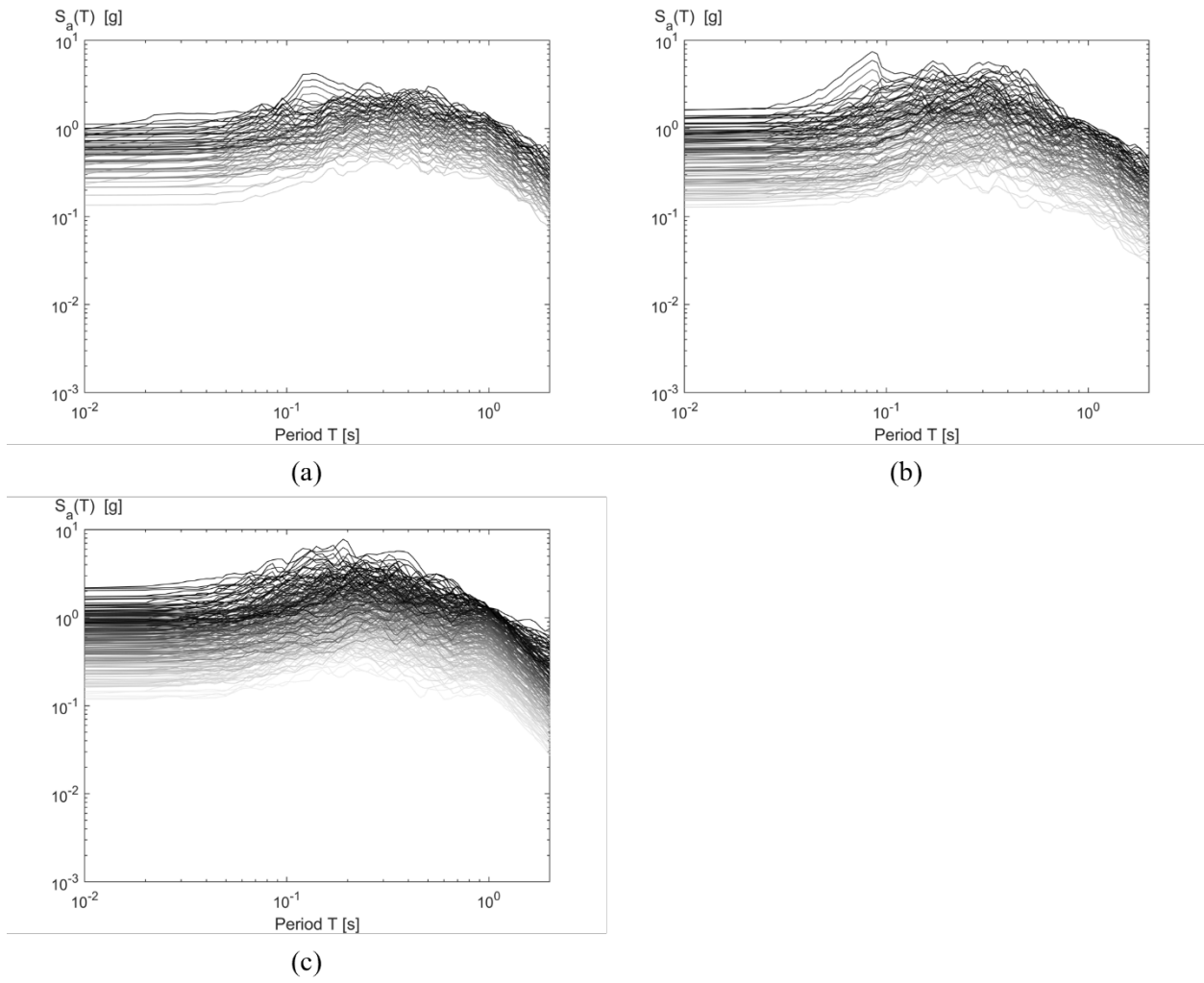


Figure 5.10 CSS for Armenia according to source type: (a) Interface. (b) Intralab. (c) Crustal.

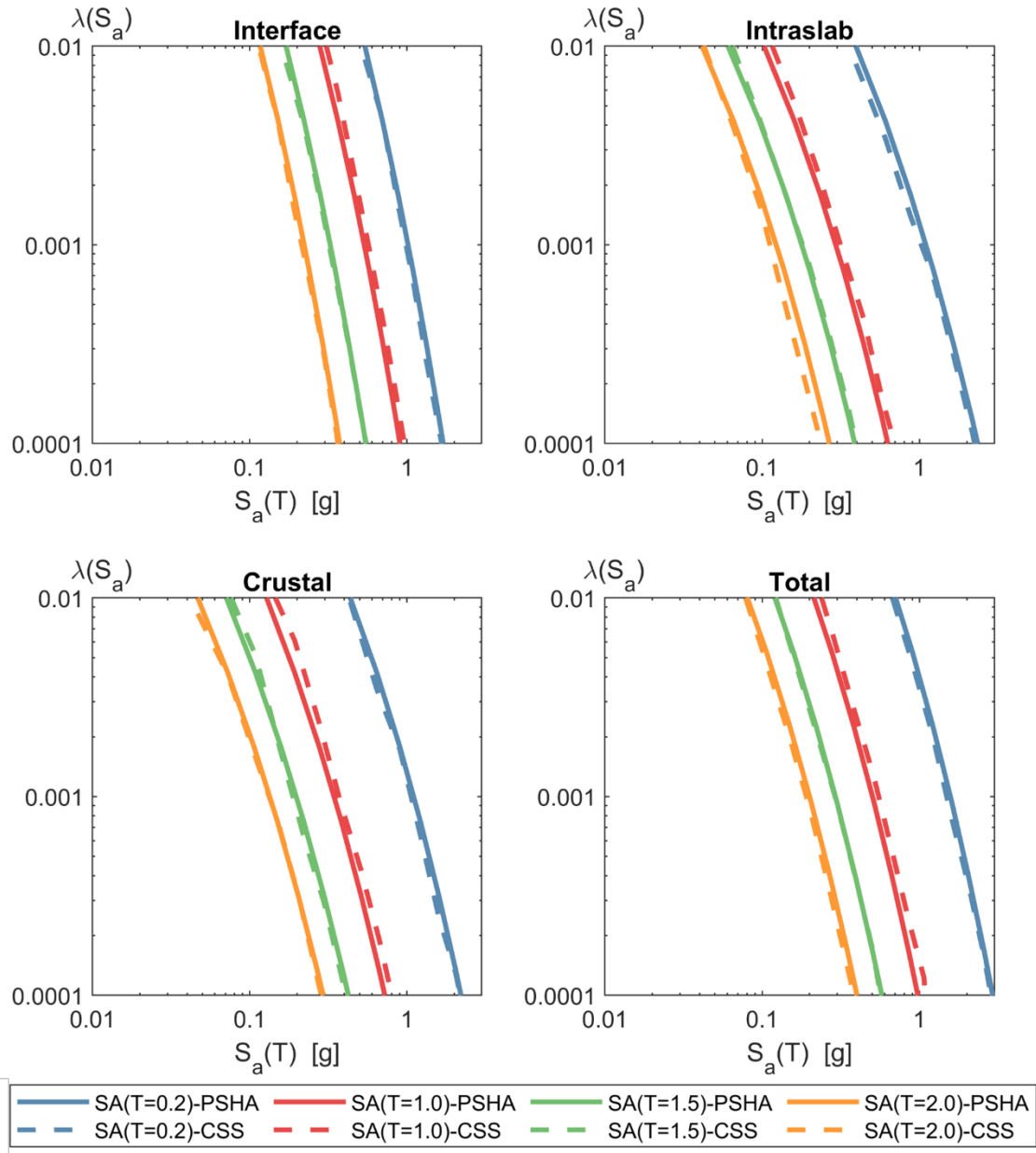


Figure 5.11 Hazard curves at different periods recovered from the CSS.

5.4.6 Hazard Consistent EDP-Risk and PBEE Analysis

As discussed in section 3.3, the PEER-PBEE methodology allows assessing a structure's expected performance in terms of decision variables (DVs) that are of direct interest to the different stakeholders (Moehle and Deierlein 2004, Günay and Mosalam 2013). This can be done using the total probability theorem according to Equation (3.1), herein repeated for convenience:

$$\lambda(DV > dv) = \iiint G(DV|DM) dG(DM|EDP) dG(EDP|IM) d\lambda(IM) \quad (3.1)$$

where $\lambda(DV > dv)$ is defined as the MAF of exceedance of the decision variable of interest, $d\lambda(IM)$ is the rate of occurrence of the intensity measure of choice, which is the slope of the hazard curve, and the remaining variables were previously defined in section 3.3.

In addition to combining all four phases, combining a smaller number of phases might also lead to meaningful results of interest. For example, one might be interested in knowing the annual rate of exceedance of a given EDP, which is the inverse of the return period of a given engineering response level (Arteta and Abrahamson 2019). Similarl to how Equation (3.1) was obtained, the total probability theorem can be used to combine the hazard and structural analyses as follows:

$$\lambda(EDP > edp) = \int G(EDP > edp | IM = im) |d\lambda(IM)| \quad (5.5)$$

where $\lambda(EDP > edp)$ is defined as the MAF of exceedance of the EDP of interest, and $d\lambda(IM)$ is the rate of occurrence of the intensity measure of choice, which is the slope of the hazard curve. Some authors call plots of $G(EDP > edp | IM = im)$ vs. im fragility curves or functions, this work will call these plots system-level fragility functions; to ensure the distinction with component-level fragility functions that will be discussed later in Chapter 7.

We can make use of the outputs of the CSS methodology, to estimate $\lambda(EDP > edp)$ in a hazard-consistent approach as follows:

$$\lambda(EDP > edp) = \sum_{i=1}^n Rate_{CSS,i} H(EDP - edp) \quad (5.6)$$

where $Rate_{CSS,i}$ are the assigned rates of the i -th time history, and H is the Heaviside function, which will be 1 or 0, if the demand level due to the i -th time history exceeds edp .

Similarly, the annual rate of exceedance of a decision variable can be directly estimated with help of the CSS as follows:

$$\lambda(DV > dv) = \sum_{i=1}^n Rate_{CSS,i} H(DV - dv) \quad (5.7)$$

where $Rate_{CSS,i}$ are the assigned rates of the i -th time history, and H is the Heaviside function, which will be 1 or 0, if the decision variable exceeds a certain threshold dv due to the i -th time history.

6. Structural Analysis

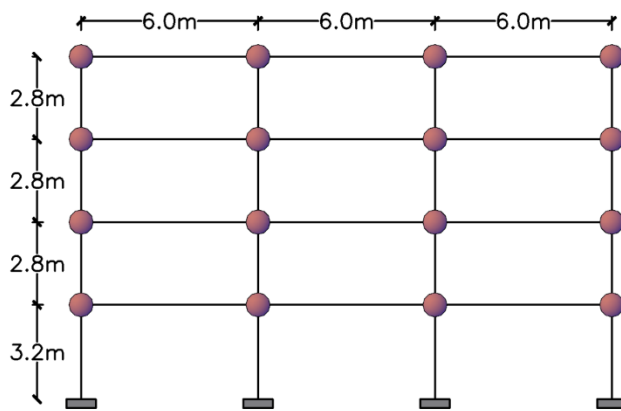
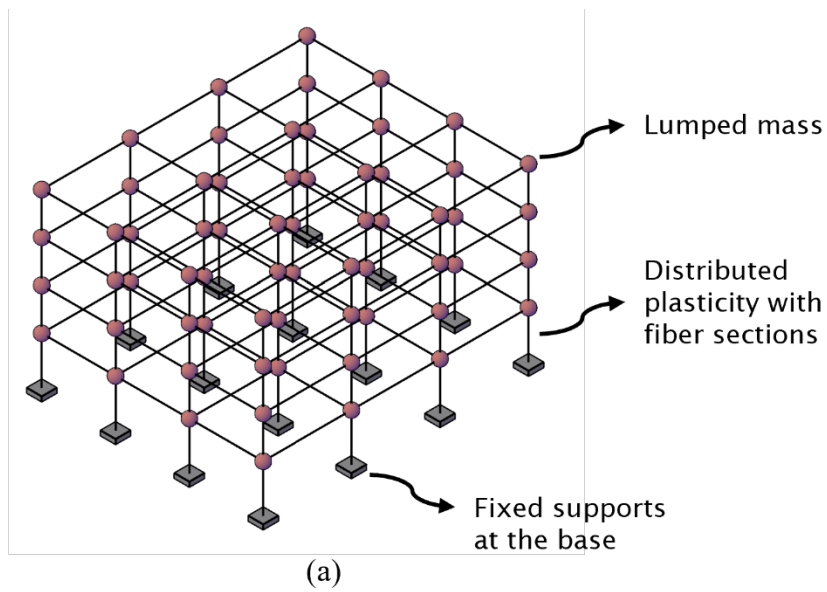
6.1 Nonlinear Model for Seismic Performance Assessment

The behavior of the two archetype buildings under realistic suites of ground motions that represent the hazard at the site was assessed by means of Nonlinear Time History (NLTH) analyses. The numerical models were developed using the software framework OpenSees (McKenna et al. 2010).

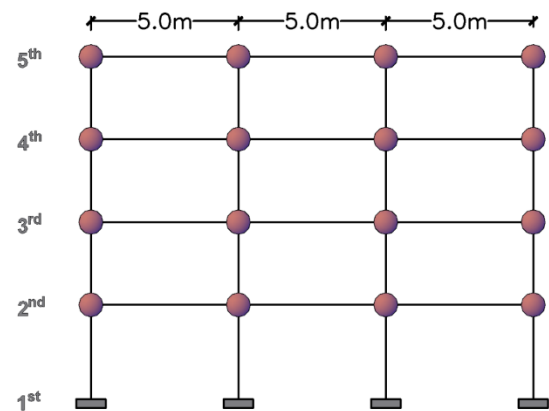
6.1.1 Modeling of the Moment Resisting Frame

Beams and columns were modeled using standard nonlinear elements with distributed plasticity and fiber sections at the integration points (Spacone et al. 1996). Second-order P- Δ effects were considered in columns. Figure 6.1 shows the configuration of the 3D nonlinear model implemented in OpenSees for the moment-resisting frames. Column shear damage due to forces transferred from the URM infill walls was modeled using bilinear shear springs in OpenSees through zero-length elements,. Concrete was modeled as either confined or unconfined, depending on the location within the cross-section, using the uniaxial material *Concrete02* available in OpenSees. Linear softening post-peak branch of the concrete model was regularized according to Coleman and Spacone (2001) to enforce objectivity in the local and global post-peak response. Fracture energy of concrete in compression (Jansen and Shah 1997) was considered as 25 N/mm and 150 N/mm for unconfined and confined concrete, respectively. Tensile concrete strength was assumed to be $0.62\sqrt{f'_c}$ (MPa), with a strain-softening branch adjusted to fracture energy of 0.1 N/mm (Wittmann et al. 1988).

The tensile and compressive behavior of reinforcing steel was modeled using the uniaxial *Hysteretic* material implemented in OpenSees. This material allows modeling the asymmetry of the tensile and compressive behavior that reinforcing steel undergoes due to bar buckling when subjected to lateral loading. Material parameters to model the cyclic response of the steel were set to simulate a trilinear backbone curve using Mazzoni et al. (2007) recommendations. The stress-strain relationships used for concrete and reinforcing steel are depicted in Figure 6.2.



(b)



(c)

Figure 6.1 Configuration of nonlinear model of the moment-resisting frames: (a) 3D of the model. (b) Elevation view of a typical longitudinal frame. (c) Elevation view of a typical transverse frame.

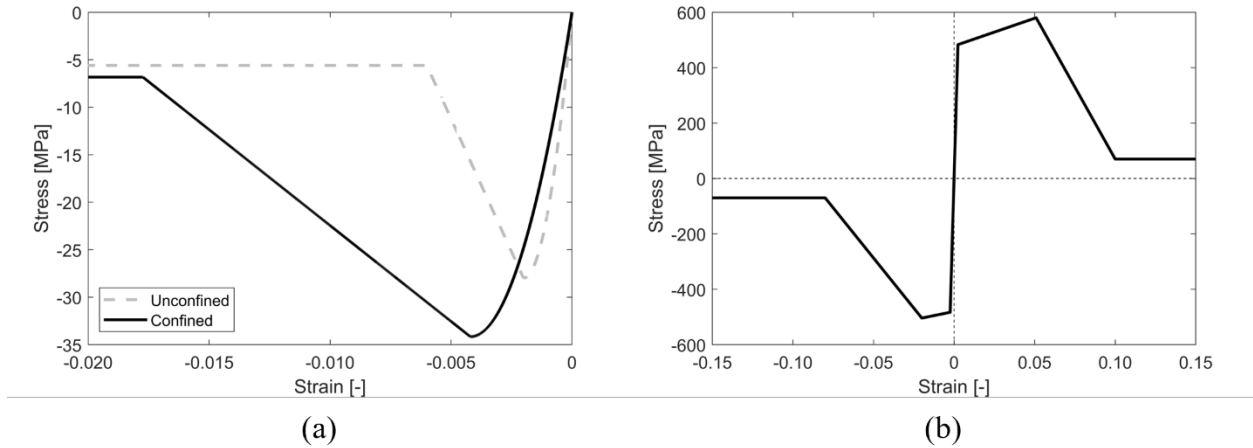


Figure 6.2 Stress-strain relationship for (a) concrete, (b) reinforcing steel.

6.1.2 Modeling of the Unreinforced Masonry Infill (URM) Walls

The URM infill walls are modeled using a revised formulation of a numerical model previously proposed by Kadysiewski and Mosalam (2009) and later expanded by Mosalam and Günay (2015) to include an element removal algorithm. In their formulation, each infill wall is modeled by a single diagonal strut. Each diagonal includes two beam-column elements, namely the force-based “beamWithHinges” elements, and their cross-section is modeled using fiber sections. The two beam-column elements are connected by an Out-of-Plane (OOP) lumped mass at the midpoint of the diagonal strut. Figure 6.3 shows the idealized infill wall as implemented in OpenSees. 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.

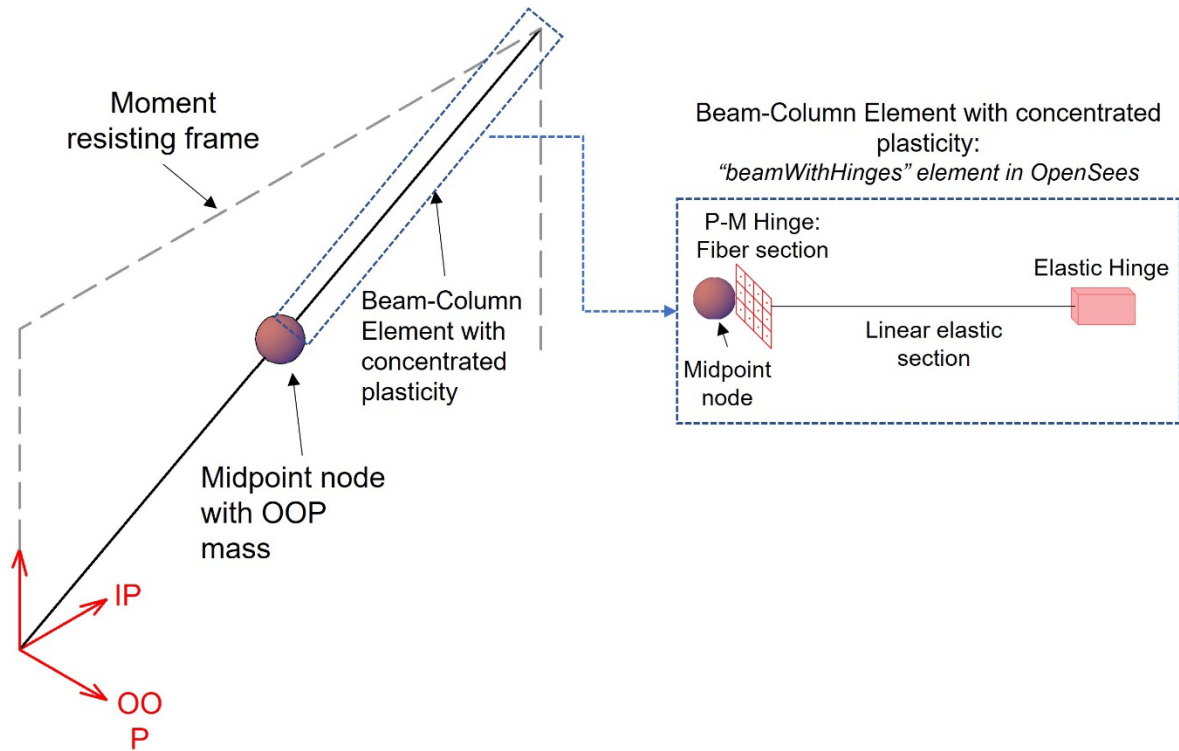


Figure 6.3 Infill Wall model for IP/OOP interaction

As previously mentioned, the proposed model uses a single diagonal strut, which is easy to implement and less computationally expensive to run. One drawback of using one diagonal member instead of two is that the model cannot properly describe the local distribution of forces resulting from the interaction between the infill wall and the surrounding moment frame. Nevertheless, the model can represent the behavior of the masonry infill wall in a global sense, especially if the floor diaphragms are relatively stiff, which is often the case for these types of structures. Furthermore, the benefits of using this model align with the overall goal of the dissertation, which is to determine the global performance of the structure rather than just the infill walls.

When Kadysiewski and Mosalam (2009) sought out to develop the infill model, they wanted their model to comply with the following minimum requirements: (i) under pure IP actions, the model should deliver a prescribed axial stiffness and strength. (ii) under pure OOP actions, the model should have the same natural frequency as the first mode of the infill wall. Moreover, the model should generate the same support reactions and exhibit yielding at the same level of support motion as the original infill wall. (iii) for combined IP and OOP interaction, the model should follow a prescribed strength interaction between IP and OOP strength.

6.1.2.1 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 (ASCE 2000). To find a detailed

description of how the unidirectional (i.e.: IP or OOP) strengths and elastic stiffness are calculated please refer to the original source (ASCE 2000) or Section 3.2 – 3.3 in the report written by Kadysiewski and Mosalam (2009). It is worth mention that the choice to follow FEMA-356 to obtain the IP or OOP capacities is not a requirement of the model. In fact, any other formulation to estimate the IP or OOP capacity of the infill walls can be used. In fact, in the 6-story non-ductile archetype building, the formulations outlined by Chrysostomou and Asteris (2012) were used to obtain the in-plane properties and capacities of the URM infill walls. They suggest estimating the stiffness of the infills in a similar fashion as FEMA 356, but to estimate the strength of the elements, unlike FEMA-356, they considered multiple potential modes of failure, such as sliding-shear failure, compression failure, diagonal tension failure, and general shear failure of the panel.

To include the IP/OOP interaction into the numerical model this dissertation follows the work done by Kadysiewski and Mosalam (2009), which was recently adopted in ASCE 41-17 (ASCE 2017) suggesting a convex curve of the following form:

$$\left(\frac{Q_{IP}}{Q_{IP_0}}\right)^{3/2} + \left(\frac{Q_{OOP}}{Q_{OOP_0}}\right)^{3/2} \leq 1.0 \quad (6.1)$$

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

To achieve this desired interaction curve in the numerical model of the infill panel, Kadysiewski and Mosalam (2009) developed a fiber-section model, like the one depicted in Figure 6.3. This numerical model places the non-linearities of the model at the midspan node of the diagonal strut, using a fiber-section that acts as a P-M hinge. The fiber layout is shown in Figure 6.6a, here the local z-axis is aligned with the OOP direction, normal to the axis of the diagonal strut. To obtain the properties of the fiber sections, namely the yield strength f_{y_i} , the location of each fiber z_i and their area A_i , it's necessary to discretize the P-M interaction curve (from Equation 6.1) at a finite number of points, as shown in Figure 6.6b. A detailed explanation of how to obtain the properties of the fiber section can be found in Chapter 3 of the report written by Kadysiewski and Mosalam (2009). Appendix C lists a summary of the fiber properties used in the model of the masonry infill walls of the case study building. All these fibers are modeled to behave in an elastic perfectly plastic fashion and the strength and stiffness degradation are mainly handled by the element removal.

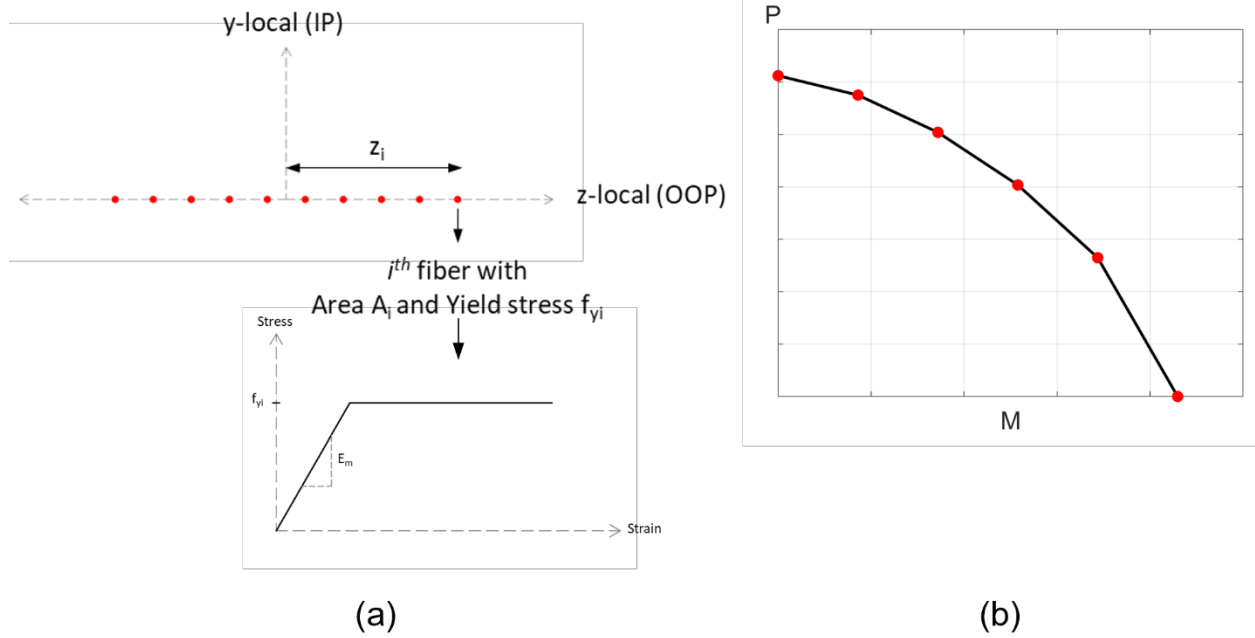


Figure 6.4 (a) Fiber layout in the proposed IP/OOP model; (b) P-M interaction curve from Equation (6.1) Adapted from Kadysiewski and Mosalam (2009)

6.1.2.2 Element removal algorithm

The inclusion of this element removal algorithm is motivated by observations in previous earthquakes around the world, where some of these infill walls collapsed or separated from the surrounding frame. An algorithm of progressive collapse developed by Talaat and Mosalam (2008) and implemented by Mosalam and Günay (2015) in OpenSees was used to explicitly account for the removal of the collapsed URM infill walls. If the reader is interested in learning about the mechanics and implementation of the element removal algorithm, they are encouraged to read Talaat and Mosalam (2008) and Mosalam and Günay (2015) for the specific infill wall removal implementation. A short discussion on how this element removal algorithm was implemented for the automated removal of a diagonal infill wall element is presented in Appendix B.

For the current study, the removal criteria used was a displacement failure curve, that follows a similar 3/2-power relationship like the one presented in Equation (6.1). given by Equation (6.2). The failure curve implemented as a collapse criterion is given by the following expression:

$$\left(\frac{\Delta_{IP}}{\Delta_{IP_0}}\right)^{3/2} + \left(\frac{\Delta_{OOP}}{\Delta_{OOP_0}}\right)^{3/2} \leq 1.0 \quad (6.2)$$

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

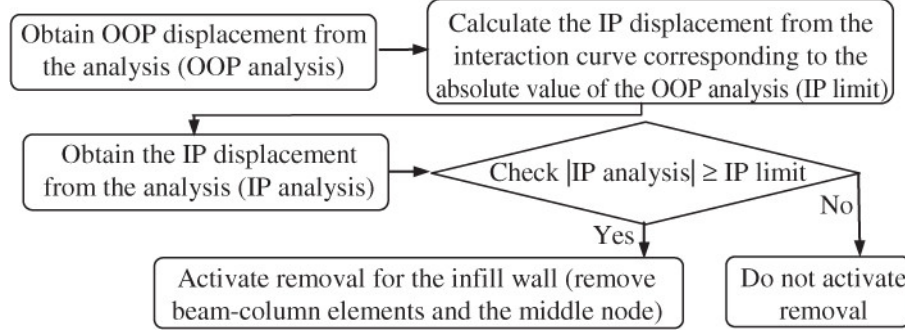


Figure 6.5 Element removal algorithm for URM infill walls. Reproduced with permission of Mosalam and Güney (2015).

The pseudo-algorithm for the element removal for URM infill walls is shown in Figure 6.5. Moreover, Figure 6.6 shows an example of the exact moment where an URM infill wall exceeds its displacement capacity and is deemed collapsed according to the criterion, therefore the infill panel will be physically removed from the simulation. Additionally, Figure 6.7 shows an example of a roof displacement history under seismic loading for the non-ductile six-story case study building, six different instances are highlighted to demonstrate the ability of the progressive collapse algorithm to modify the structural configuration as soon as the URM infill walls reach their capacity and are deemed collapsed. Deformed shapes associated with those six points in time are also presented, in which it is seen that as time progresses and some infills reach their capacity, they are physically removed from the numerical simulation.

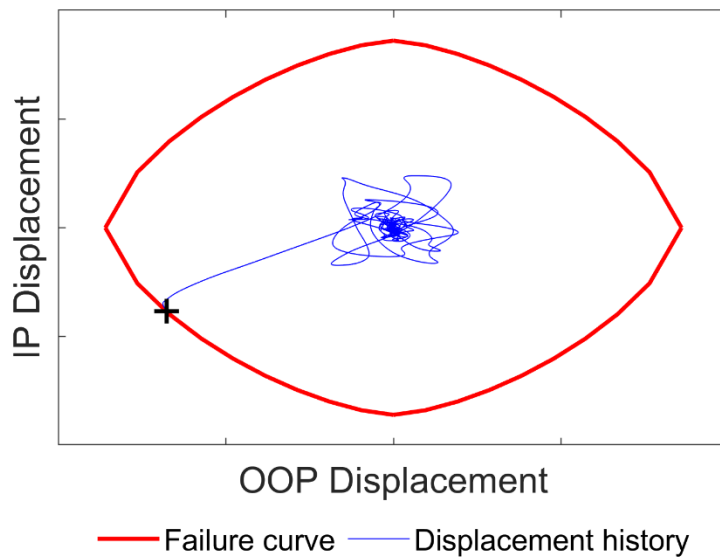


Figure 6.6 Displacement history of a typical URM infill wall.

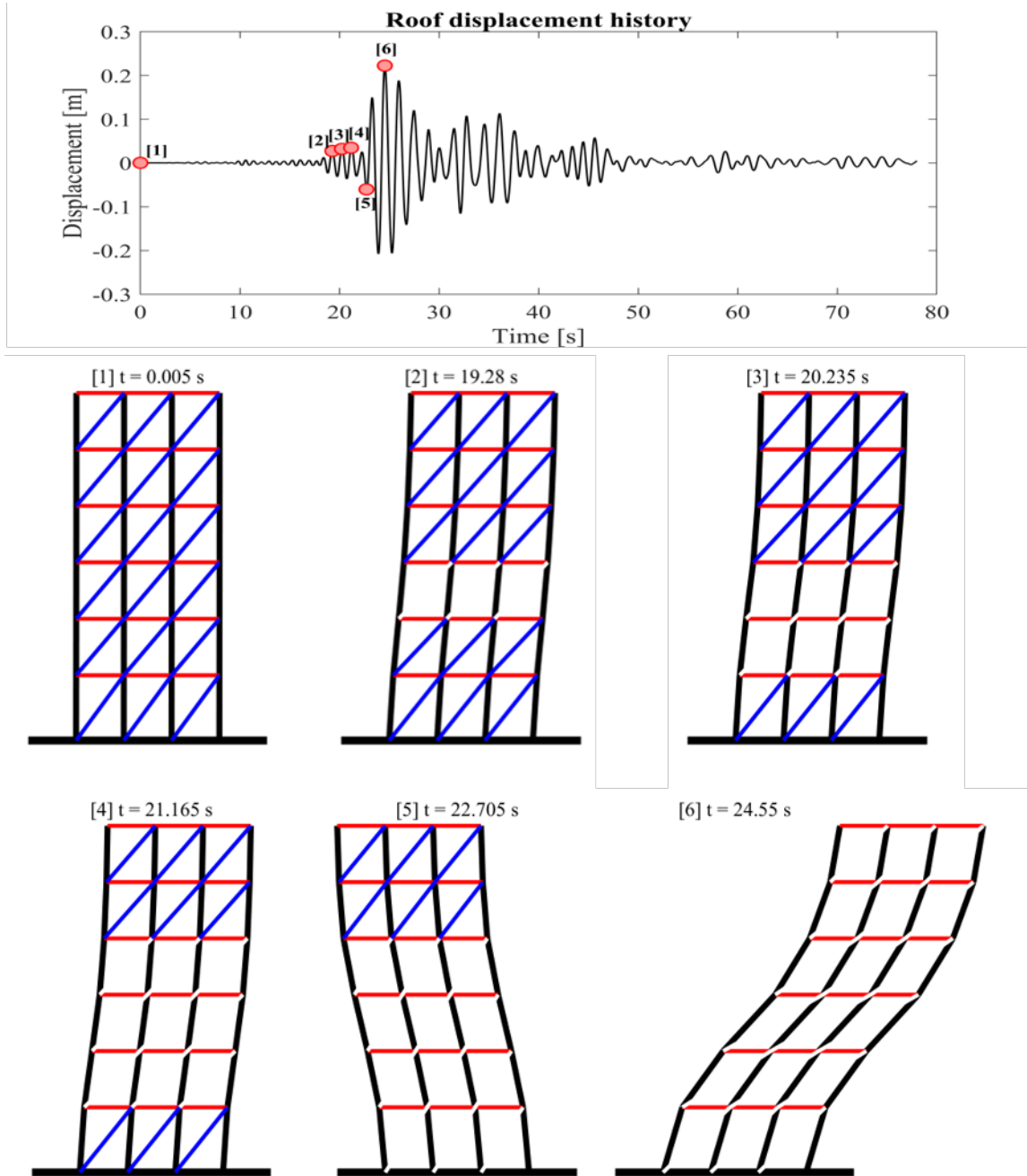


Figure 6.7 Example of implementation of the element removal algorithm.

6.1.3 Performance of the Infill Model

After determining the properties of the fiber-sections used to represent the different types of masonry infill walls of the case study building, these sections are implemented in OpenSees (McKenna et al. 2010) and studied to confirm that the desired model objectives are achieved (i.e.:

prescribed IP and OOP strength, and their interaction). To accomplish this, section level analyses were run, namely moment-curvature and axial force-deformation analyses were performed. As shown in Appendix C, for the 4-story ductile case study building there are four ‘types’ of URM infill walls, namely: (i) Longitudinal, first floor; (ii) Longitudinal, floors 2 to 4; (iii) Transverse, first floor; and (iv) Transverse, floors 2 to 4. Herein, only results for the fiber section model corresponding to the first-floor longitudinal infill wall are presented as reference. The performance of the infill model developed for the 6-story archetype building was also performed, but it’s not shown here. For the case of the first-floor longitudinal infill wall, the correct behavior of the fiber-section is confirmed, as shown in Figure 6.8a, the cross-section achieves the prescribed IP axial strength in the absence of OOP actions, namely the axial strength capacity reaches the expected value of 370.6 kN. Similarly, the cross-section attains the maximum OOP capacity as prescribed, with a moment of 82.1 kN-m. Similar results were obtained for the other three types of URM infill walls but are not shown here.

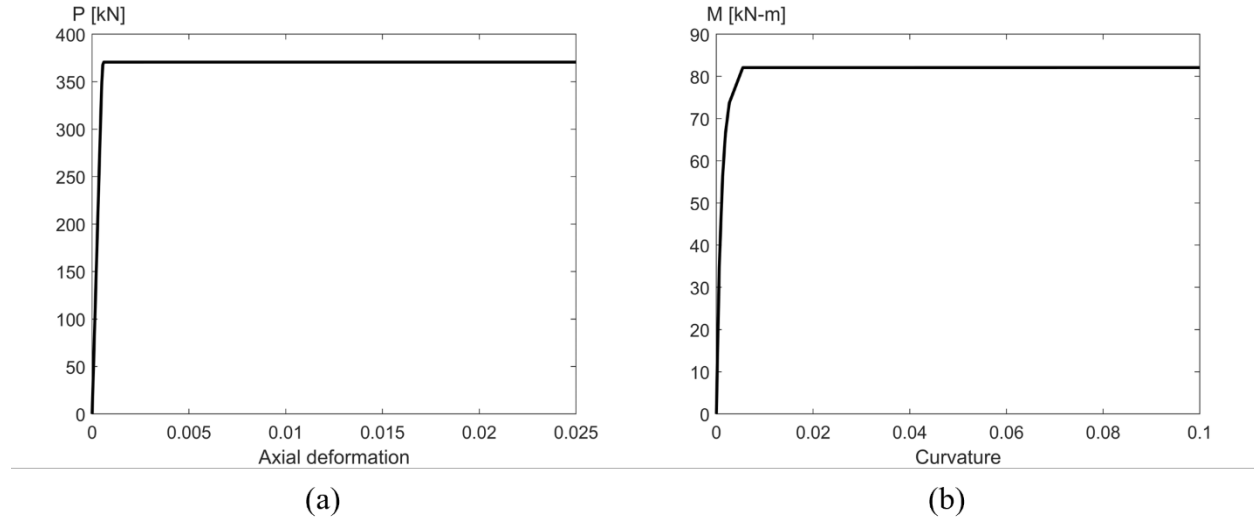


Figure 6.8 (a) Axial force-deformation relation and (b) Moment-curvature relation of first-floor longitudinal infill walls.

To further validate the implementation of the single-strut model based on fiber-sections, a one-story, one-bay frame as the one depicted in Figure 6.9 is analyzed. The dimensions and properties of this simplified model correspond to those of a first-floor longitudinal bay of the 4-story archetype building. The one-story, one-bay model is first evaluated under a pushover analysis (displacement control) in the IP direction. The frame was pushed at the top-right node until the in-plane displacement capacity was reached, namely it was pushed until $\Delta_{IP0} = 0.028275 \text{ m}$. We can see as expected, in Figure 6.10, that the internal forces of the fiber-section follow the anticipated pure in-plane path and reaches its capacity, but do not exceed the prescribed P-M interaction diagram. Similarly, the frame was then pushed in the OOP direction at the midpoint node in the diagonal member until the OOP displacement capacity was reached, namely it was pushed until $\Delta_{OOP0} = 0.027379 \text{ m}$. Just like before, Figure 6.11 shows that the internal forces of the fiber section follow the anticipated out-of-plane path and reaches its capacity, but it never exceeds the prescribed P-M interaction diagram.

Finally, a dynamic uniform earthquake ground motion analysis is performed for the simplified model, where an earthquake ground motion is applied at the fixed supports in both horizontal directions. The applied ground motion is one of the motions selected for the 475 years return period event. Once again, the values of the internal forces, namely the axial force and moment, stay within the limits prescribed by the P-M interaction diagram, as seen in Figure 6.12.

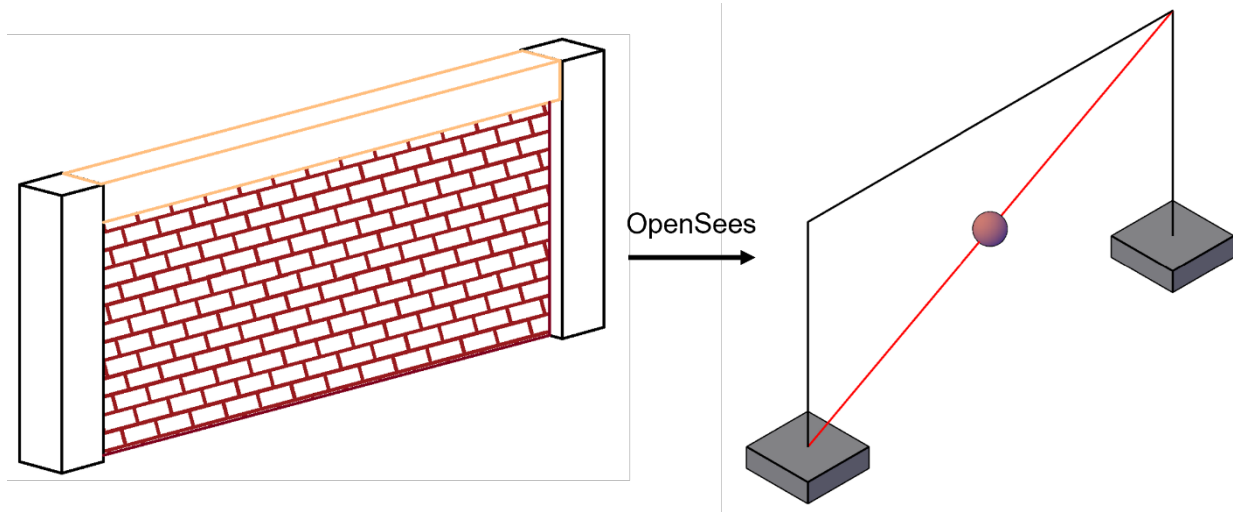


Figure 6.9 Simplified model used to test out the performance of the fiber-sections.

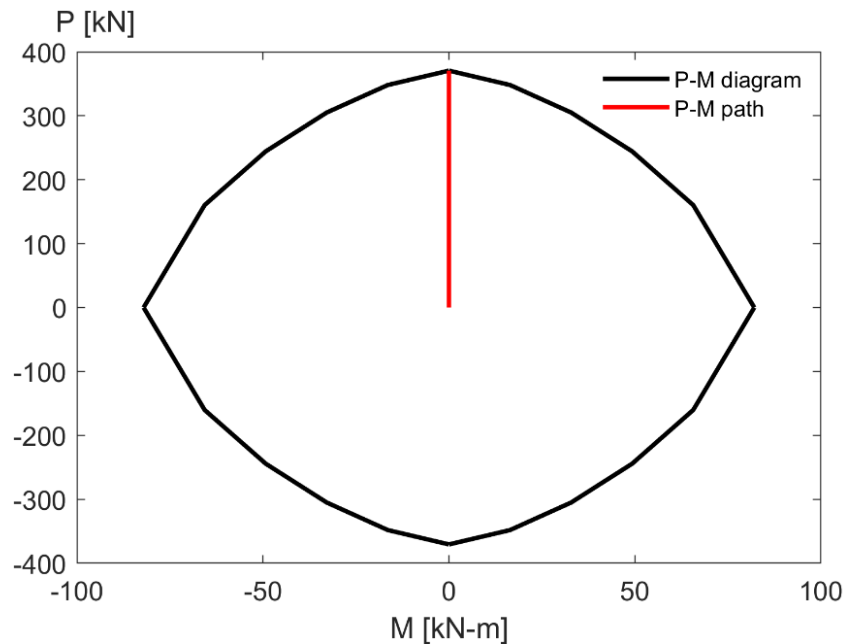


Figure 6.10 P-M diagram and internal forces path for IP pushover analysis.

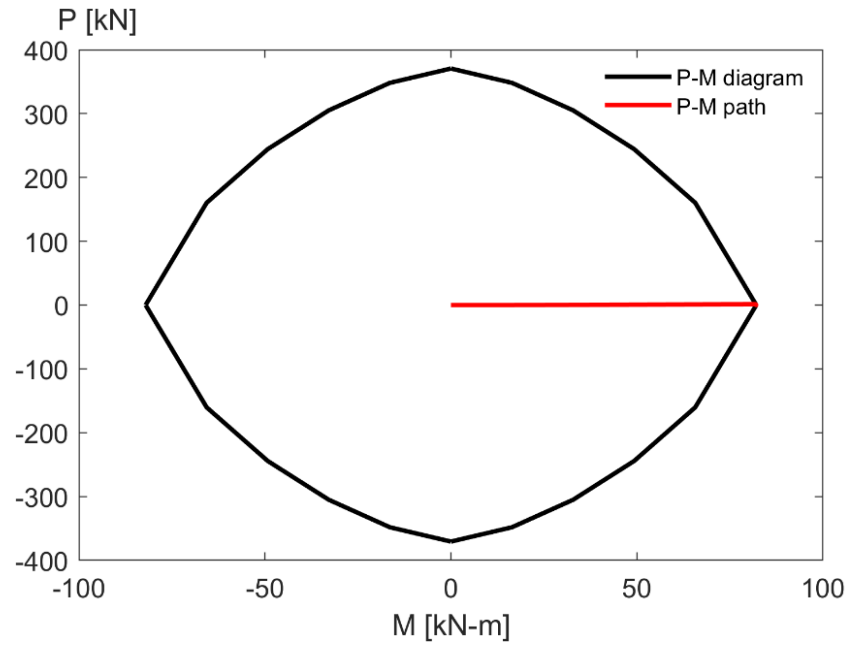


Figure 6.11 P-M diagram and internal forces path for OOP pushover analysis.

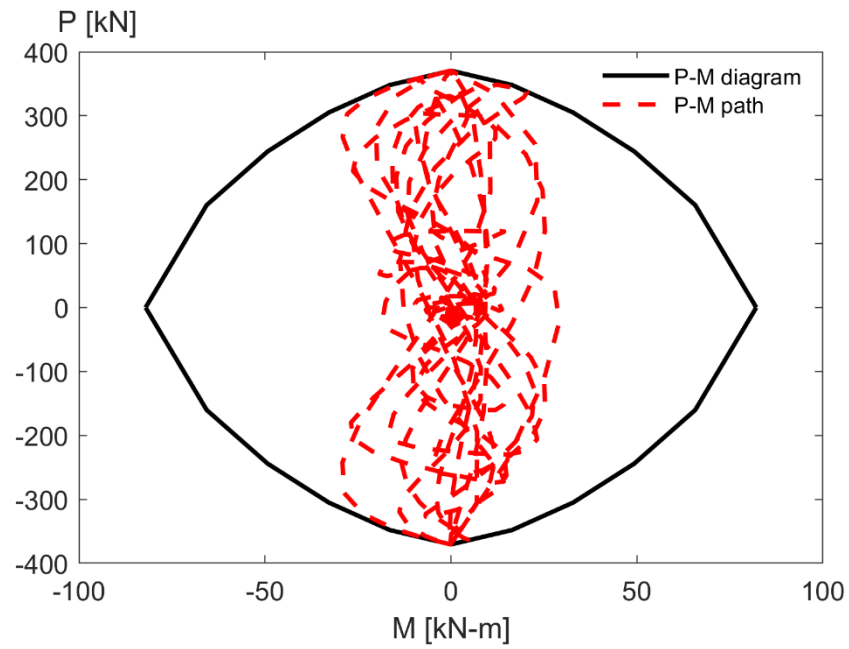


Figure 6.12 P-M diagram for ground motion analysis.

6.1.4 Pushover analysis

6.1.4.1 4-story archetype building

Nonlinear static analyses (pushover analyses) were performed in both horizontal directions of the building, using a lateral load pattern proportional to the respective fundamental mode shape to get an initial insight of the behavior, in terms of strength and global deformation capacity of the archetype building. The bare frames were also analyzed, just for comparison. First, the infilled building was pushed in both directions until a roof drift ratio corresponding to a residual strength of 30% of the peak strength. The pushover response for the infilled frames in the longitudinal and transverse direction is presented in Figure 6.13a and Figure 6.13b, respectively. These pushover curves are presented in terms of the base shear V_s normalized by the weight of the building $W = 12817.95 \text{ kN}$ and the roof drift ratio (RDR) which is defined as the displacement of the roof d normalized by the height of the building $h_{building} = 11.6 \text{ m}$. From these figures, it can be seen that the peak strength is attained at an RDR of 0.53% and 0.63% for the longitudinal and transverse direction respectively. For both directions, the RDR for which the load drops 50% of the peak strength is approximately 2%. Based on this, for this case study, the 2% RDR mark is used as a collapse criterion later when running non-linear time-history analyses.

As previously stated, pushover analyses were also run on the bare frames of the archetype building, and the results for the bare building are presented in Figure 6.14. Just as before, the pushover curves are presented in terms of the base shear V_s normalized by the weight of the bare building $W_{bare} = 11527.53 \text{ kN}$ and the roof drift ratio (RDR) which is defined as the displacement of the roof d normalized by the height of the building. From these figures, it can be seen that the peak strength when the frames are bare is attained at an RDR of 1.788% and 1.71% for the longitudinal and transverse direction respectively. It's worth noticing, that the maximum base shear capacity is greater for the infilled case. This increase in strength is not surprising, and it's due to the presence of infills in the perimeter frames. Having said that, the increase in strength in the longitudinal and transverse directions are only about 15% and 19%, respectively. The limited increase in strength can be attributed to the fact that for the building studied here, only URM infill walls were considered in the perimeter frames, which means there are not that many of them. In buildings where URM infill walls are also considered in the interior of the structure, a significant increase in the peak strength of the building would be expected. A second observation from the pushover curves is the detrimental effect that the inclusion of the URM infill walls into the building has on the ultimate displacement capacity. We see that the peak strength is attained at a significantly lower value of RDR and the ultimate strength is also achieved at significantly lower values of RDR for the case of the infilled building in both horizontal directions, reducing the system ductility significantly compared to the case of a bare frame. A comparison of pushover curves for the infilled and bare frames in both directions is presented in Figure 6.15.

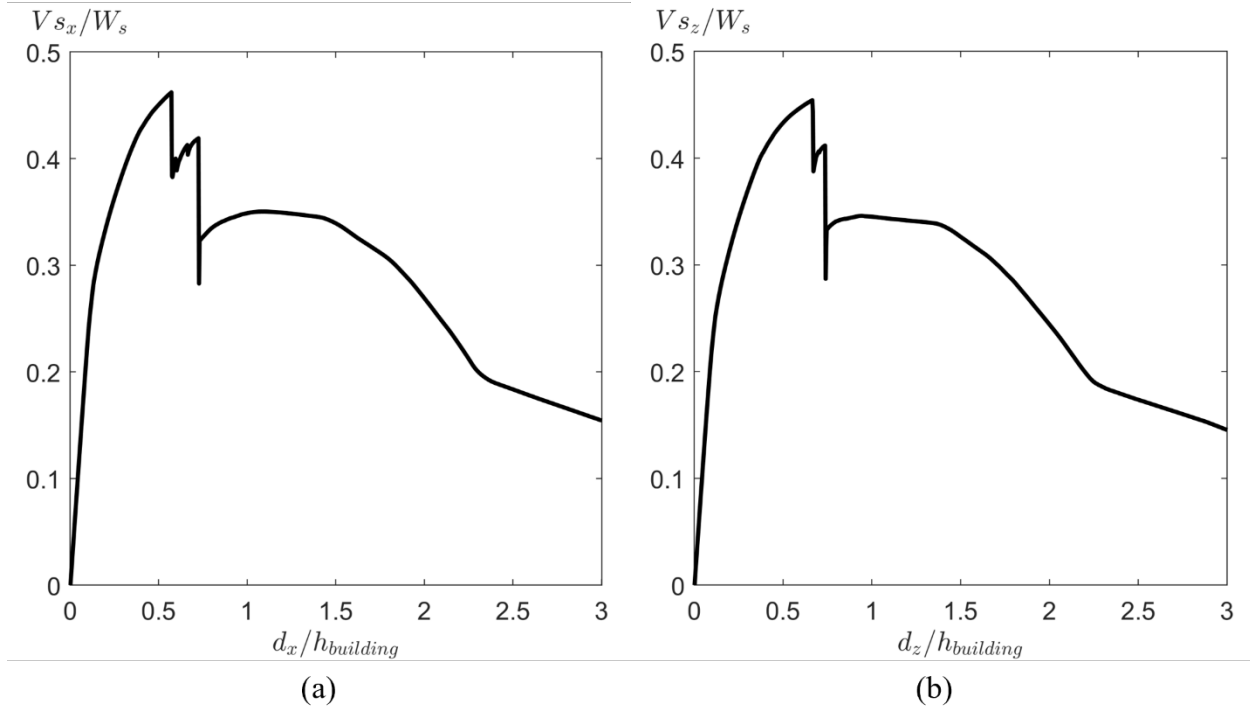


Figure 6.13 Pushover curves for the 4-story archetype building in the (a) longitudinal and (b) transverse direction.

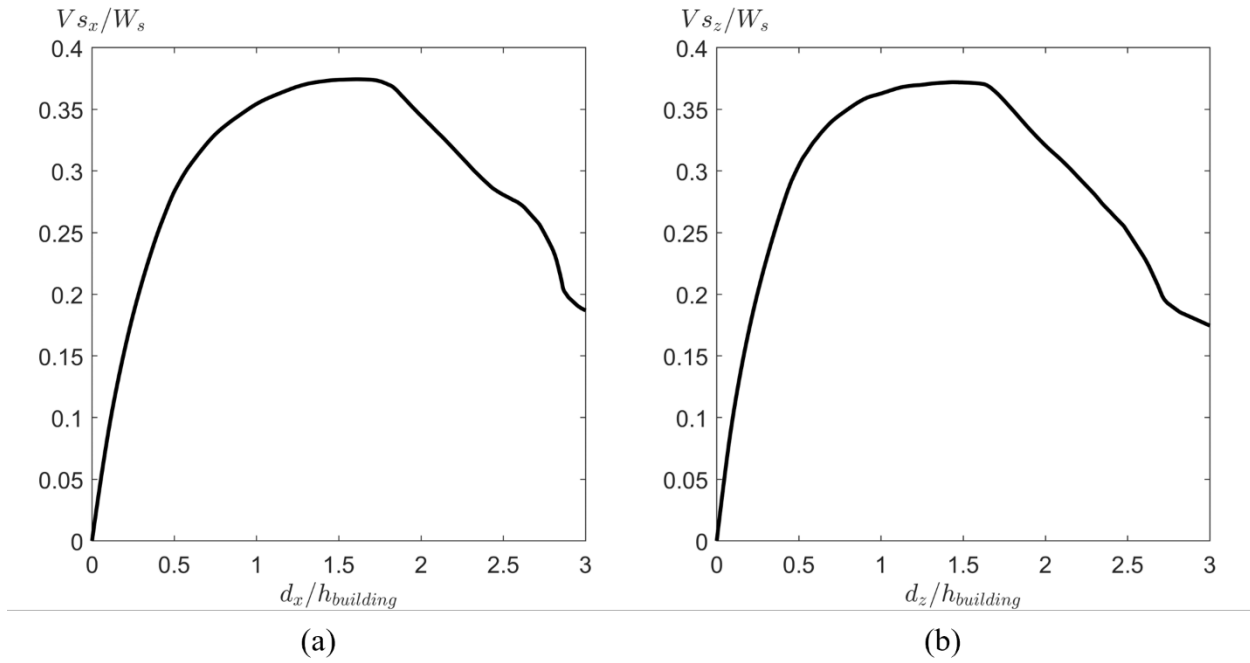


Figure 6.14 Pushover analysis result for the 4-story bare frame in the (a) longitudinal and (b) transverse direction.

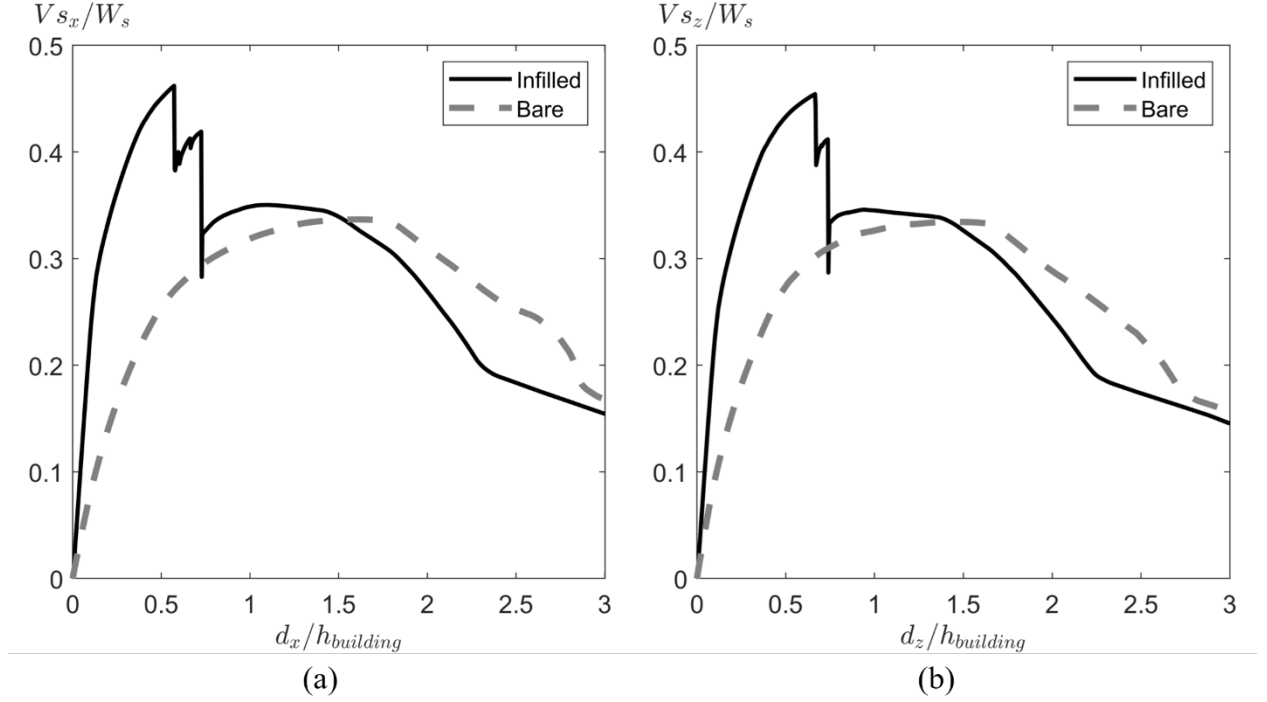


Figure 6.15 Comparison between Pushover analysis result for the 4-story infilled and bare frame in the (a) longitudinal and (b) transverse direction.

6.1.4.2 6-story archetype building

Similar to the previous case study building, nonlinear static analyses (pushovers) were performed in both horizontal directions, using a lateral load pattern proportional to the respective fundamental mode shape. The bare frames were also analyzed, just for comparison. The pushover response for the infilled frames in the longitudinal is presented in Figure 6.16. The pushover in the other direction was performed but is not shown here. This pushover curve is presented in terms of the base shear V_s normalized by the weight of the building $W = 26880 \text{ kN}$ and the roof drift ratio (RDR) which is defined as the displacement of the roof d normalized by the height of the building $h_{building} = 19.7 \text{ m}$. From Figure 6.16, it can be seen that the peak strength is attained at an RDR of 0.81% for the longitudinal direction. For both directions, the RDR for which the load drops 20% of the peak strength is approximately 1.68%. Based on this, for this case study, the 1.68% RDR mark is used as a collapse criterion later when running non-linear time-history analyses.

As previously stated, pushover analyses were also run on the bare frames of the archetype building, the results for the bare building are presented in Figure 6.17. Just as before, the pushover curve is presented in terms of the base shear V_s normalized by the weight of the bare building $W_{bare} = 24942 \text{ kN}$ and the roof drift ratio (RDR) which is defined as the displacement of the roof d normalized by the height of the building. The peak strength when the frame is bare is attained at an RDR level of 0.98% for the longitudinal direction. It's worth noticing that the maximum base shear capacity does not change much for this second archetype building. The lack

of significant change in the capacity of the infilled frame vs the bare frame can be explained due to the significantly lower IP and OOP capacity of the infills resulting from using the formulations in Chrysostomou and Asteris (2012), compared to those obtained in the first archetype building following only FEMA-356.

As in the previous case study building, it can be observed from the pushover curves a slight detrimental effect that the inclusion of the URM infill walls into the building has on the ultimate displacement capacity. For this archetype building, the two pushover curves are quite similar, due to the fact that the infill walls considered in this building have low strength. A comparison of the pushover curves for the bare and infilled frames for this building is presented in Figure 6.18. We see that the peak strength is attained at a slightly lower value of RDR and the ultimate strength is also achieved at a lower values of RDR for the case of the infilled building in the studied direction.

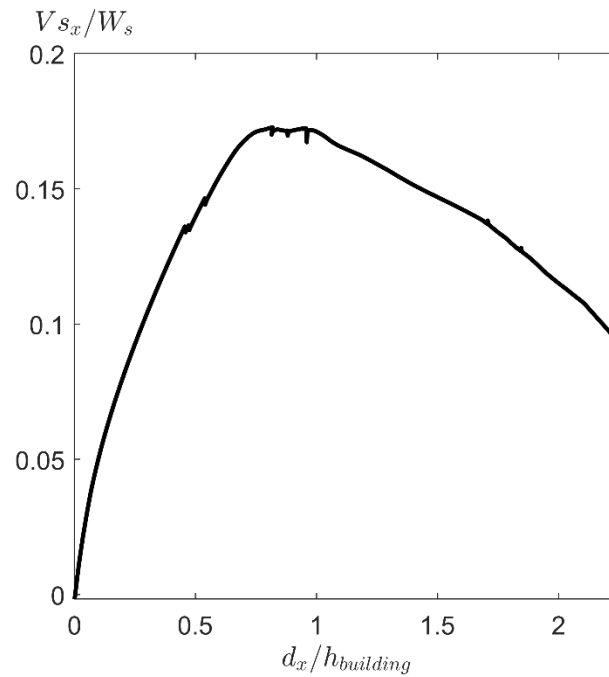


Figure 6.16 Pushover curves for the 6-story archetype building in the longitudinal direction.

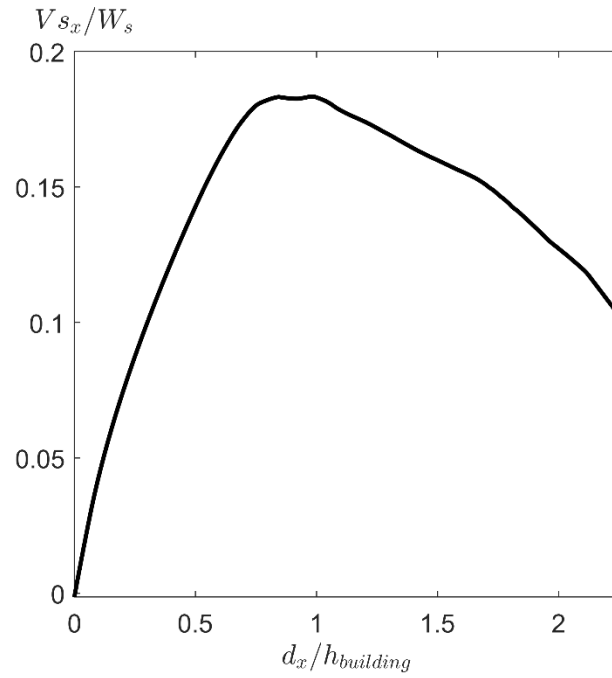


Figure 6.17 Pushover analysis result for the 6-story bare frame in the longitudinal direction.

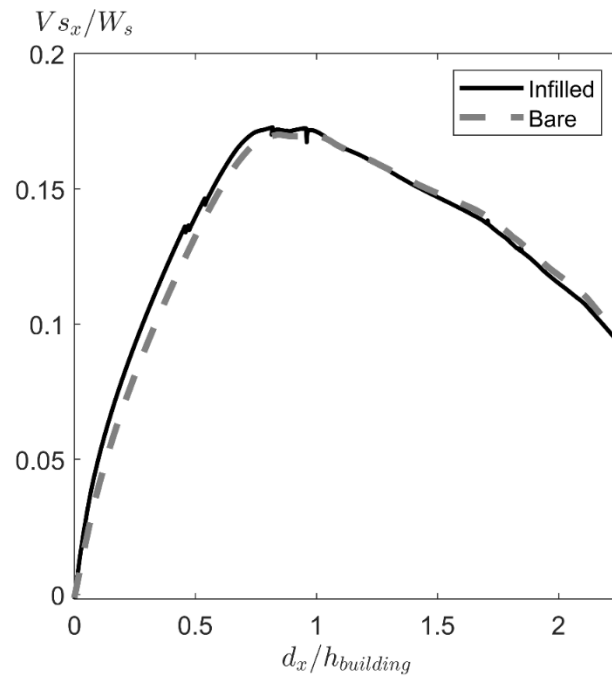


Figure 6.18 Comparison between Pushover analysis result for the 6-story infilled and bare frame in the longitudinal.

6.1.5 Nonlinear time history analyses for PBEE

6.1.5.1 4-story archetype building

Nonlinear time-history analyses are performed using OpenSees (McKenna et al. 2010) for all the hazard levels discussed in Section 5.4.5. For these nonlinear dynamic analyses, mass and stiffness-proportional Rayleigh damping was assumed, the damping coefficients were computed to achieve a damping ratio $\xi = 2.5\%$ at periods corresponding to the first and third fundamental vibration mode of the model, following recommendations found in the literature (Reinhorn et al. 2010). The first two fundamental periods for the bare frame building are $T_{1Bare} = 0.569 \text{ sec}$ and $T_{2Bare} = 0.517 \text{ sec}$. When the URM infill walls are included into the model, a reduction in the fundamental periods is observed as expected. The corresponding first two fundamental periods for the infilled building are $T_{1infilled} = 0.348 \text{ sec}$ and $T_{2infilled} = 0.347 \text{ sec}$. The 4-story case study building was analyzed with pairs of two horizontal component accelerograms input as uniform excitation at the base. The selected pair of ground motions used as inputs to the model are obtained from the Conditional Scenario Spectra (CSS) discussed in Section 5.4.5.

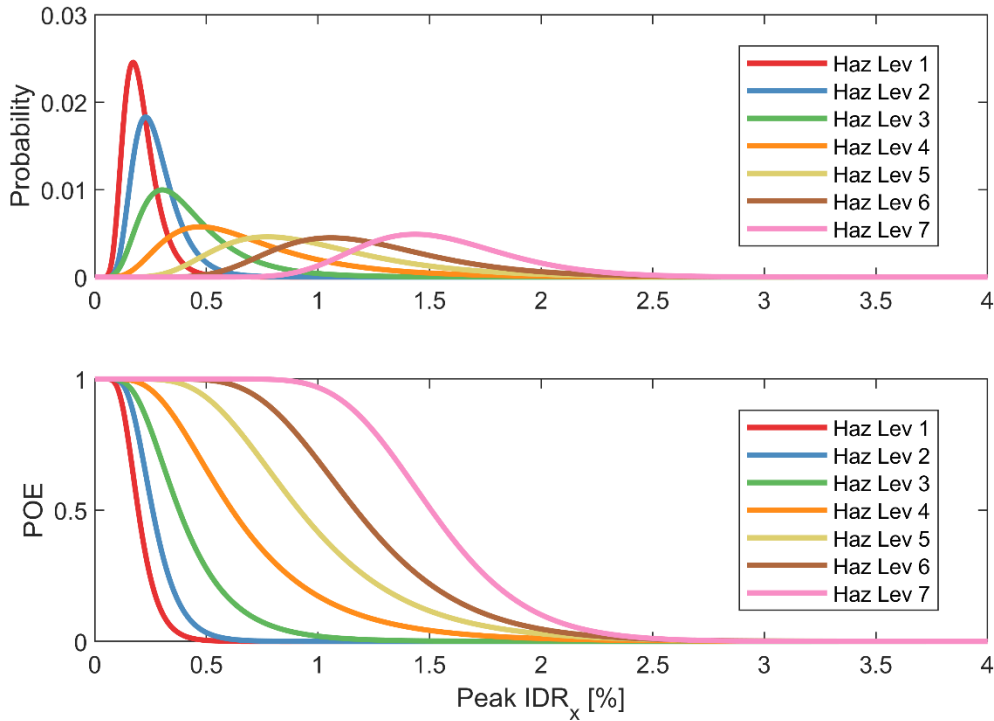
As part of the PEER-PBEE methodology, it's necessary to break down the building into damageable groups that are affected by the same EDP in a similar manner. Thus, at this stage three damageable groups are identified: structural components, non-structural components, and URM infill walls. Maximum inter-story drift ratio (IDR) and peak floor acceleration (PFA) are selected as the EDPs of interest. IDR is used in the next section to determine the damage state of the structural components and URM infill walls, whereas PFA is related to the performance of non-structural components.

As discussed in Section 3.3, three probabilities quantities are calculated: $p(EDP > edp|IM = im)$, $p(C|IM)$ and $p(NC|IM)$ for each hazard level. The probability of collapse is calculated, simply, as the number of ground motions that result in structural collapse divided by the total number of ground motions run at that hazard level. The 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 RDR, 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 6.1. From Table 6.1, we can see that in first three hazard levels none of the ground motions resulted in collapse, and that from hazard level 4 on, the number of collapses increases until we get to the last hazard level where all time history analyses resulted in the structure to collapse.

Table 6.1 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	35	0	0	35	1
2	44	0	0	44	1
3	48	0	0	48	1
4	45	2	0.044	43	0.956
5	45	8	0.178	37	0.822
6	46	17	0.370	29	0.630
7	41	34	0.829	7	0.171

The probabilities of the EDPs, namely, $POE(EDP > edp|IM = im)$ are calculated only from the non-collapsed cases. It is worth noting, that it was not possible to estimate statistics for the last hazard level, since all of the ground motions ran resulted in collapse of the structure. 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 (Baker 2015). Probability and POE of peak IDR_x , and PFA are shown in Figure 6.19 to Figure 6.22.

**Figure 6.19** Probability and POE of peak IDR_x in the longitudinal direction.

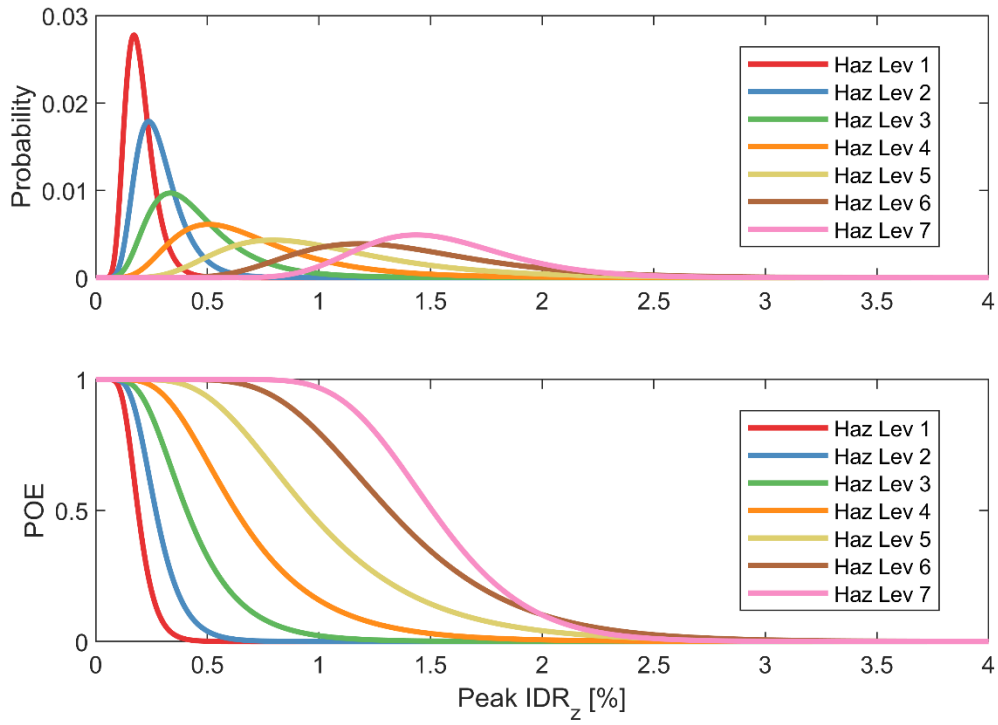


Figure 6.20 Probability and POE of peak IDR in the transverse direction.

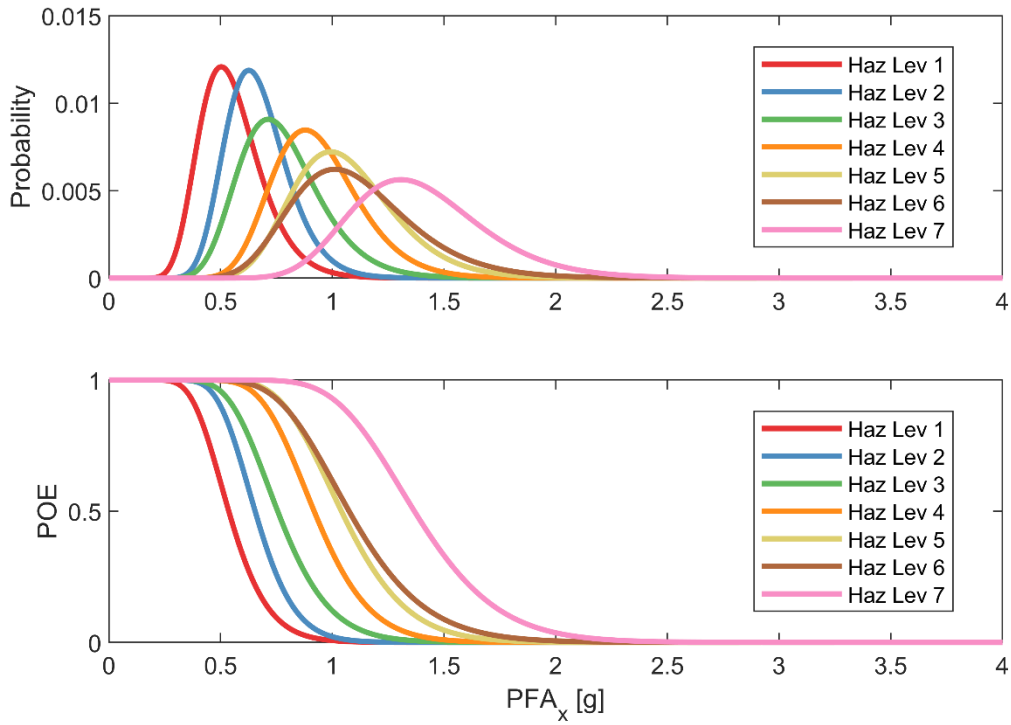


Figure 6.21 Probability and POE of PFA in the longitudinal direction.

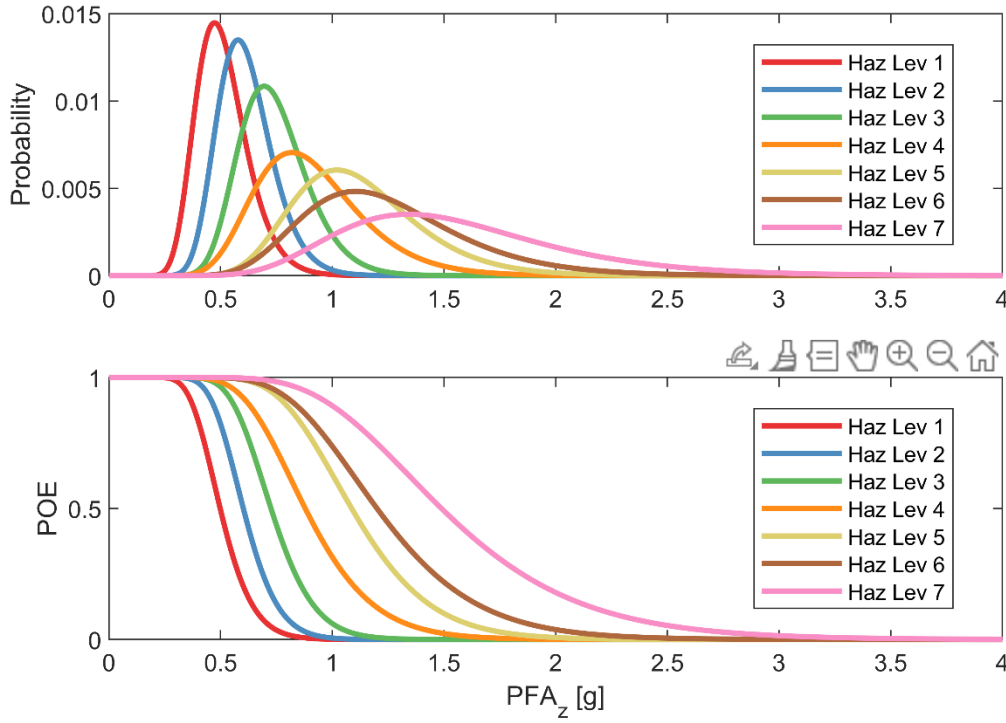


Figure 6.22 Probability and POE of PFA in the transverse direction.

6.1.5.2 6-story archetype building

Similarly, for the 6-story non-ductile archetype building, NLTH analyses are performed using OpenSees (McKenna et al. 2010) for all the hazard levels discussed in Section 5.4.5. For these nonlinear dynamic analyses, mass and stiffness-proportional Rayleigh damping was assumed, and the damping coefficients were computed to achieve a damping ratio $\xi = 2.5\%$ at periods corresponding with the first and third fundamental vibration mode of the model, following recommendations found in the literature (Reinhorn et al. 2010). The first two fundamental periods for the bare frame building are $T_{1Bare} = 1.08 \text{ sec}$ and $T_{2Bare} = 0.99 \text{ sec}$. When the URM infill walls are included into the model, a reduction in the fundamental periods is observed as expected. The corresponding first two fundamental periods for the infilled building are $T_{1infilled} = 0.91 \text{ sec}$ and $T_{2infilled} = 0.81 \text{ sec}$. As in the previous case study, the 6-story case study building was analyzed with pairs of two horizontal component accelerograms input as uniform excitation at the base. The selected pair of ground motions used as inputs to the model are obtained from the Conditional Scenario Spectra (CSS) ran in Section 5.4.5.

For this case study, the collapse state is assumed to be reached when the structure's peak RDR, 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 6.2. From this table, we can see that in first three hazard levels none of the ground motions ran made the structure

collapse, and that from hazard level 4 on, the number of collapses increases until we get to the last hazard level where all time history analyses resulted in the structure to collapse.

Table 6.2 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	35	0	0	35	1
2	44	0	0	44	1
3	48	1	0.0208	47	0.9792
4	45	4	0.0889	41	0.9111
5	45	20	0.4444	25	0.5556
6	46	33	0.7174	13	0.2826
7	41	41	1	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 it was not possible to estimate statistics for the last two hazard level, since all of the ground motions ran resulted in collapse of the structure. 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 (Baker 2015). Probability and POE of peak IDR_x , and PFA are shown in Figure 6.23 to Figure 6.26.

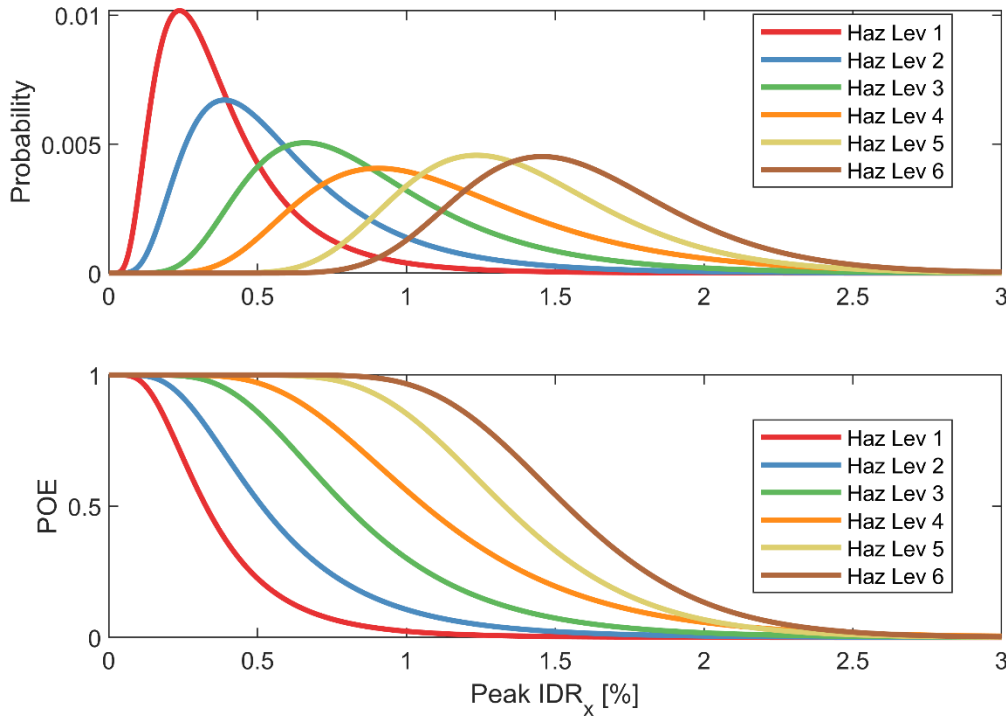


Figure 6.23 Probability and POE of peak IDR in the longitudinal direction.

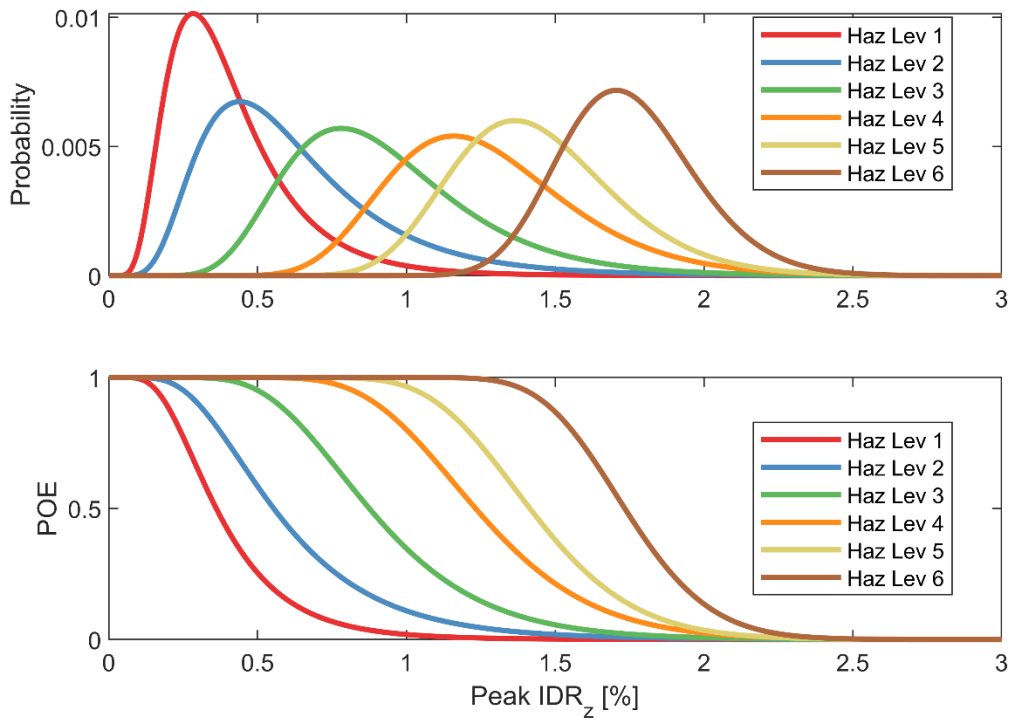


Figure 6.24 Probability and POE of peak IDR in the transverse direction.

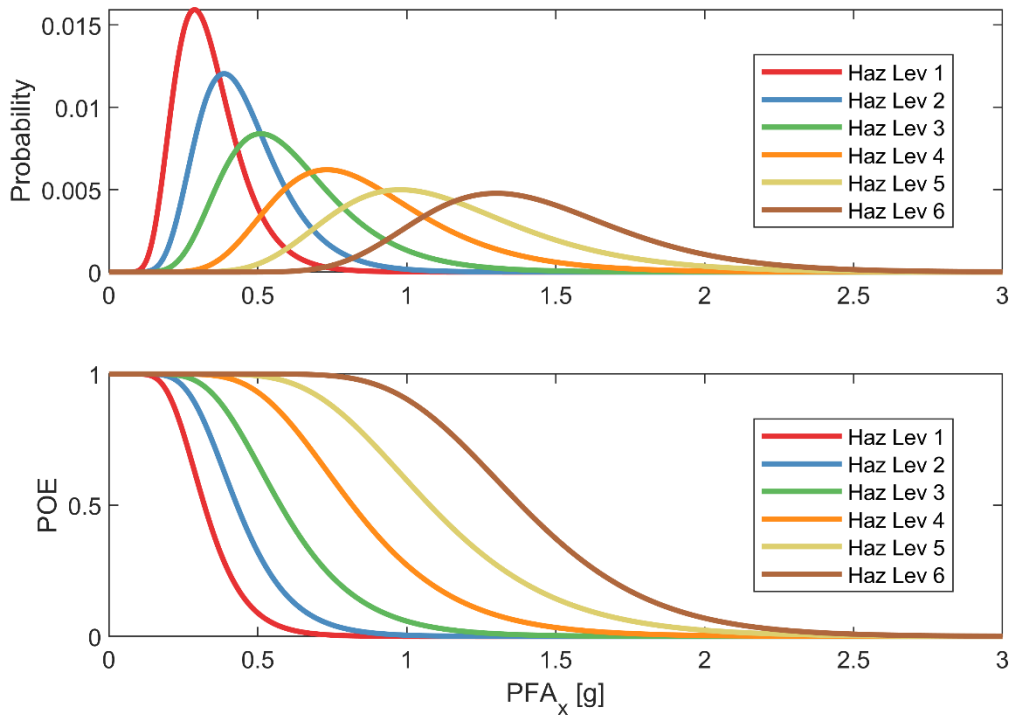


Figure 6.25 Probability and POE of PFA in the longitudinal direction.

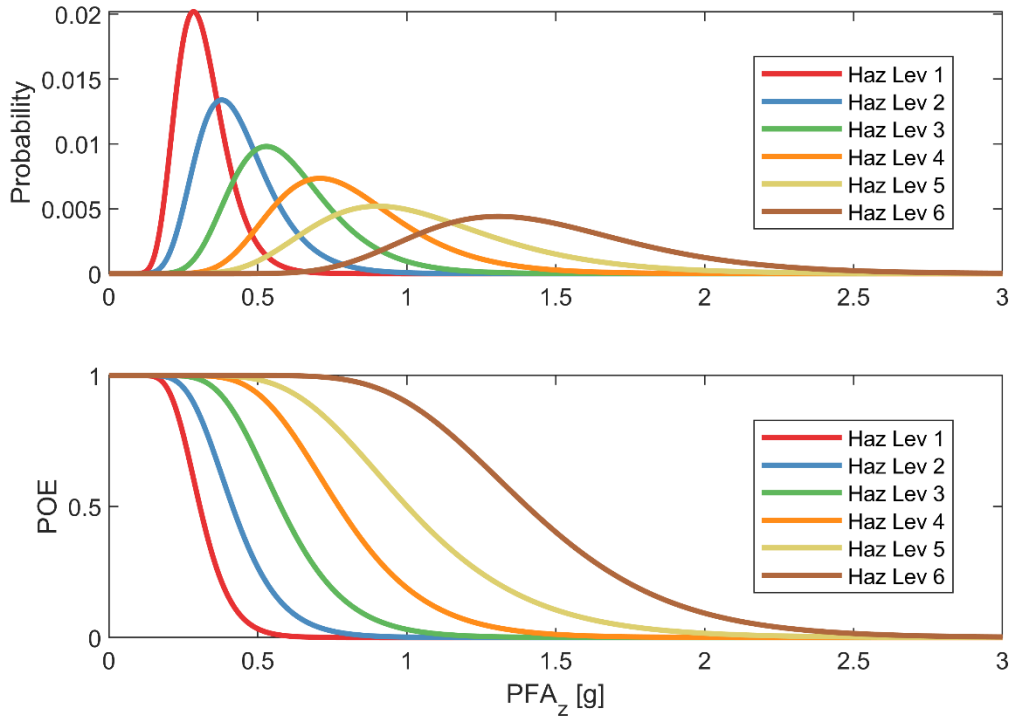


Figure 6.26 Probability and POE of PFA in the transverse direction.

6.1.6 Fragility Analysis

As previously mentioned, system fragility functions are plots of $P\{EDP > edp | IM = im\}$ versus IM , and they represent a cumulative distribution of the capacity of a structure to resist an adverse limit state (Porter 2019). One can estimate this fragility functions using the results from the NLTH analyses resulted from using the ground motions obtained from the CSS in Chapter 5. From these NLTH runs, sets of EDPs and their corresponding IM are obtained, as shown in the scatter plots of Figure 6.27 and Figure 6.30, for the 4-story and 6-story case study buildings respectively. In this case, the intensity measure of choice was $S_a(T_1)$; from these scatter plots it can be noted that increasing values of the intensity measure yield increasing EDP levels, as anticipated. On the other hand, it's possible to see that the dispersion of the data increases as the IM grows. This is a sign that $S_a(T_1)$ might not be a good IM to estimate the chosen EDPs (Arteta and Abrahamson 2019, Araujo 2021). These pairs of EDPs vs IM are then binned by intensity level, from there the estimation of fragility functions is straight forward, and can be obtained following the methodology presented by Baker (2015) by fitting lognormal distributions to the sorted and binned data.

The EDPs used in this section to estimate fragility are IDR and PFA, these two are chosen because they can be later linked to damage and loss estimation of the components of the structure. System-level fragility functions for the 4-story building are presented in Figure 6.28 and Figure

6.29 for several peak IDR levels in the longitudinal and transverse direction, respectively. From these curves, one could estimate for example that for the 4-story building in the longitudinal direction, given an intensity measure $S_a(T_1) = 1.0g$ the probability that the archetype building exceeds a peak IDR level of 0.2%, 0.5%, 1.0%, and 2.0% are 100%, 63%, 30%, and 7%, respectively.

System-level fragility functions for the non-ductile 6-story building are presented in Figure 6.31 and Figure 6.32 for several peak IDR levels in the longitudinal and transverse direction, respectively. As before, from these curves, one could estimate for the 6-story building in the longitudinal direction, given an intensity measure $S_a(T_1) = 1.0g$ the probability that the archetype building exceeds a peak IDR level of 0.2%, 0.5%, 1.0%, and 2.0% are 100%, 100%, 99%, and 73%, respectively.

Comparing the results from both archetype buildings, it's clear that the probability of exceedance a given IDR level for the up-to-code 4-story building are much smaller than those of the non-ductile 6-story building, as expected. In general, we can see that the fragility functions of the non-ductile 6-story building are shifted to the left for same values of IDR levels when compared to those of the 4-story building, indicating that the non-ductile building is *more fragile* than the 4-story building.

6.1.6.1 4-story archetype building

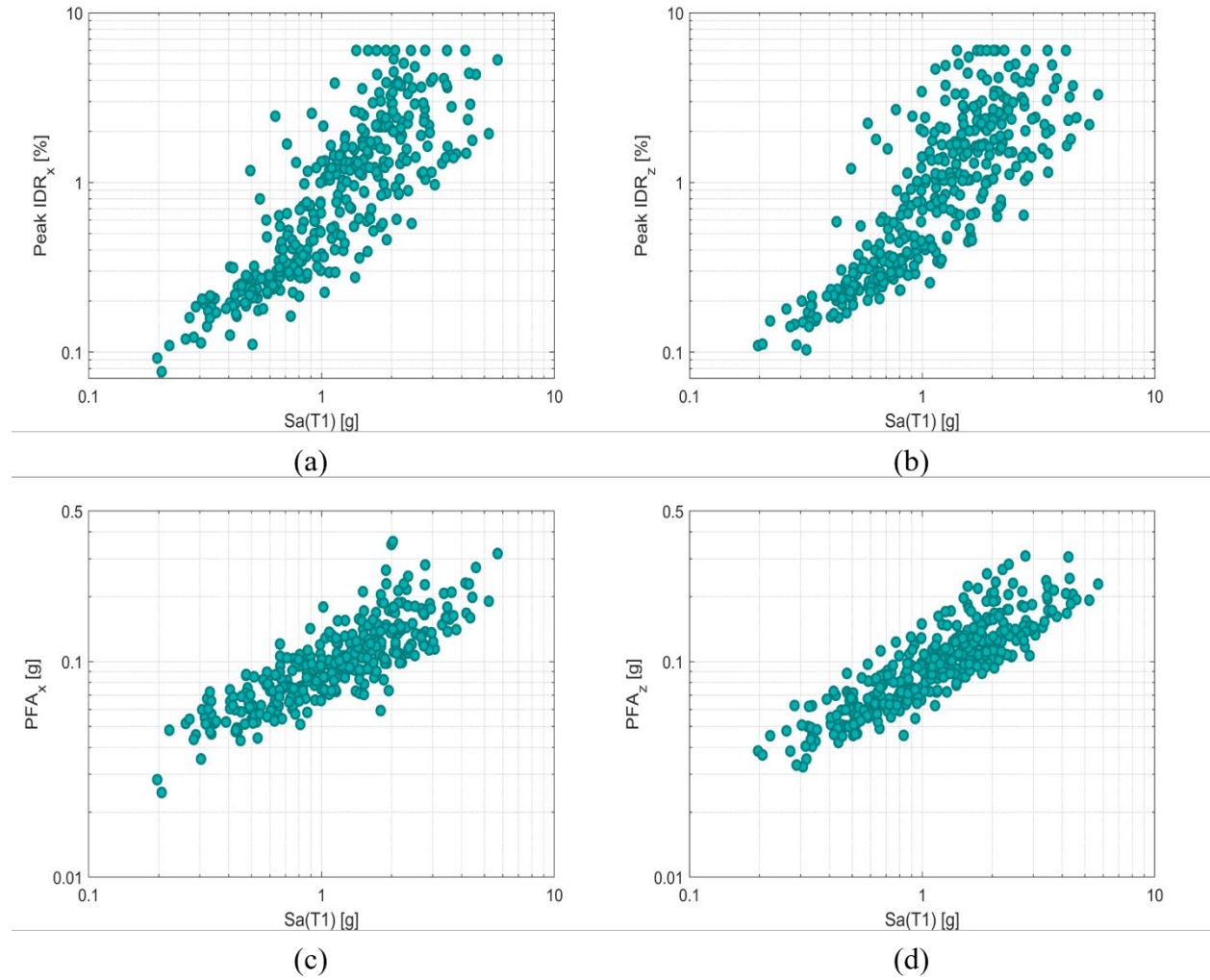


Figure 6.27 Scatter plots of global EDPs from CSS runs for the 4-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction. For these runs, the IDR shown were capped for the sake of presentation at 6%.

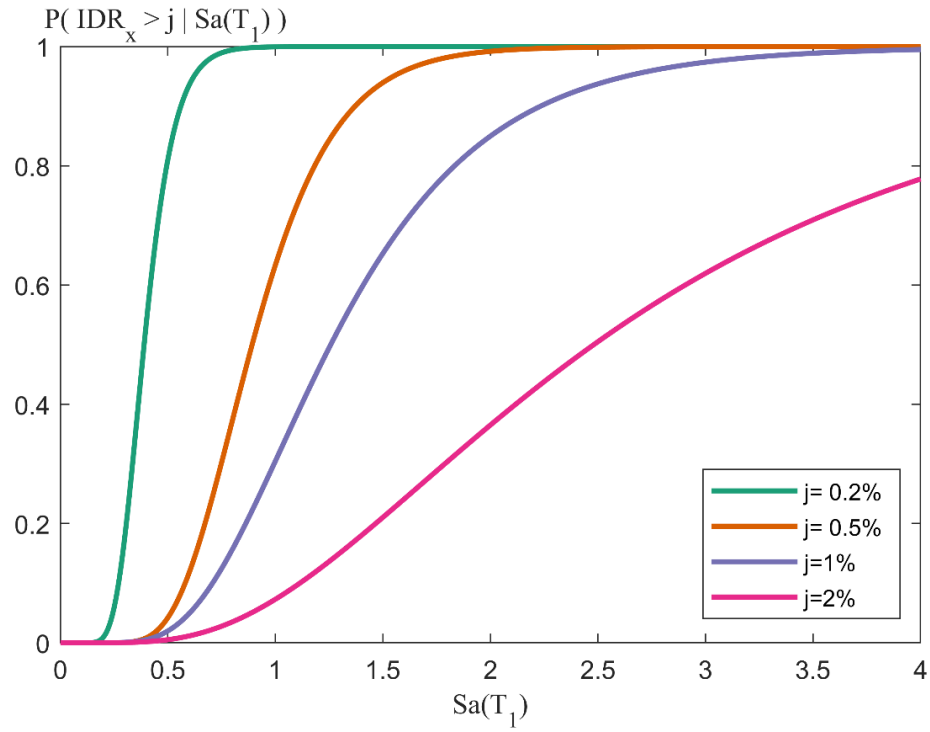


Figure 6.28 Fragility curves for the peak IDR in the longitudinal direction for several IDR levels.

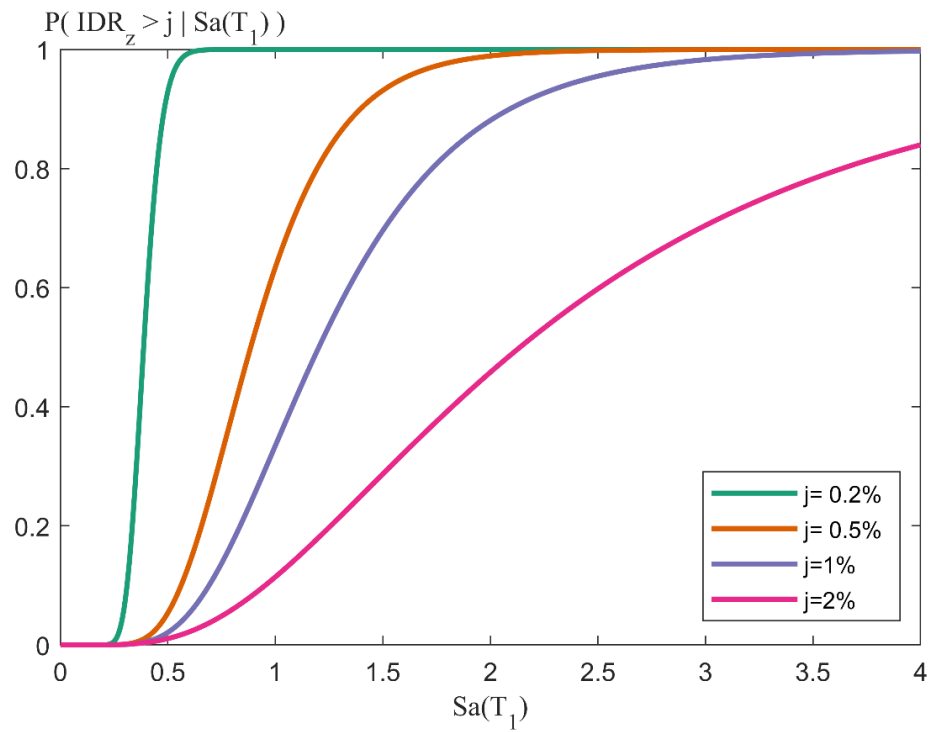


Figure 6.29 Fragility curves for the peak IDR in the transverse direction for several IDR levels.

6.1.6.2 6-story archetype building

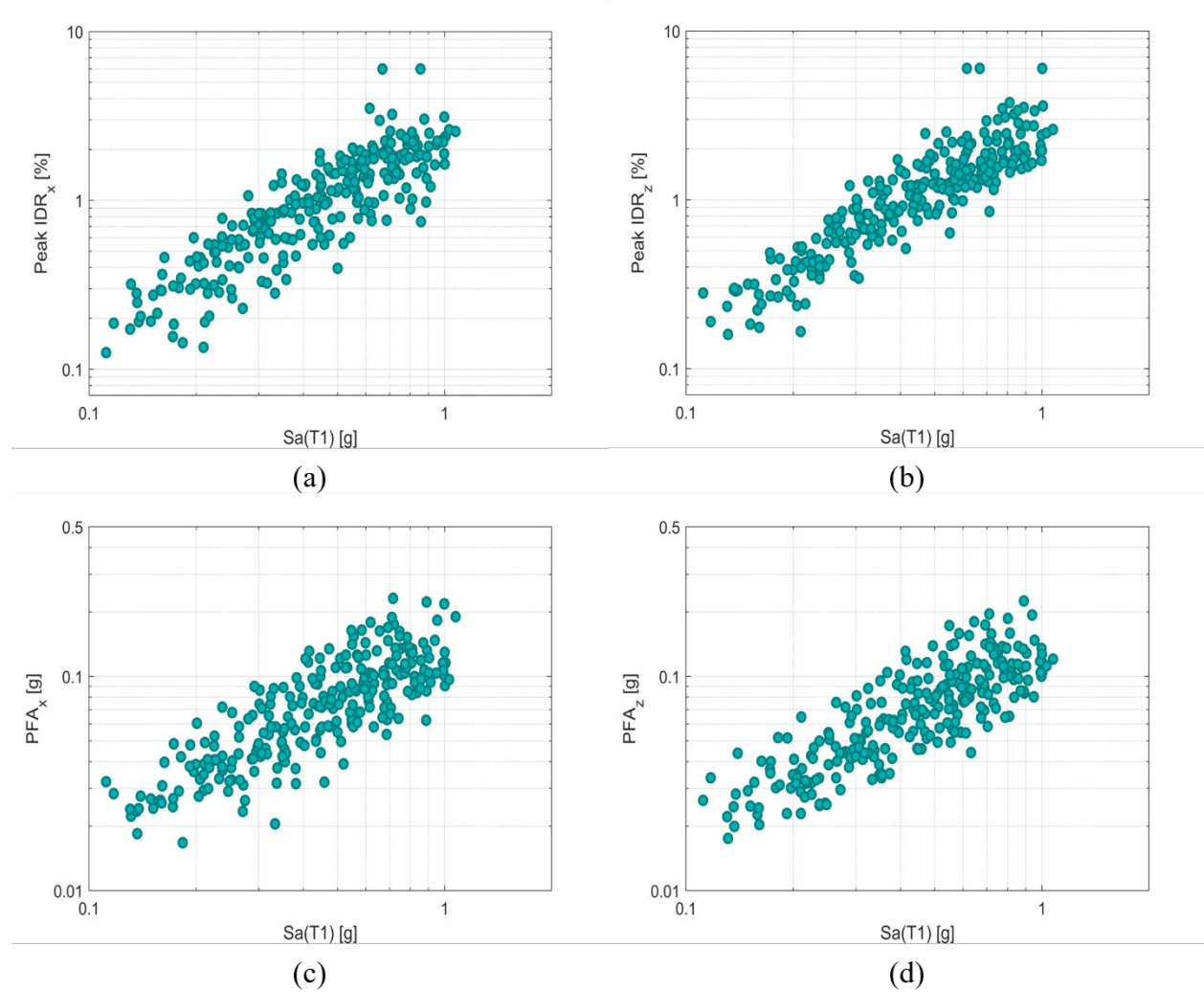


Figure 6.30 Scatter plots of global EDPs from CSS runs for the 6-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction. For these runs, the IDR shown were capped for the sake of presentation at 6%.

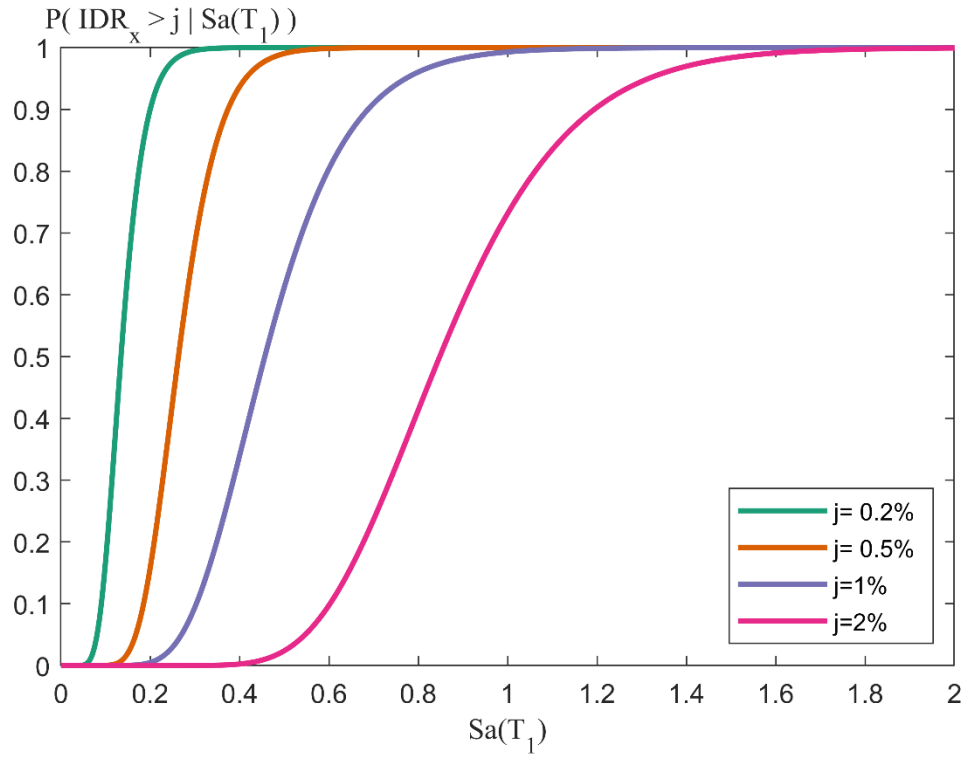


Figure 6.31 Fragility curves for the peak IDR in the longitudinal direction for several IDR levels.

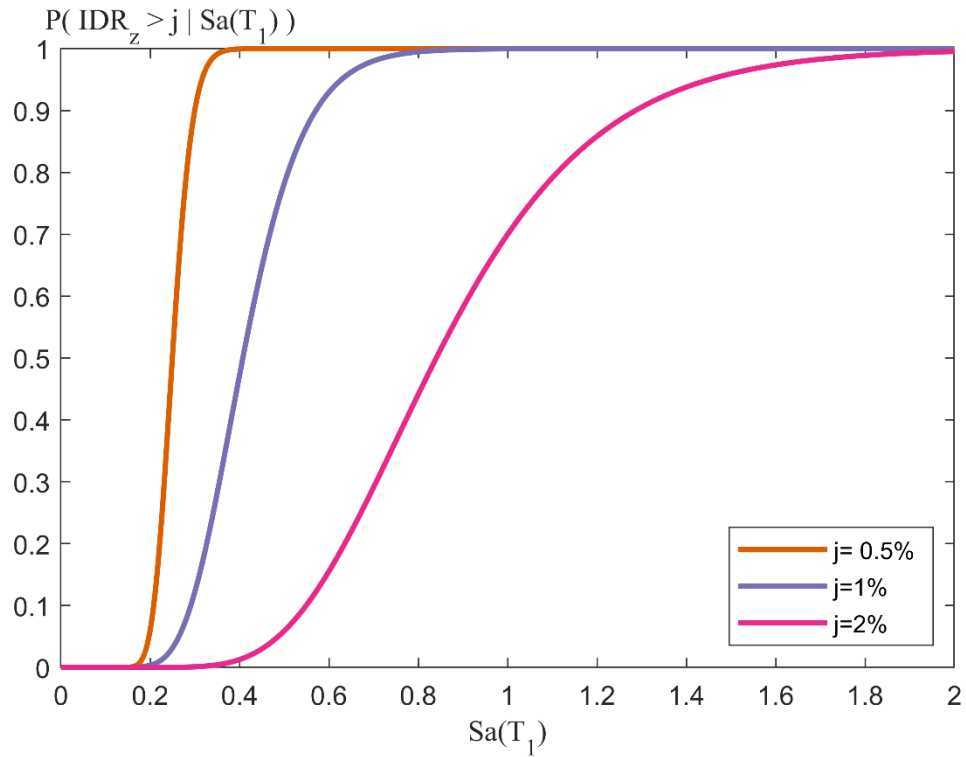


Figure 6.32 Fragility curves for the peak IDR in the transverse direction for several IDR levels.

6.1.7 Estimation of EDP-Risk

Recall that from the CSS methodology, as discussed in section 5.4.6, one can assign to a resulting EDP obtained from NLTH the same rate of occurrence of that of the time history that produced such EDP. Figure 6.33 and Figure 6.35 show scatter plots of rates of occurrence versus peak EDPs for the 4-story and 6-story archetype building, respectively.

From there, as previously mentioned, we can make use of the outputs of the CSS methodology, to estimate $\lambda(EDP > edp)$ in a hazard-consistent approach using Equation (5.6):

$$\lambda(EDP > edp) = \sum_{i=1}^n Rate_{CSS,i} H(EDP - edp) \quad (5.6)$$

Figure 6.34 and Figure 6.36 show annual rate of exceedance of peak global EDPs from CSS runs for the 4-story archetype building, and the 6-story building, respectively. As stated by Arteta and Abrahamson (2019), the inverse of $\lambda(EDP > edp)$, represents the return period (TR) of a given EDP level. The estimation of the EDP risk can be used to assess if a given performance objective was met or vicesversa if a risk associated with a given EDP level is acceptable. For example, Figure 6.34a shows that an EDP level of 2% in the longitudinal direction of the case study building has an annual rate of exceedance of approximately 0.000589, which corresponds to a return period of approximately 1695 years. Similar analysis can be done for the other EDPs in resulting from the assessment of both case studies.

6.1.7.1 4-story archetype building

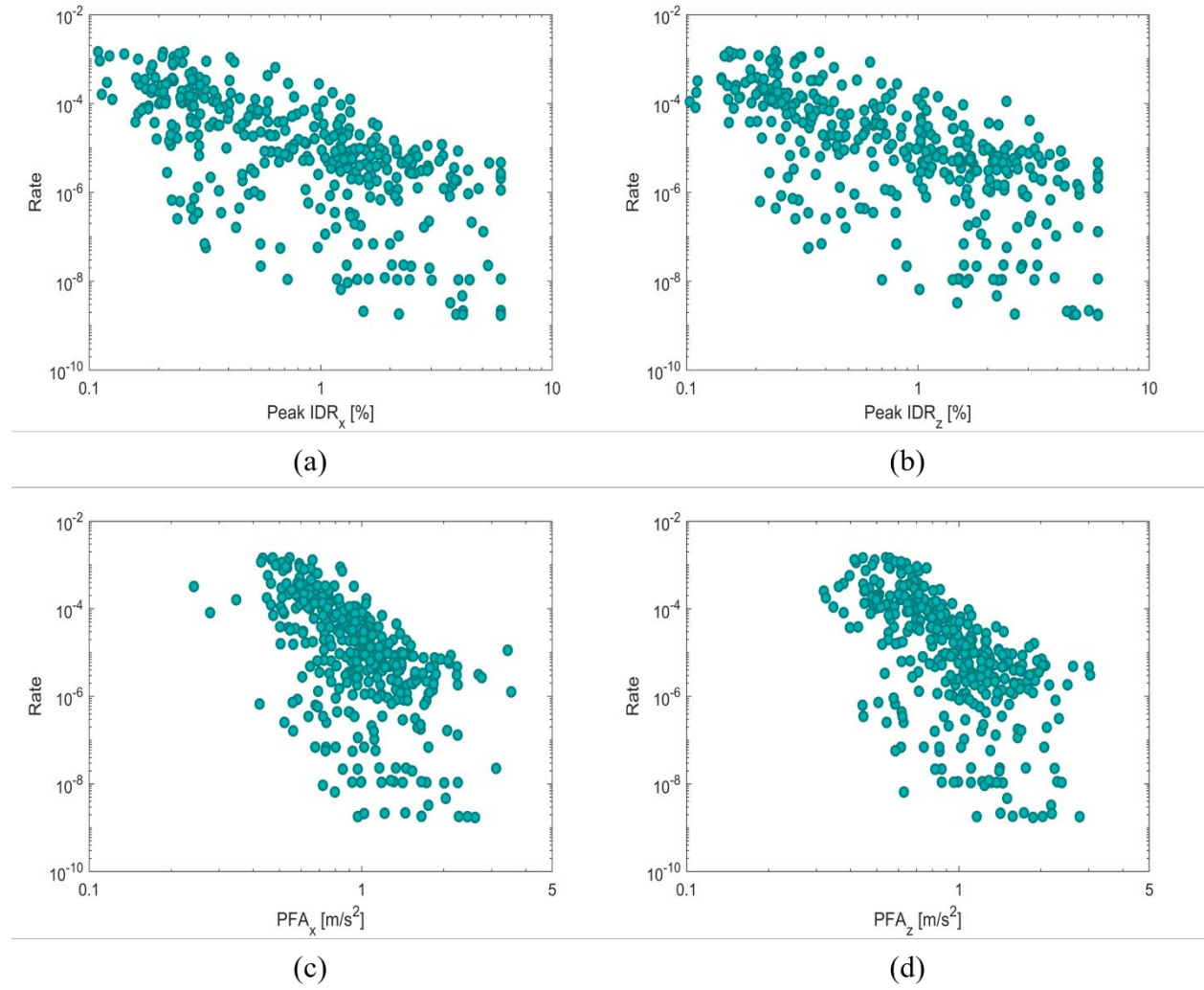


Figure 6.33 Scatter plots of rates of occurrence versus peak global EDPs from CSS runs for the 4-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction.

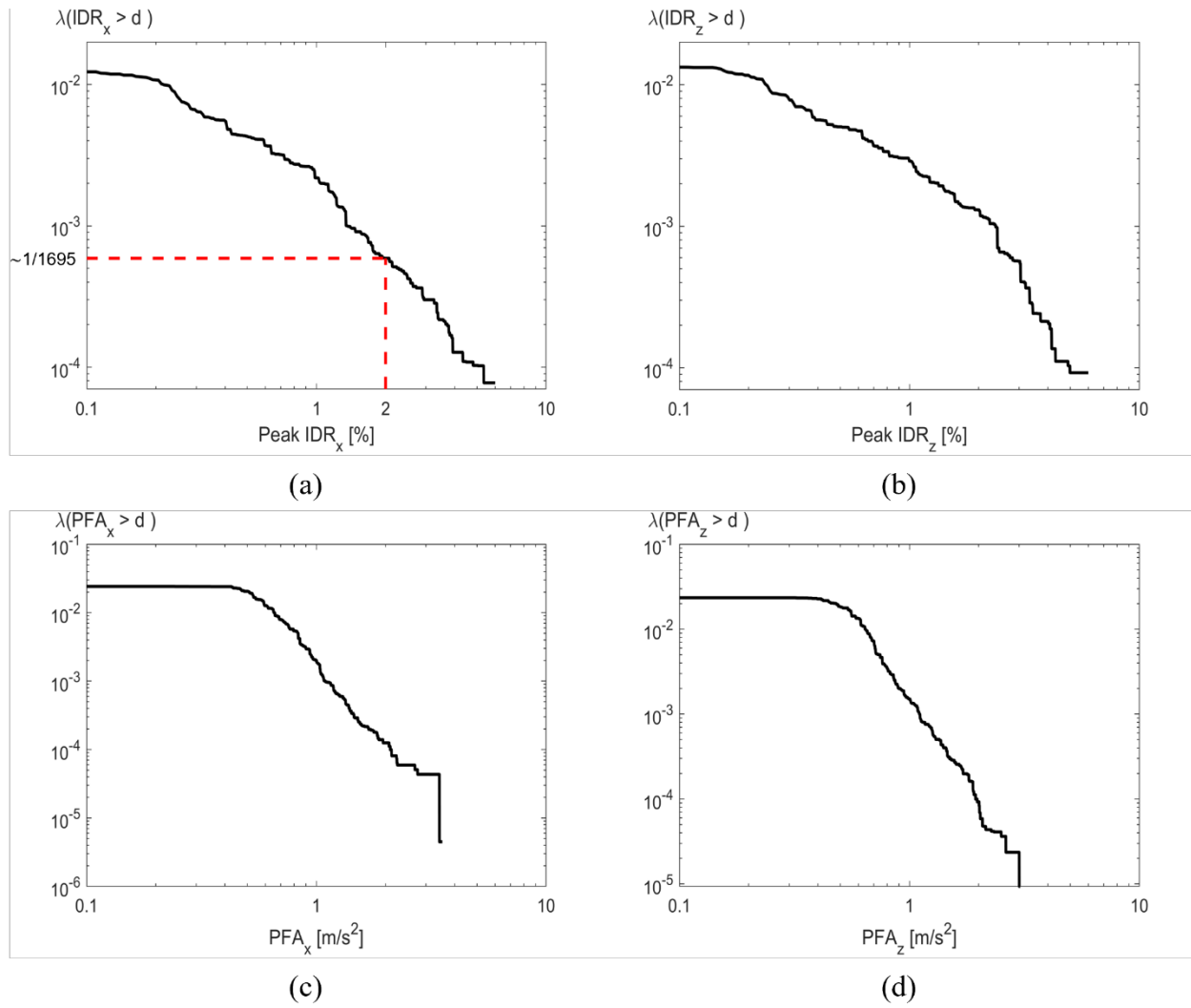


Figure 6.34 Annual rate of exceedance of peak global EDPs from CSS runs for the 4-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction.

6.1.7.2 6-story archetype building

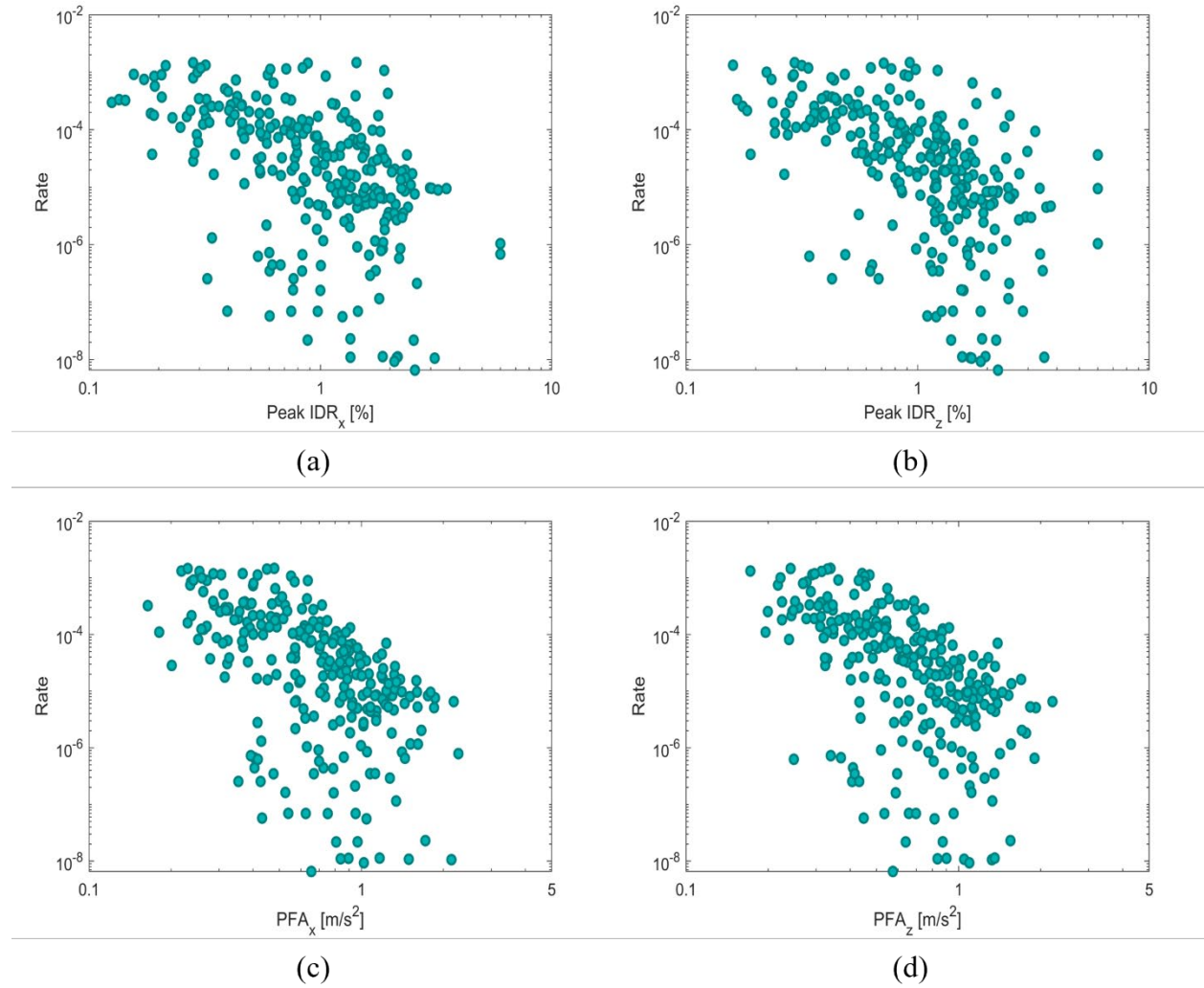


Figure 6.35 Scatter plots of rates of occurrence versus peak global EDPs from CSS runs for the 6-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction.

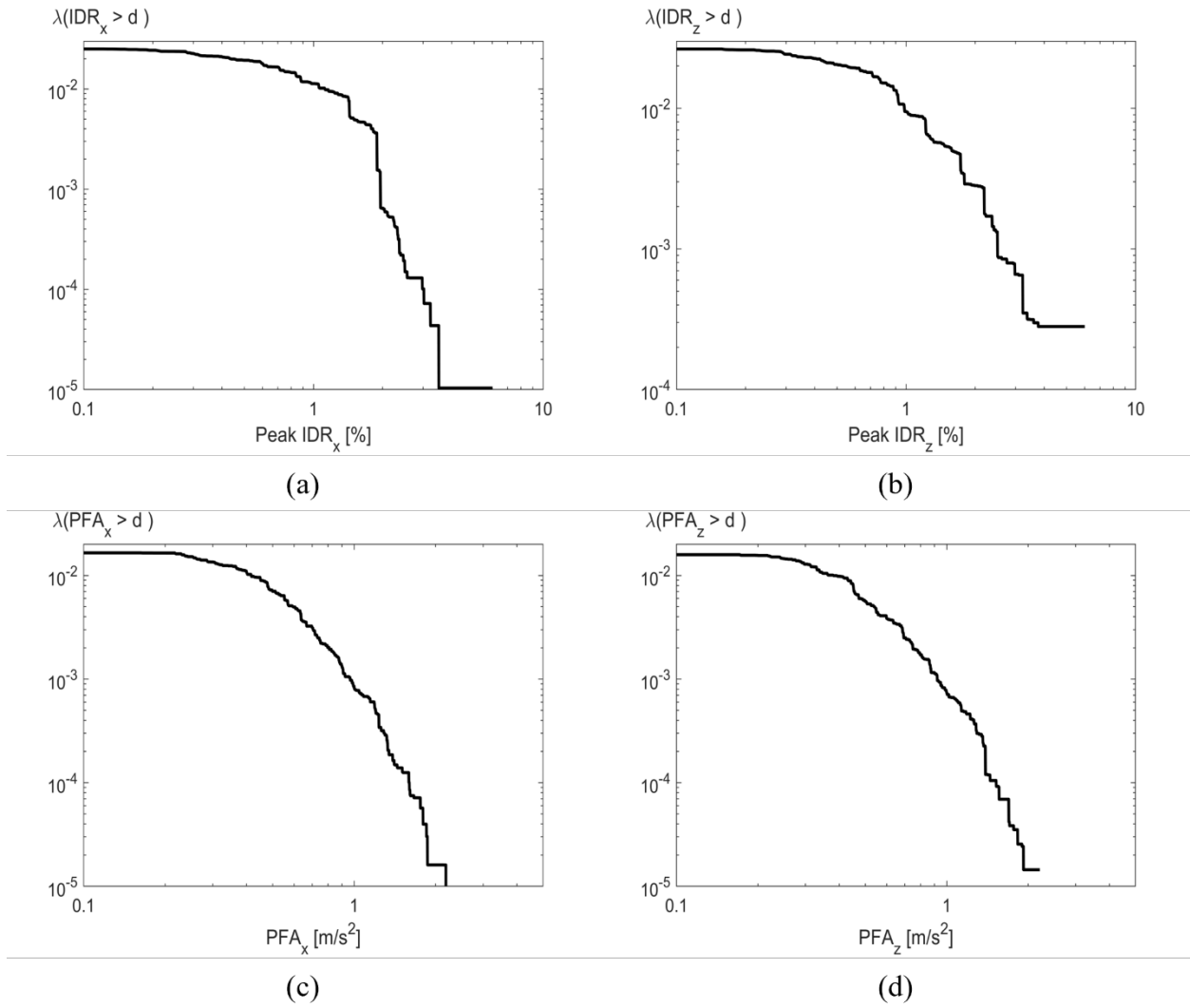


Figure 6.36 Annual rate of exceedance of peak global EDPs from CSS runs for the 6-story archetype building: (a) interstory drift ratio (IDR) in the longitudinal direction, (b) IDR in the transverse direction, (c) peak floor acceleration (PFA) in the longitudinal direction, and (d) PFA in the transverse direction.

7. Damage Analysis

As previously mentioned, performance-based assessment strives to study the performance of a given structure (or group of structures) in terms of decision variables (DVs) that are easy to understand by different stakeholders, such as monetary losses, fatalities, injuries, or downtime. For that purpose, it's necessary to have a way to translate the results from the structural analysis phase, which are usually in terms of engineer demand parameters (EDPs) that are only relevant to engineers, into those desirable decision variables. To perform the conversion from EDPs to DVs, component fragility functions are used, which estimate the probability of a given structural component attaining or exceeding one or more damage states at increasing engineer demand levels.

The primary goal of this chapter is to review the literature for previously developed fragility functions that allow the estimation of levels of damage in unreinforced masonry infill (URM) infill walls as a function of in-plane (IP) and out-of-plane (OOP) actions, and compare how the use of one set of fragility or other impacts the loss estimation of the case study buildings. This is done to study the epistemic uncertainty brought into the problem due to the lack of certainty on the models used for mapping the EDPs to different damage states.

Moreover, this section describes the fragility functions used for the other components of the case-study buildings, namely for the structural elements (i.e., beams and columns) and all other non-structural components. These fragility functions are obtained from the literature since the focus of this study is neither of these two damageable groups. Inter-story drift ratio (IDR) is used to determine the damage state of the structural components and URM infill walls. On the other hand, peak floor acceleration (PFA) is related to the performance of non-structural components.

7.1 Fragility Function for Unreinforced Masonry Infills.

One of the advantages of the PEER-PBEE methodology is the direct consideration of uncertainties in all four phases of the methodology. In the damage state determination process, that uncertainty primarily comes from the specimen-to-specimen variability, i.e., the *edp* at which a given damage state is attained or exceeded varies from specimen to specimen or from component to component. This uncertainty can be captured in a rigorous probabilistic manner by developing fragility functions, which estimate the probability of a given structural component attaining or exceeding one or more damage states at increasing engineer demand levels.

This section aims to study the epistemic uncertainty present in the damage analysis phase of the methodology due to the lack of information on how to correctly translate a given structural demand state of a component into the corresponding damage level. To this end, this section presents a literature review of the limited number of formulations of fragility functions available explicitly for masonry infill walls. These different sets of fragility functions will be later used to estimate the losses associated with these damage levels in the next chapter and quantify the variability in the final loss estimation due to the epistemic uncertainty of the masonry infill fragility functions.

As documented by Chiozzi and Miranda (2017), there is not a lot of research about fragility functions for masonry infills. To the best of the author's knowledge, only four studies have focused on this: Cardone and Perrone (2015), Sassun et al. (2016), Chiozzi and Miranda (2017), and, more recently, Del Gaudio et al. (2019). The first group of authors summarizes the results of 55 lab specimens from 19 experimental campaigns; the second group summarizes the results of 50 specimens from 14 experimental campaigns; the third group of researchers collected data from 33 experimental programs, with information for 152 specimens; and the last group collected data from 79 infill wall specimens without openings. All the studies chose inter-story drift ratio (IDR) as the engineering demand parameter to which the damage states are correlated. These four research studies are of great significance. Still, so far, no study has been done to study the impact of choosing one set of fragility functions on the loss estimation of structures.

7.1.1 Cardone and Perrone (2015)

Cardone and Perrone (2015) collected information from 19 different experimental campaigns, covering 55 infill wall specimens. They collected relevant geometrical and mechanical information about the 55 specimens, such as the type of masonry unit, mortar type, the presence of openings, and the type of loading, among other properties. Furthermore, they defined four sequential discrete damage states in terms of crack width and the damage extension throughout the infill panel. Their four damage states (DS) are in incremental order: DS1 (Light cracking), DS2 (Extensive Cracking), DS3 (Corner Crushing), and DS4 (Collapse). It's worth noting that although they described their first three DS in terms of in-plane demands, for DS4, which refers to the collapse of the infill, they considered the possibility that it might fail due to in-plane or out-of-plane demands. Figure 7.1 reproduces a figure from the original document and depicts examples of infill walls exhibiting a specific DS.

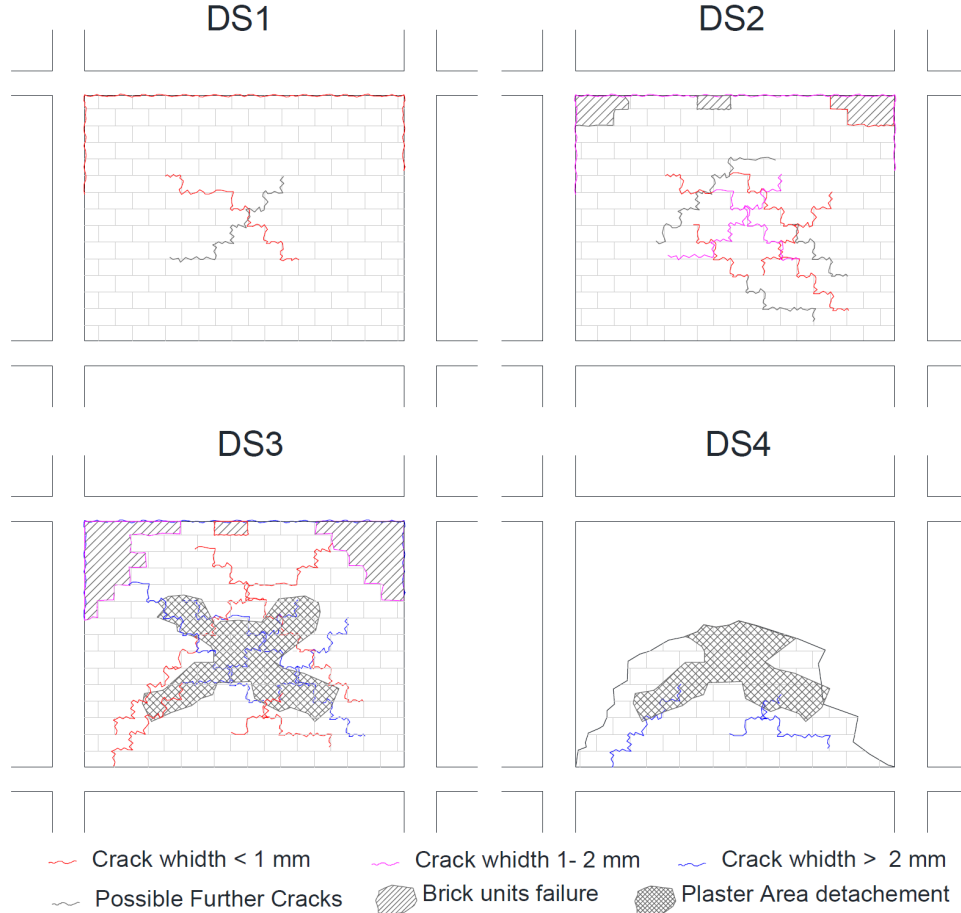


Figure 7.1 Damage states of masonry infills according to Cardone and Perrone (2015)

In their study, Cardone and Perrone (2015) chose IDR as the EDP to measure in-plane damage to the infill walls and peak floor acceleration (PFA) as the EDP to measure out-of-plane collapse. Furthermore, they reported that it was not straightforward to determine the EDP level at which a specific damage state was attained in most cases. The main reason for this is that researchers did not report enough information regarding damage levels in terms of crack patterns and size and the level at which they occurred for most experimental campaigns. Thus, when sufficient damage information was not reported, they decided to use the test's hysteresis loops or backbone curves to estimate the different levels of IDR for each DS. For example, DS1 occurred at the IDR level equivalent to 2/3 of the IDR corresponding to the peak strength. DS2 is attained at the IDR where the peak strength occurs. And DS3 and DS4 were said to occur at IDR levels where the strength capacity of the infill decreases by 20% and 50%, respectively.

As was already mentioned, they collected data on 55 laboratory specimens. For each specimen, they recorded, when possible, the IDR and PFA at which each of the four damage states occurred. With the *edp* data organized, they sought to compute drift-based fragility functions.

At this stage, a decision has to be made regarding a probability density function to use to fit the data to estimate the statistics for the fragility functions. In this study, lognormal is the distribution of choice. This decision is based on its simplicity, the fact that it's been used for decades in the industry to fit fragility data, and more importantly, it seems to fit engineering data relatively well (Porter 2019). A random variable X is said to have a lognormal distribution if $\ln(X)$ is normally distributed (Ross 1987). The probability of exceedance (POE) a given damage state dm given that a certain edp level has occurred is given by:

$$POE(DM > dm | EDP = edp) = \Phi\left(\frac{\ln(edp) - \mu_{\ln(edp)}}{\beta}\right) \quad (7.1)$$

where $\mu_{\ln(edp)}$, and β are the median and the logarithmic standard deviation of the distribution, respectively. Table 7.1 presents the statistics parameters for the different damage levels previously defined (Cardone and Perrone 2015). The resulting fragility curves from the statistical parameters presented in Table 7.1 are shown in Figure 7.2.

Table 7.1 Fragility function parameters for masonry infill walls (Cardone and Perrone 2015)

Damage Level	EDP	Median	β
DS1 (Light Cracking)		0.15	0.5
DS2 (Extensive Cracking)	IDR (%)	0.40	0.5
DS3 (Corner Crushing)		1.00	0.4
DS4 (Collapse)	IDR (%)	1.75	
	PFA (g)	0.65 g	0.35

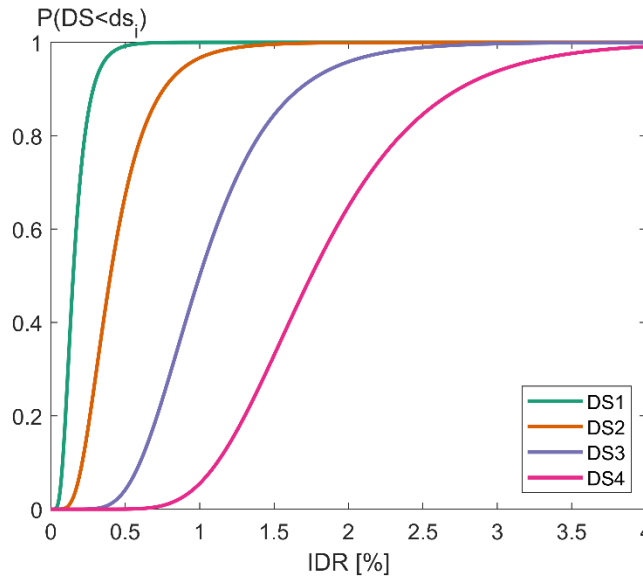


Figure 7.2 Fragility functions for masonry infill walls (Cardone and Perrone 2015)

These fragility functions are then used to estimate the probability that an infill is at a specific damage state given a set edp level ($P(DS = ds_i)$), as shown in Figure 7.3. The proposed damage states occurred in increasing order, typically called sequential damage states, this means

that the infill wall must first experience damage state 1 (DS1) before it can attain DS2, and it must enter DS2 before DS3, etc (Porter 2019). One could find then $P(DS = ds_i)$ as:

$$P(DS = ds_i | IDR_k) = \frac{P(DS \geq ds_i | IDR_k) - P(DS \geq ds_{i+1} | IDR_k)}{P(DS \geq ds_i | IDR_k)} \text{ if } 1 \leq ds_i \leq 3 \quad (7.2)$$

$$P(DS \geq ds_i | IDR_k) \text{ if } ds_i = 4$$

According to this equation, it is noted that the probability at DS4 is equal to the probability of exceedance at DS4.

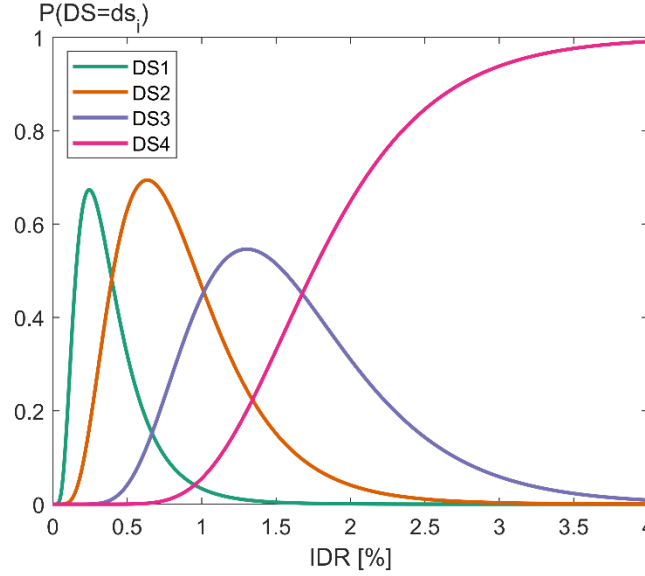


Figure 7.3 Probability of being at each DS. Adapted from Cardone and Perrone (2015)

7.1.2 Sassun et al. (2016)

Sassun et al. (2016) collected information mainly from laboratory tests of solid and hollow clay and concrete block infill panels subjected to in-plane actions. All in all, they collected information about 50 lab specimens resulting from 14 different experimental campaigns. Similar to Cardone and Perrone, they reported some basic information about the masonry infill wall characteristics, such as type of masonry unit, dimensions of the masonry unit, mortar type and its corresponding strength, the presence of openings, and the overall dimensions of the infill wall. Sassun et al. (2016) limited themselves to selecting experimental campaigns that explicitly reported visual descriptions of damage levels related to IDR. Similar to Cardone and Perrone (2015), they defined four discrete damage states based on increasing damage to the infill wall: DS1 (Operational limit state), DS2 (Damage Limitation limit state), DS3 (Life Safety or Significant Damage limit state), and DS4 (Near Collapse limit state). They referred to the first damage state as only “cosmetic” damage with no need for repairs to minor repairs; DS2 refers to the formation of diagonal cracking along the two diagonals of the panel, where the panel can be repaired with minor financial impact. When DS3 is attained, crushing and spalling of the masonry unit is widespread, furthermore, it’s considered that it is more economically advantageous to demolish and build a new panel than to repair it. When the damage state DS4 is reached, the infill wall must be fully replaced because the panel has collapsed or partially collapsed. One main difference between the study of Sassun et al.

(2016) with the Cardone and Perrone (2015) study is that they tried to obtain the *edp* level at which a given damage state was attained from the visual description of the damage and not from force-displacement response.

Table 7.2 presents the statistics parameters for the different damage levels previously defined. The probability density function of choice by Sassun et al. (2016) to fit the data for the fragility function was the log-normal distribution; as they stated in the original document, several statistical distributions fit the data well, but the lognormal distribution has been traditionally used in structural engineering to fit damage data. The resulting fragility curves from the statistical parameters presented in Table 7.2 are shown in Figure 7.4.

Table 7.2 Fragility function parameters for masonry infill walls (Sassun et al. 2016).

Damage Level	EDP	Median	β
DS1 (Operational)	IDR (%)	0.18	0.52
DS2 (Damage Limitation)		0.46	0.54
DS3 (Life Safety)		1.05	0.40
DS4 (Ultimate)		1.88	0.38

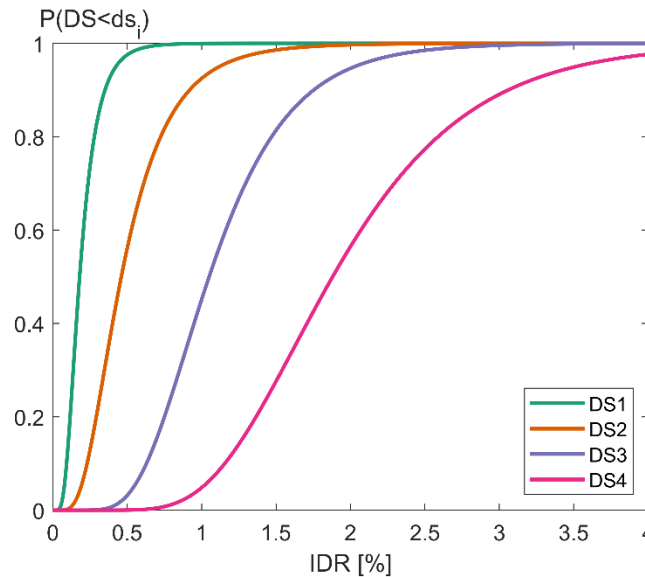


Figure 7.4 Fragility functions for masonry infill walls (Sassun et al. 2016).

These fragility functions are then used to estimate the probability that an infill is at a specific damage state given a set *edp* level ($P(DS = ds_i)$), as shown in Figure 7.5. The proposed damage states occurred in increasing order, as was the case with the damage state definition by Cardone and Perrone (2015). Thus, $P(DS = ds_i)$ is obtained just as before, by using Equation (7.2).

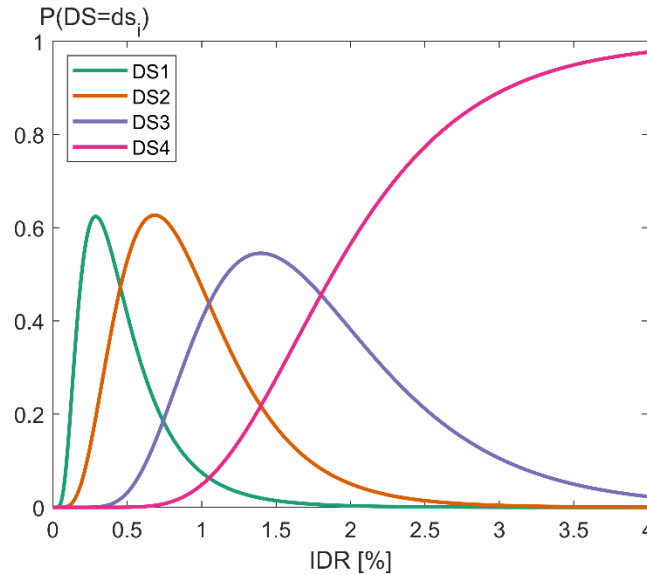


Figure 7.5 Probability of being at each DS. Adapted from Sassun et al. (2016).

7.1.3 Chiozzi and Miranda (2017)

Chiozzi and Miranda (2017), following the work previously done by Cardone and Perrone and Sassun et al., also developed fragility functions for URM infill walls based on IDR as the *edp* of interest. They collected data from 33 experimental programs, with damage information for 152 specimens, a considerable increase in the sample size compared to the previous studies, which only had approximately 50 specimens each. Chiozzi and Miranda (2017) defined only three damage states, which were specified based on the repair actions required and their respective cost due to attained said damage level. The definition of their three damage states was mainly based on crack patterns and crack widths. The first damage state DS_1 refers to light cracking, they defined it as the presence of small cracks less than 2 mm wide, furthermore they don't expect joint sliding or crushing of the masonry units to occur at this stage. The second damage state DS_2 refers to moderate cracking, and it was defined as crack more than 2 mm wide spreading through the masonry units and mortar, furthermore limited joint sliding and crushing of the masonry units is to be expected. The third and last damage state DS_3 refers to heavy cracking, with the widespread presence of wide diagonal cracks more than 4 mm wide, furthermore considerable joint sliding and widespread crushing of the masonry units it's to be expected at this damage level. Chiozzi and Miranda (2017) collected and reported relevant information for each of the 152 specimens they studied, ranging from geometrical and mechanical properties, amongst others. Moreover, they inferred the IDR at which each damage level was attained for the specimens under review based on a detailed visual description of the damage accompanying the laboratory test they studied.

After the IDR at which the three damage states were attained, a plotting probability was chosen to estimate cumulative frequency distributions from the experimental data. For this purpose, a cumulative frequency distribution was obtained by plotting ascending-sorted *edps* at

which each of the damage states was attained against the following plotting probability F_i (Aslani and Miranda 2005, Chiozzi and Miranda 2017).

$$F_i = \frac{(i - 3/8)}{(N + 1/4)} \quad (7.1)$$

where i is the position after sorting the data in ascending order, and N is the total number of data points in the sample.

Second, a decision had to be made in terms of a probability density function to use to fit the data to estimate the statistics for the fragility functions. Similar to the previous two studies, Chiozzi and Miranda (2017) also chose the lognormal distribution. Table 7.3 presents the statistics parameters for the different damage levels previously defined. The resulting fragility curves from the statistical parameters presented in Table 7.3 are shown in Figure 7.6.

Table 7.3 Fragility function parameters for masonry infill walls (Chiozzi and Miranda 2017).

Damage Level	EDP	Median	β
DS1 (Light cracking)	IDR (%)	0.125	0.325
DS2 (Moderate cracking)		0.327	0.278
DS3 (Heavy cracking)		0.820	0.320

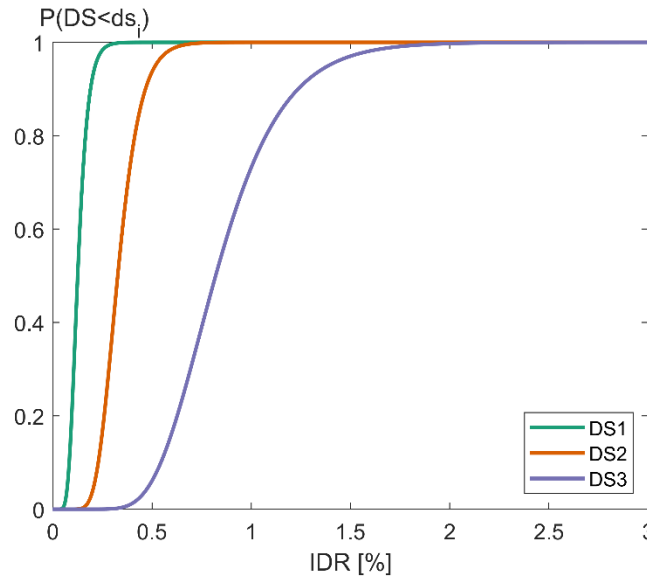


Figure 7.6 Fragility functions for masonry infill walls (Chiozzi and Miranda 2017).

These fragility functions are then used to estimate the probability that an infill is at a specific damage state given a set edp level ($P(DS = ds_i)$), as shown in Figure 7.7. The proposed damage states occurred in increasing order. Thus, $P(DS = ds_i)$ is obtained just as before, by using Equation (7.2).

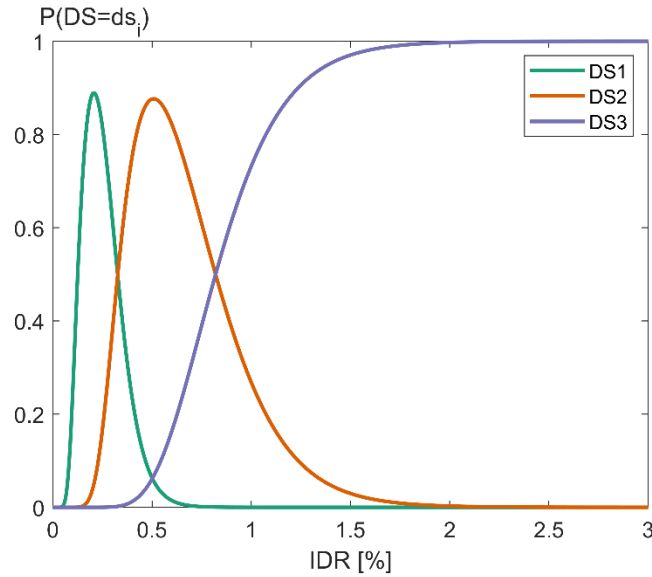


Figure 7.7 Probability of being at each DS. Adapted from Chiozzi and Miranda (2017).

7.1.4 Del Gaudio et al. (2019)

Del Gaudio et al. (2019) collected data from 105 infill wall specimens from 38 laboratory tests. It's worth noting that they sought to find specimens representative of what one might find in and around Italy and the Mediterranean. Like the previous groups, they summarized the relevant geometrical and mechanical information of all 105 specimens, including the direction of the holes within the masonry units. Of the 105 sets, 79 were infill walls without openings, and 26 had openings. In their efforts to use reliable information without the need to infer a lot, they only considered specimens from experimental campaigns where there was a complete characterization of the damage evolution was present (Del Gaudio et al. 2019). Furthermore, they defined four sequential discrete damage states, very similar to those described by (Cardone and Perrone 2015), which are mainly in terms of crack width and the damage extension throughout the infill panel. As usual, they used a lognormal distribution to fit the damage data and obtained IDR-based fragility functions. Table 7.4 presents the statistics parameters for the different damage levels defined in their studies. The resulting fragility curves from the statistical parameters presented in Table 7.4 are shown in Figure 7.8.

Table 7.4 Fragility function parameters for masonry infill walls (Del Gaudio et al. 2019).

Damage Level	EDP	Median	β
DS1 (light cracking)	IDR (%)	0.08	0.70
DS2 (extensive cracking)		0.32	0.40
DS3 (corner crushing)		0.94	0.40
DS4 (collapse)		1.78	0.40

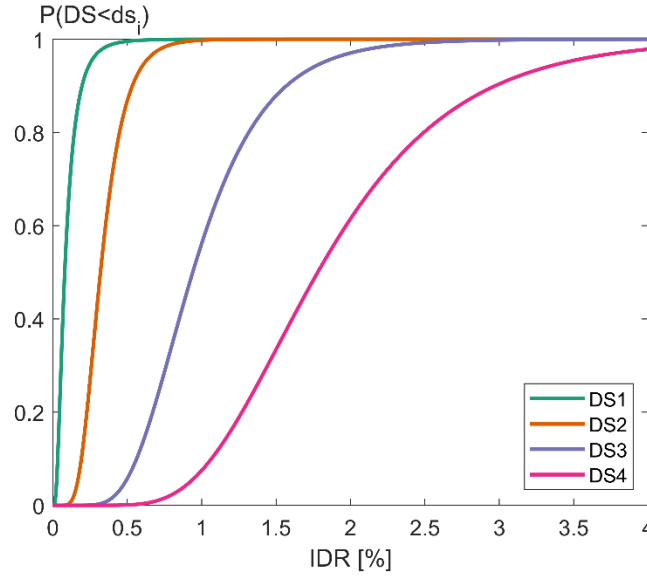


Figure 7.8 Fragility functions for masonry infill walls (Del Gaudio et al. 2019).

These fragility functions are then used to estimate the probability that an infill is at a specific damage state given a set *edp* level ($P(DS = ds_i)$), as shown in Figure 7.9. The proposed damage states occurred in increasing order. Thus, $P(DS = ds_i)$ is obtained just as before, by using Equation (7.2).

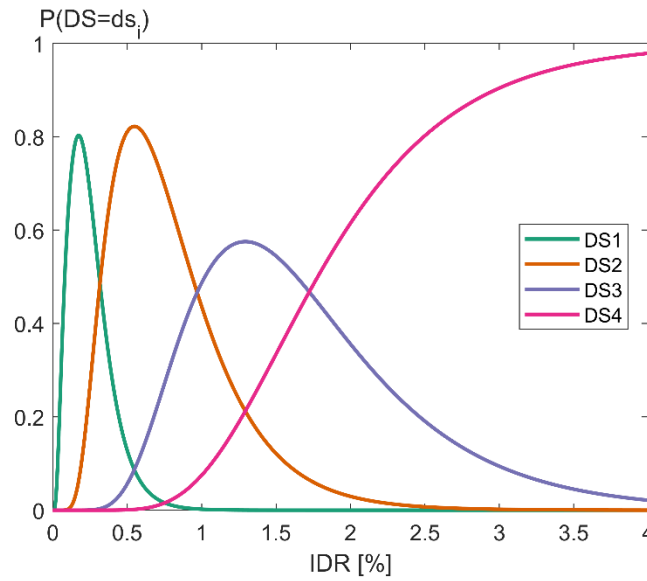


Figure 7.9 Probability of being at each DS. Adapted from Del Gaudio et al. (2019).

To close out this section, and to give some perspective to the reader on how these four sets that are currently available in the literature for fragility functions for URM infill walls compare to each other, Figure 7.10 and Figure 7.11 show how these formulations compare with each other. Namely, Figure 7.10 presents the comparison for each damage state (DS) of the fragility functions

for all four formulations; and Figure 7.11 shows the comparison for each DS of the probability that an infill is a specific damage state for all four formulations.

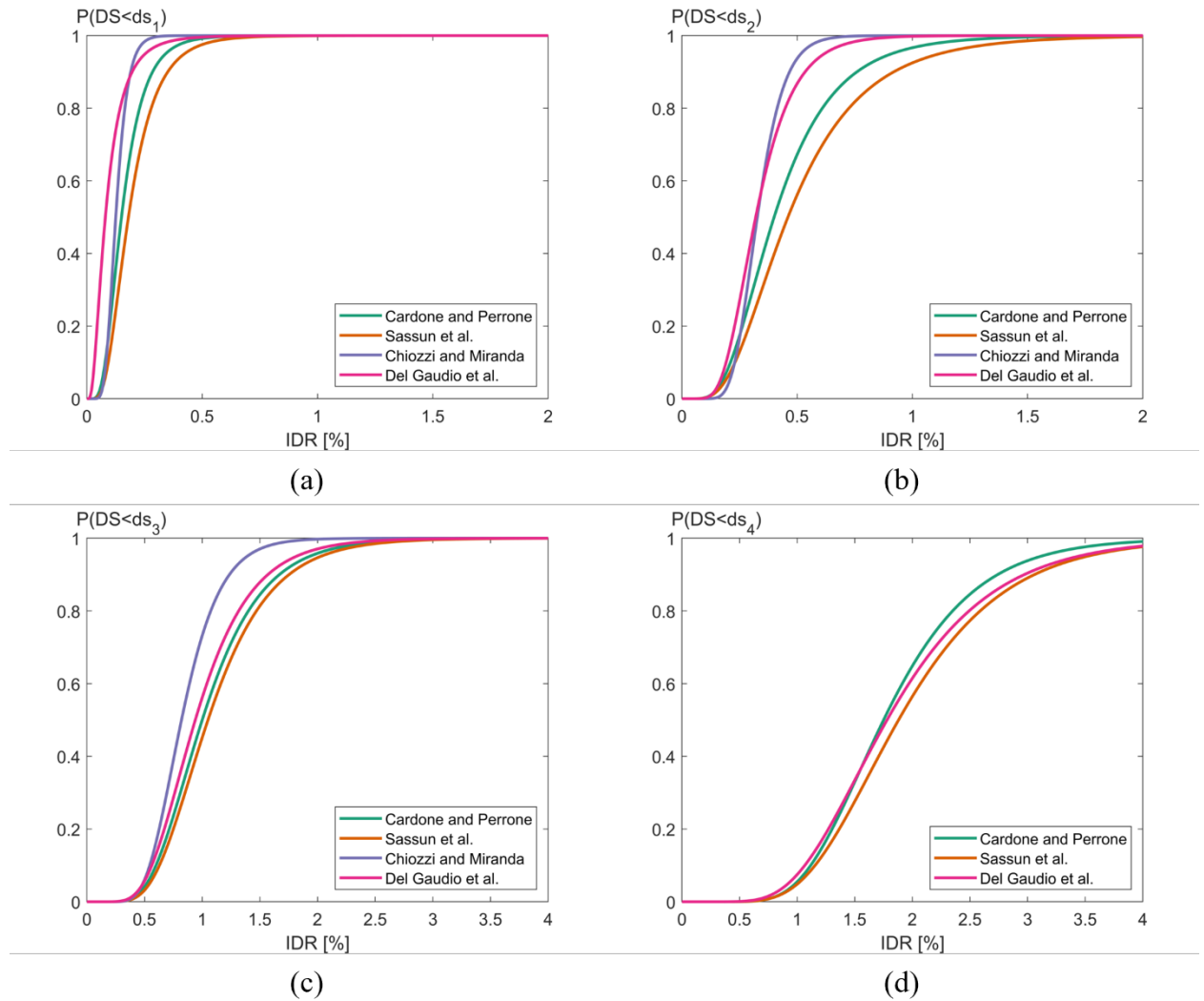


Figure 7.10 Comparison of fragility functions for masonry infill walls for (a) DS1, (b) DS2, (c) DS3, and (d) DS4.

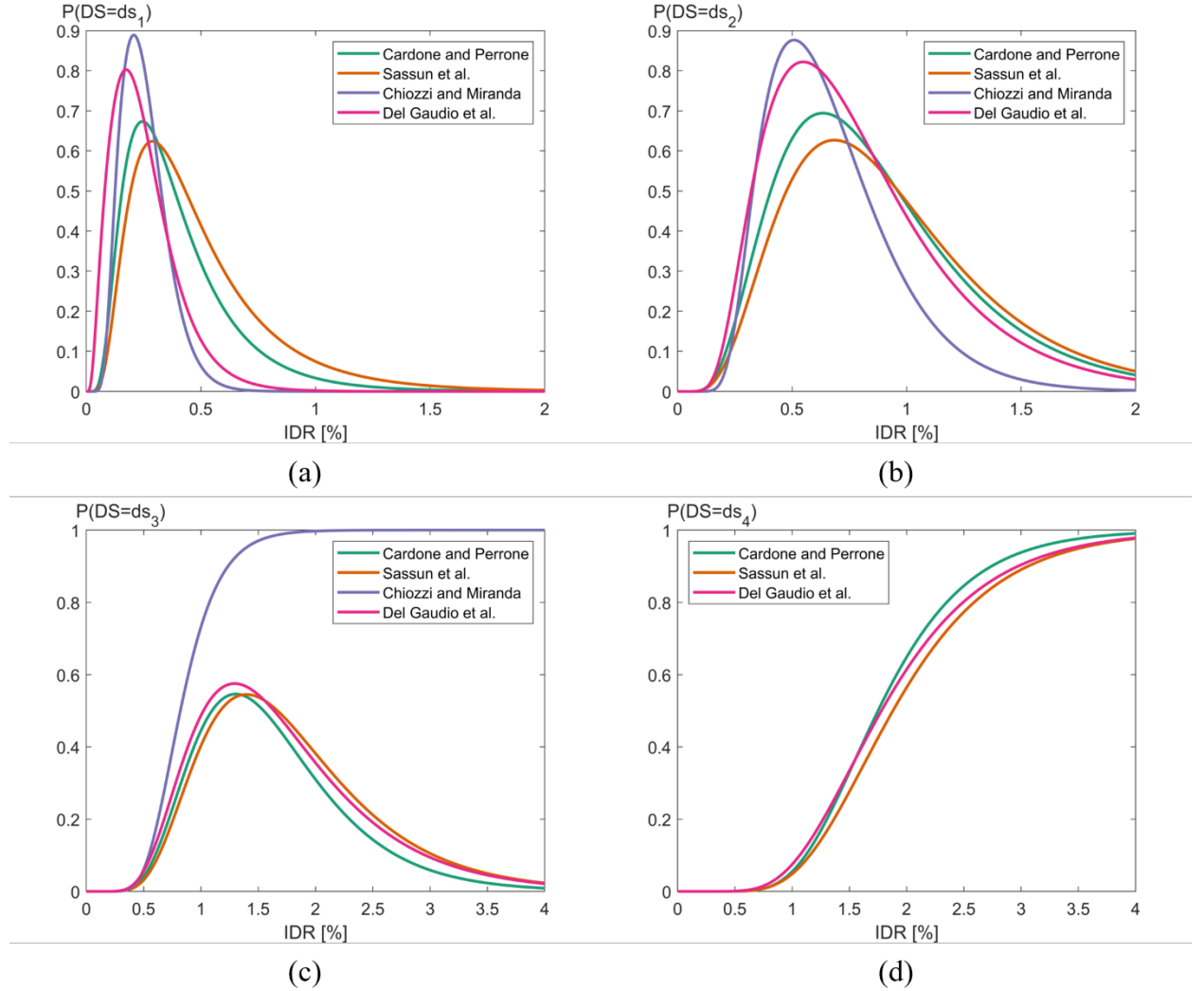


Figure 7.11 Comparison of probability of being at a specific damage state, for (a) DS1, (b) DS2, (c) DS3, and (d) DS4.

7.2 Fragility Function for other components.

Fragility functions used in this section are obtained from the literature for the remaining damageable groups previously discussed, i.e., structural components and non-structural components. Structural components refer to the lateral load resisting system, namely to the RC moment frame elements. Non-structural components refer to everything else that is not the RC moment frames or the URM infill walls, including the building's content.

A lognormal distribution is used to fit the fragility functions of these two damageable groups; as stated before, this decision was based on the fact that it's been used for decades in the industry to fit fragility data and because it fits engineering data relatively well (Porter 2019). IDR

is used to determine the damage state of the structural components. On the other hand, PFA is related to the performance of non-structural components.

Two different sets of fragility functions need to be defined for the structural components, which refer to the lateral load resisting system. One refers to reinforced concrete frames with light transverse reinforcement in their columns, which will be used in the damage and loss estimation of the 6-story non-ductile archetype building. And another set refers to ductile reinforced concrete frames, which will be used in the loss estimation of the 4-story ductile archetype building.

For the first group, referring to nonductile RC concrete frames, fragility functions are defined following the work of (Aslani 2005). In his work, Aslani (2005) defined four damage states. The first damage state DS_1 refers to light cracking, it was defined as the presence of small cracks that may need light repair work to restore to a pre-damage state. The second damage state DS_2 refers to severe cracking, and it was defined as the presence of larger and deeper cracks. The third damage state DS_3 refers to shear failure, which is a common failure mode in a lightly reinforced column subjected to lateral loads. The fourth and last damage state DS_4 refers to Axial Failure, which refers to a partial or complete loss of vertical carrying capacity for the column within the RC moment frame, which can lead to local collapse or progressive total collapse (Aslani 2005).

For the second group, referring to ductile RC concrete frames, the fragility function information came from the Performance Assessment Calculation Tool (PACT) (FEMA 2018), a software developed as part of the FEMA P-58 project. PACT has a database of fragility functions for common structural and non-structural components present in buildings in the United States. The fragility information from an ACI 318 conforming intermediate moment frame concrete column and beam of 24 in x 24 in was used from its database, as they are somehow representative of the cross-sectional dimensions of the prototype building components. For this particular type of component, three damage states were defined. The first damage state DS_1 refers to residual cracks present in the beams or joints, with no significant spalling present at this stage. The second damage state DS_2 refers to wider and deeper residual cracks present in the beams or joints, typically with crack widths larger than 0.06 in, minor cover spalling is present. The third damage state DS_3 refers to ever-growing crack widths, with significant cover concrete spalling present. Furthermore, fracture, buckling of the reinforcement and crushing of the core concrete may appear beyond this limit state .

Values to determine the fragility of the non-structural components are based on damage information (Aslani 2005) reported in Table 4.20 of his Ph.D. dissertation for the “Acceleration sensitive non-structural component.” Four damage states ranging from slight to complete damage were defined. Notice the high value of the logarithmic standard deviation, highlighting the large uncertainty that exists in estimating the damage state of this fragility group.

Table 7.5 presents the median and COV values for the damage levels of both damageable groups. Fragility curves for the structural components and non-structural components are presented in Figure 7.10 and Figure 7.11, respectively.

Table 7.5 Median and COV of EDPs for different damage levels.

Component	Damage Level	EDP	Median	σ_{Ln}
Structural (Ductile)	DS_1 (Slight damage)	IDR	1.75%	0.40
	DS_2 (Moderate damage)		2.50%	0.30
	DS_3 (Severe damage)		3.50%	0.30
Structural (Non-ductile)	DS_1 (Light cracking)	IDR	0.35%	0.37
	DS_2 (Severe cracking)		0.71%	0.44
	DS_3 (Shear failure)		2.00%	0.58
	DS_4 (Axial failure)		3.10%	0.63
Non-structural	DS_1 (Slight damage)	PFA	0.25 g	0.60
	DS_2 (Moderate damage)		0.50 g	0.60
	DS_3 (Extensive damage)		1.00 g	0.60
	DS_4 (Complete damage)		2.00 g	0.60

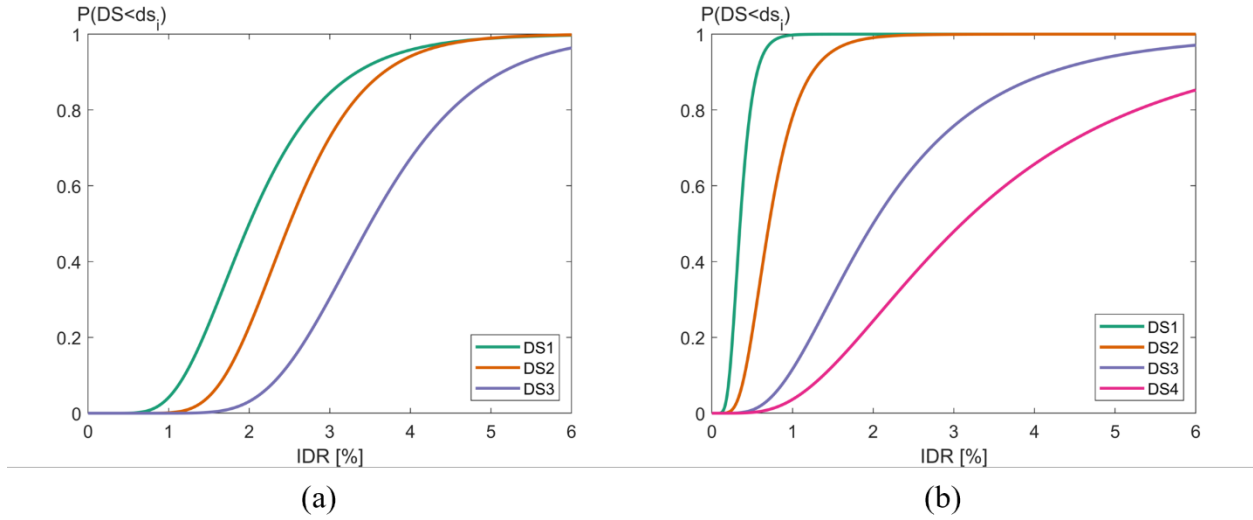


Figure 7.12 Fragility functions for (a) Ductile structural components. (b) Non-ductile structural components.

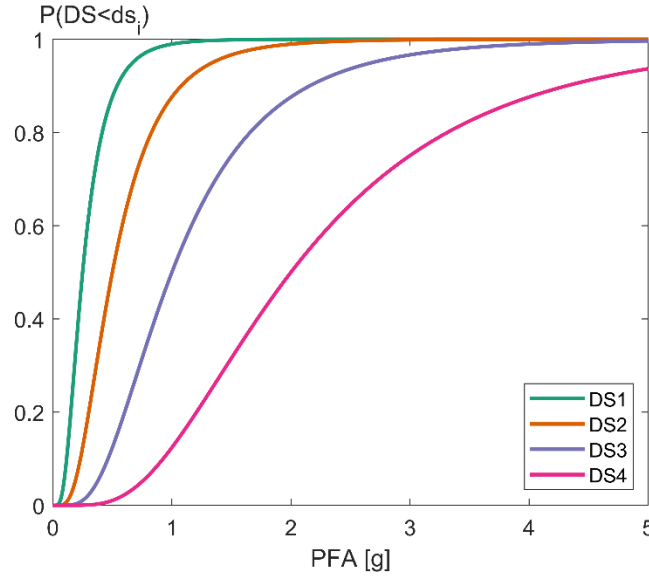


Figure 7.13 Fragility functions for non-structural components.

These fragility functions are then used to estimate the probability that a component is at a specific damage state given a set *edp* level ($P(DS = ds_i)$). The proposed damage states occurred in increasing order. Thus, $P(DS = ds_i)$ is obtained using Equation (7.2). The probability of being at each damage state for the structural components and non-structural components are presented in Figure 7.10 and Figure 7.11, respectively.

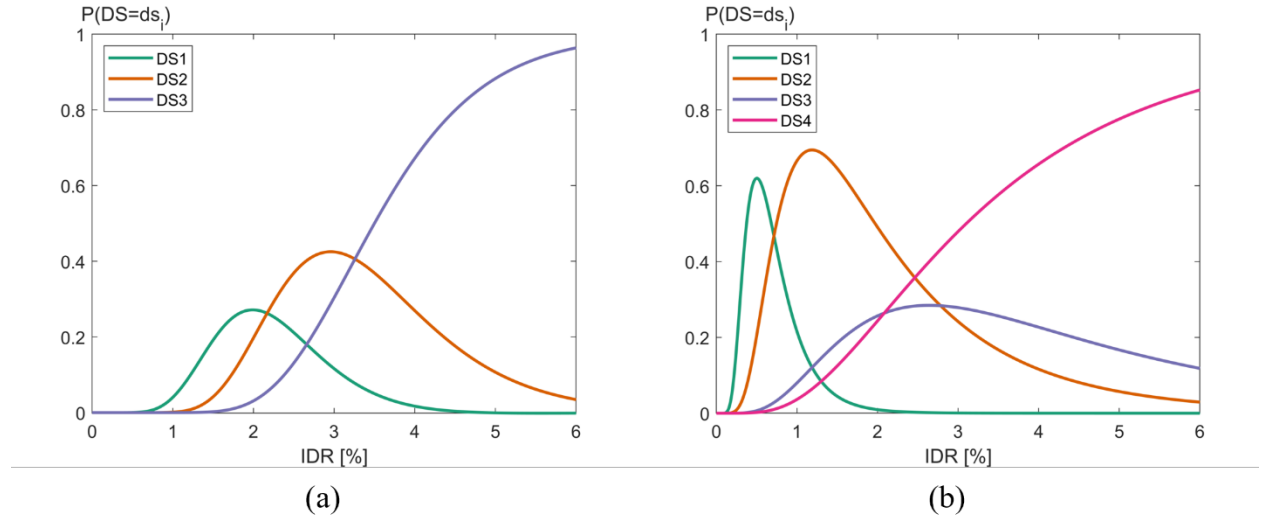


Figure 7.14 Probability of being at each damage state for (a) Ductile structural components. (b) Non-ductile structural components.

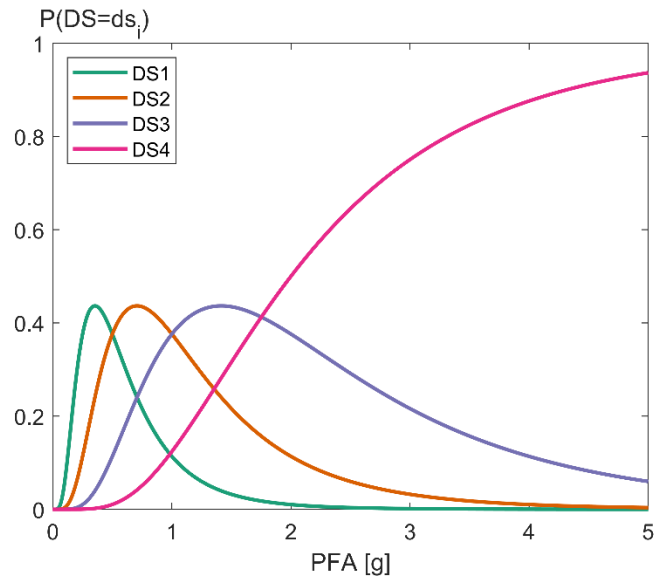


Figure 7.15 Probability of being at each damage state for non-structural components.

8. Loss Analysis

After completing the first three phases of the PBEE methodology, a Loss Analysis needs to be performed to convert the results obtained in the previous stage, from Chapter 7, into *Decision Variables (DVs)* that are of direct interest to the stakeholders. Common DVs used in PBEE analysis are the number of fatalities, injuries, economic losses, and downtime. For the present case study buildings, the DV of interest is economic losses. To convert the damage information into economic losses, loss functions need to be developed for all the components that make up the case-study buildings. Loss functions relate the probability of exceedance of a specific economic loss level given that a particular damage state has occurred.

Because loss functions are available for most structural and nonstructural components, this section primarily aims at developing loss functions for the URM infill walls. Herein, four sets of loss functions one for each fragility function derived from the literature in the previous chapter, will be developed.. This allows studying the epistemic uncertainty present in the loss analysis phase of the methodology due to the lack of information on how to correctly translate a given structural demand state of a component into economic losses due to such demands. To this end, this section presents loss functions that are developed in close connection to a literature review of the limited number of formulations of fragility functions available explicitly for masonry infill walls. These different sets of loss functions will be later used to estimate the performance of the two case study buildings in terms of the decision variable of interest and to quantify the variability in our final loss estimation due to the epistemic uncertainty of the masonry infill fragility and loss functions.

Moreover, this section describes the loss functions used for the other components of the case-study buildings, namely for the structural elements (i.e., beams and columns) and all other non-structural components. It's worth noting that all loss functions developed here were obtained using cost estimates that are representative of local prices for the city of Armenia, where the two archetype buildings are located.

8.1 Loss Function for Unreinforced Masonry Infills

In Chapter 7, four different sets of fragility functions for URM infill walls were presented. In the current section, loss functions associated with each of the damage states for all the fragility function sets presented will be developed. As mentioned earlier, the estimated costs of repairs will be obtained using local prices representative of those one could find in the city of Armenia, Colombia. The four works reviewed in the previous chapter defined their respective damage states based on specific repairs needed to restore the infill panel to its pre-damage state (Cardone and Perrone 2015). This is a key feature in the loss estimation of the entire building due to earthquake loads while using the PEER PBEE methodology.

Three out of the four works reviewed presented repair cost estimates for each respective damage state; Chiozzi and Miranda (2017) did not go beyond developing fragility functions. In the following subsections, those works that reported repair cost estimates are reviewed and adapted

to the Colombian context; specifically, the repair actions described there would be adjusted to local practices. Due to this adjustment, some repair actions may look different from those found in the original works, some actions will not appear, and some new will be added.

Following the work of the literature reviewed here and what is typically done in practice, all loss functions are assumed to follow a lognormal distribution, which is also in line with guidelines by FEMA P-58 (Cardone and Perrone 2015).

8.1.1 Cardone and Perrone (2015)

Cardone and Perrone (2015) described concrete repair actions and associated preliminary and supplementary activities needed once each damage state was attained to restore the infill panel to its pre-damage state (basically to a new-like state); the repair actions they proposed are presented in Table 8.1. The unit costs associated with each of the repair actions mentioned in Table 8.1 are also presented, this unit costs were obtained using manual costs available for the Colombian market as well as conversations with local contractors, it is important to mention that these and all other unit costs presented on this section include labor costs already in the calculation. To estimate the median cost of the total repair of the infill walls for each damage estate, it is necessary to determine the extent of the affected area to establish the total cost of repairing each infill panel. For all the activities of DS1, the total area of the panel $H_w \times L_w$ is considered. For the first two repair actions of DS2, namely remove broken bricks and restore infill using new brick, a range between 5-20% of the total area of the wall is considered; for the remaining actions, the total area of the panel is considered. It's worth noting that for the Colombian context, the repair actions for DS3 and DS4 proposed are virtually the same in terms of costs. For both last damages states, the total area of the wall is considered to estimate the total cost of repairing one panel. In their work, Cardone and Perrone (2015) estimated the dispersion (β) of the respective repair cost for each damage state, they reported values of 0.22, 0.44, 0.44, and 0.44 for the expected dispersion in the repair costs for DS1, DS2, DS3, and DS4, respectively. Unlike the authors of the original work, herein, no discount due to economy of scale will be included in the loss estimation of this component or any other. Figure 8.1 presents loss functions for all four damage estates following the work by Cardone and Perrone (2015) but adapted to the context of the two case study buildings.

Table 8.1 Unit cost of repair actions required to restore the infill to its pre-damage state. Modified from Cardone and Perrone (2015).

Damage state	Repair action	Unit	Cost (\$/unit)
DS1 (Light cracking)	Remove plaster along the upper (lateral) edge of the infill and across main diagonal cracks	m ²	3.5
	Prepare masonry surface (brushing, low-pressure water cleaning, etc.)	m ²	2.2
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS2 (Extensive cracking)	Remove broken and chipped bricks (if any)	m ³	65.3
	Restore infill/partition using new bricks	m ²	43.65
	Remove plaster along the upper and lateral edge of the infill and across main diagonal cracks	m ²	3.5
	Prepare masonry surface (brushing, low-pressure water cleaning, etc.)	m ²	2.2
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS3 (Corner crushing)	Demolish damaged infill/partition	m ³	65.3
	Build new infill/partition	m ²	43.65
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS4 (Collapse)	Demolish damaged Infill	m ³	65.3
	Rebuild Infill	m ²	43.65
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8

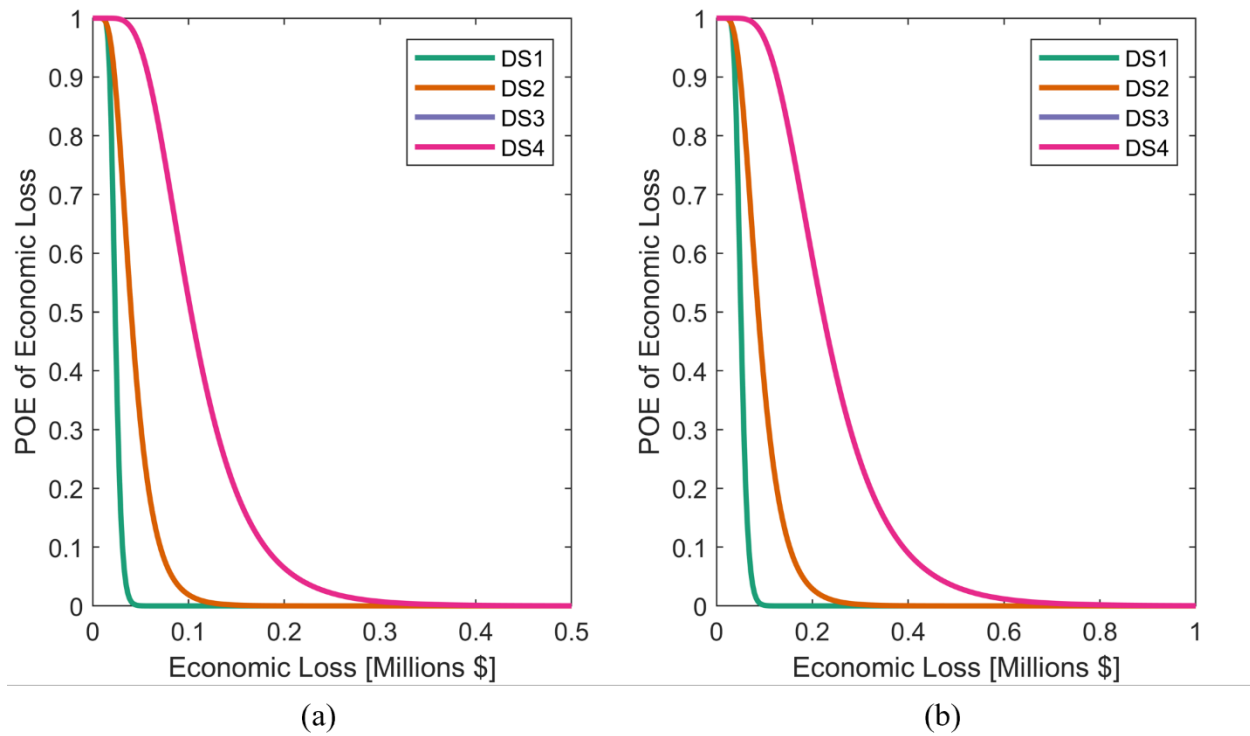


Figure 8.1 Loss functions for URM infill walls of (a) the 4-story ductile and (b) the 6-story non-ductile case study buildings. Adapted from Cardone and Perrone (2015).

8.1.2 Sassun et al. (2016)

If one pays close attention to the work by Sassun et al. (2016), their damage state definition and the required repair actions for each DS are fairly similar to those proposed by Cardone and Perrone (2015). In fact, after removing the repair actions that were not applicable or relevant in the Colombian context, the remaining repair actions for all four damage states are virtually the same for Sassun et al. and Cardone and Perrone, thus the median repair costs per square meter are the same for all four damage states. The repair actions and the unit costs associated with each repair actions they proposed are presented in Table 8.2. On the other hand, unlike Cardone and Perrone, Sassun et al. (2016) do not give any estimates for the dispersion of these costs. Due to the lack of knowledge, a large value of dispersion ($\beta = 0.40$) is chosen. As before, a lognormal distribution is used to fit the loss data. Figure 8.2 presents loss functions for all four damage estates following the work by Sassun et al. (2016) but adapted to the context of the two case study buildings.

Table 8.2 Unit cost of repair actions required to restore the infill to its pre-damage state. Modified from Sassun et al. (2016).

Damage state	Repair action	Unit	Cost (\$/unit)
DS1 (Operational)	Scraping for removal of old painting	m ²	3.5
	Prepare masonry surface	m ²	2.2
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS2 (Damage Limitation)	Demolition of broken masonry units	m ²	65.3
	Masonry reconstruction	m ²	43.65
	Scraping for removal of old painting	m ²	3.5
	Prepare masonry surface	m ²	2.2
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS3 (Life Safety)	Demolition of the masonry infill wall	m ²	65.3
	Masonry reconstruction	m ²	43.65
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
	Supplementary activities	m ²	10.0
DS4 (Ultimate)	Demolition of the masonry infill wall	m ²	65.3
	Masonry reconstruction	m ²	43.65
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
	Supplementary activities	m ²	10.0

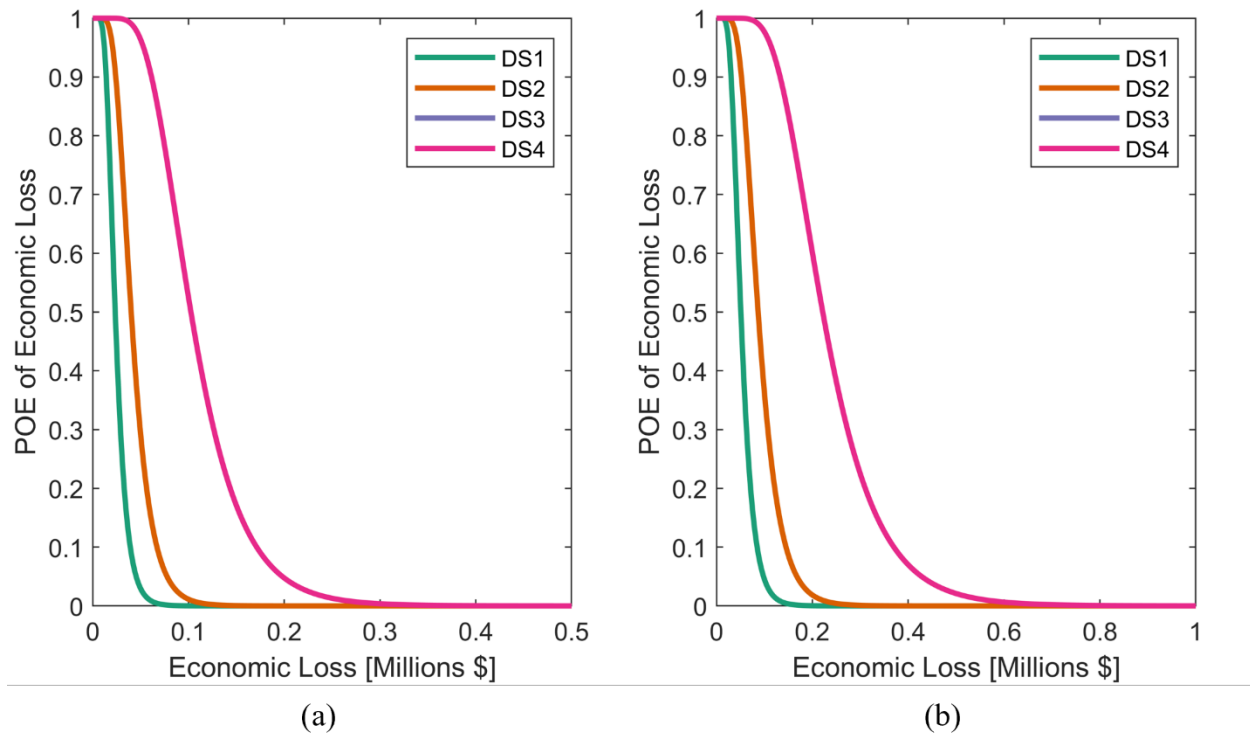


Figure 8.2 Loss functions for URM infill walls for (a) the 4-story ductile and (b) the 6-story non-ductile case study buildings. Adapted from Sassun et al. (2016).

8.1.3 Chiozzi and Miranda (2017)

Chiozzi and Miranda (2017) did not report any loss functions associated to the URM fragility functions they developed. But based on the information available for the description of the damage states they used, a set of repair actions for each DS were proposed. Furthermore, unit cost for such repair actions were also estimated based on construction cost manual for Colombia. The repair actions and the unit costs associated with each repair actions they proposed are presented in Table 8.3. As no dispersion values for the repair costs are known, a large value of dispersion ($\beta = 0.40$) is chosen. As before, a lognormal distribution is used to fit the loss data. Figure 8.3 presents loss functions for all three damage estates following the work by Chiozzi and Miranda (2017) regarding developing fragility functions for URM infill walls.

Table 8.3 Unit cost of repair actions required to restore the infill to its pre-damage state.

Damage state	Repair action	Unit	Cost (\$/unit)
DS1 (Light cracking)	Scraping for removal of old painting	m ²	3.5
	Plastering the visible cracks	m ²	15.2
	Paint	m ²	9.8
DS2 (Moderate Cracking)	Demolition of broken masonry units	m ³	65.3
	Local Masonry reconstruction	m ²	43.65
	Scraping for removal of old painting and plaster	m ²	3.5
	Prepare masonry surface	m ²	2.2
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
DS3 (Heavy cracking)	Demolition of the masonry infill wall	m ²	65.3
	Masonry reconstruction	m ²	43.65
	Patch new plaster	m ²	15.2
	Paint	m ²	9.8
	Supplementary activities	m ²	10.0

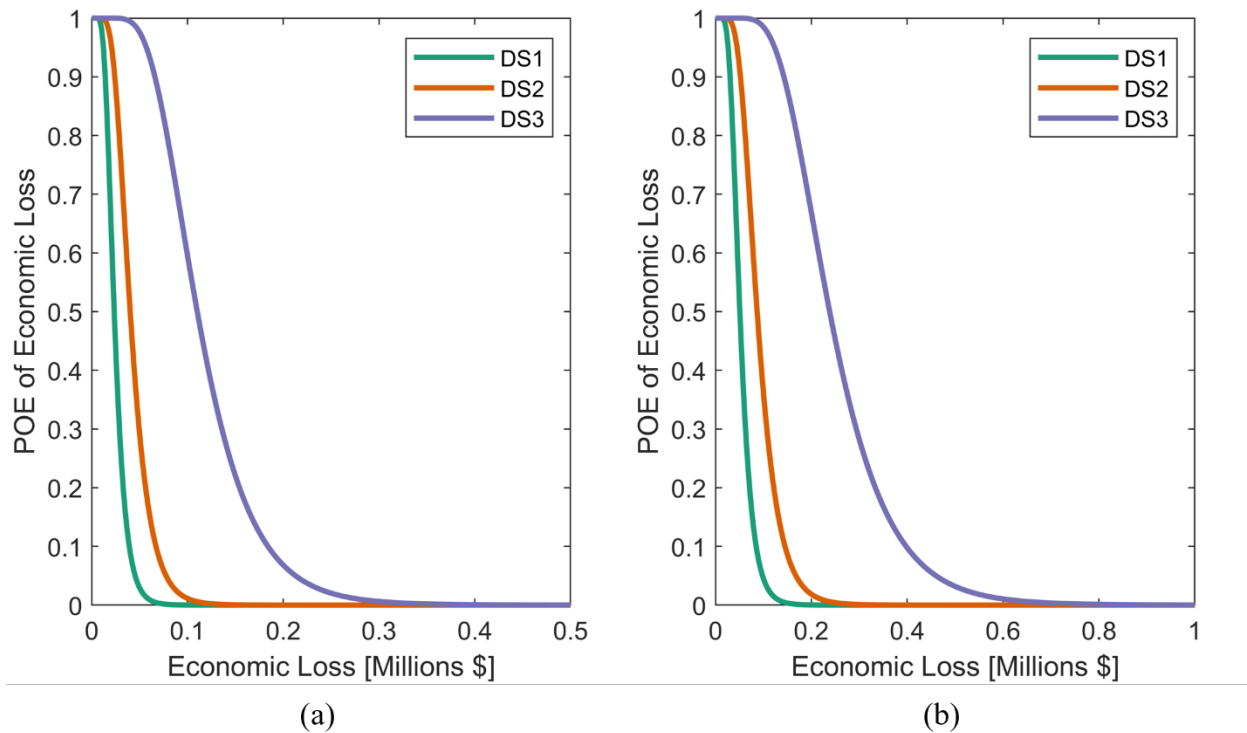


Figure 8.3 Loss functions for URM infill walls for (a) the 4-story ductile and (b) the 6-story non-ductile case study buildings. Adapted from Chiozzi and Miranda (2017).

8.1.4 Del Gaudio et al. (2019)

Del Gaudio et al. (2019) developed loss functions for each of the four damage estates they defined for URM infill walls. They described a set of repair actions and associated preliminary and supplementary activities needed to restore the infill walls to its pre-damage state. The repair actions they proposed, and the unit cost associated with each of these actions are presented in Table 8.4. Furthermore, they also provided a detailed explanation of the different areas, as a percentage of the panel area $A_w = H_w \times L_w$, affected by each repair action. Readers are referred to the original manuscript for a more detailed description of such values. Del Gaudio et al. (2019) did not give any estimates for the dispersion of these costs. Due to the lack of knowledge, a large value of dispersion ($\beta = 0.40$) is chosen. As before, a lognormal distribution is used to fit the loss data. Figure 8.3 presents loss functions for all four damage estates following the work by Del Gaudio et al. (2019).

Table 8.4 Unit cost of repair actions required to restore the infill to its pre-damage state. Modified from Del Gaudio et al. (2019).

Damage state	Repair action	Unit	Cost (\$/unit)
DS1 (Light cracking)	Preliminary operations: scaffolding installation	day	20.0
	Render or plaster removal up to 5 cm thick	m ²	3.5
	Finishing: coat rendering or plastering with cement mortar	m ²	15.2
	Paint	m ²	9.8
DS2 (Extensive cracking)	Preliminary operations: scaffolding installation	day	20.0
	Demolition activities	m ²	65.3
	Construction of infill panel	m ²	43.65
	Render or plaster removal up to 5 cm thick	m ²	3.5
	Finishing: coat rendering or plastering with cement mortar	m ²	15.2
	Paint	m ²	9.8
DS3 (Corner Crushing)	Preliminary operations: scaffolding installation	day	20.0
	Demolition activities	m ²	65.3
	Construction of infill panel	m ²	43.65
	Finishing: coat rendering or plastering with cement mortar	m ²	15.2
	Paint	m ²	9.8
	Supplementary activities	m ²	10.0
DS4 (Collapse)	Preliminary operations: scaffolding installation	day	20.0
	Demolition activities	m ²	65.3
	Construction of infill panel	m ²	43.65
	Finishing: coat rendering or plastering with cement mortar	m ²	15.2
	Paint	m ²	9.8
	Supplementary activities	m ²	10.0

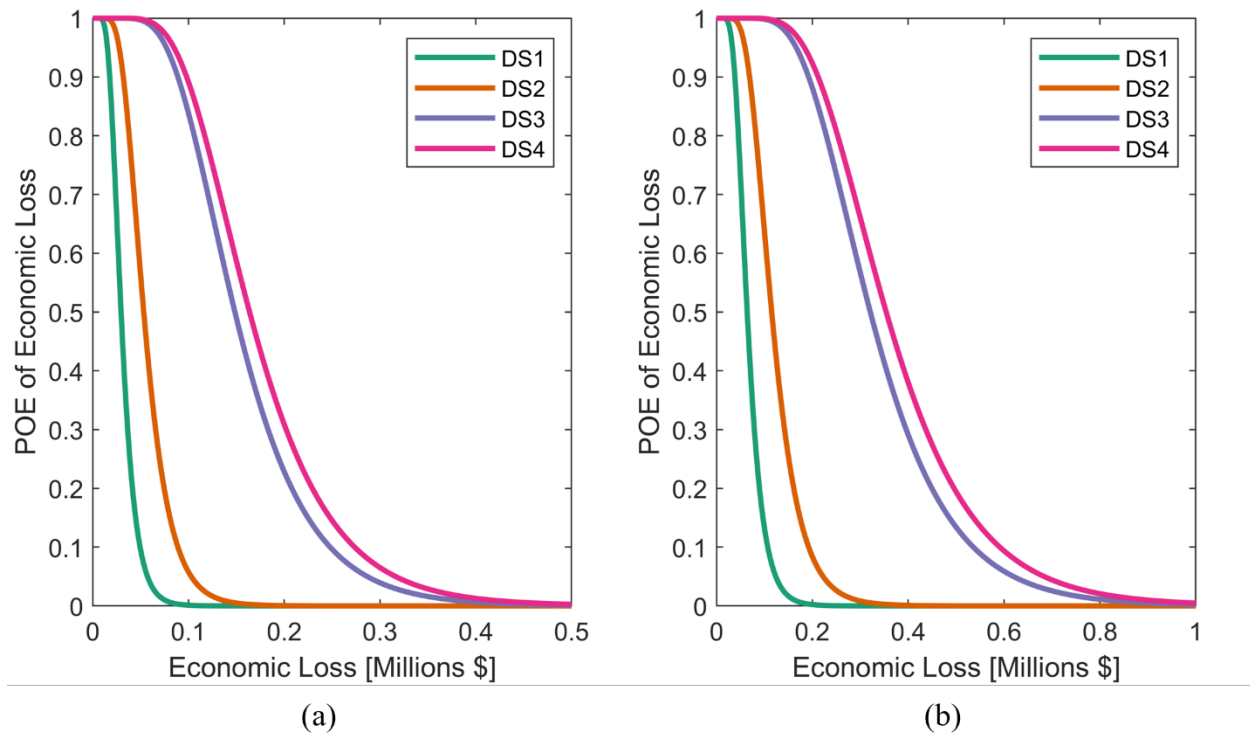


Figure 8.4 Loss functions for URM infill walls for (a) the 4-story ductile and (b) the 6-story non-ductile case study buildings. Adapted from Del Gaudio et al. (2019).

8.2 Loss function for collapse

Using the total probability theorem, the MAF of economic losses or the expected economic losses of a building can be calculated as the sum of two mutually exclusive and collectively exhaustive states that the building can be in after a given ground motion. After subjecting a building to a given ground motion time history, the building either reached collapse (C) or it did not (non-collapse NC) (Aslani 2005). Therefore a loss curve for the POE of economic losses in the case of global collapse is also developed. For this purpose, it is necessary to estimate the total cost of the building plus the complementary actions needed, such as demolition, cleaning, and technical costs, among others.

To estimate the total cost of constructing the two case-study buildings from scratch, information from a cost manual for Colombian named SISPAC (SISPAC 2022) is used. Its latest edition approximates direct costs per square meter (m^2) that can be found for different types of structures located in different cities through Colombia. For both of the case-study buildings, the estimates referring to buildings representative of a middle-class reinforced concrete building with heights varying from 5 to 8 stories tall were used. An estimate of 2.764.631 COP (Colombian pesos) per square meter is extracted for this structural typology. The above cost equals \$742.65 USD per square meter at today's exchange rate (1 USD = 3722.67 COP).

The ductile four-story building has a floor plan of 270 m^2 for each floor, thus the total construction area is 1080 m^2 . With the above estimate for cost/m^2 , the approximate total construction cost for the 4-story building comes out to be \$810,000 (3,000,000,000 COP). Similarly, the non-ductile six-story building has a floor plan of 432 m^2 for each floor, thus the total construction area is 2592 m^2 . With the above estimate for cost/m^2 , the approximate total construction cost for the 6-story building comes out to be \$1,950,000 (7,250,000,000 COP). These two values mentioned above can be interpreted and used as the mean expected total construction cost. Furthermore, for these curves, the same value of dispersion $\beta = 0.4$ is used due to the lack of knowledge. As with the previously defined loss functions, the loss functions for the case that the structure reached collapse are assumed to follow a lognormal distribution. Figure 8.5 shows the loss function for the collapse cases for the 4-story and 6-story case study buildings, respectively.

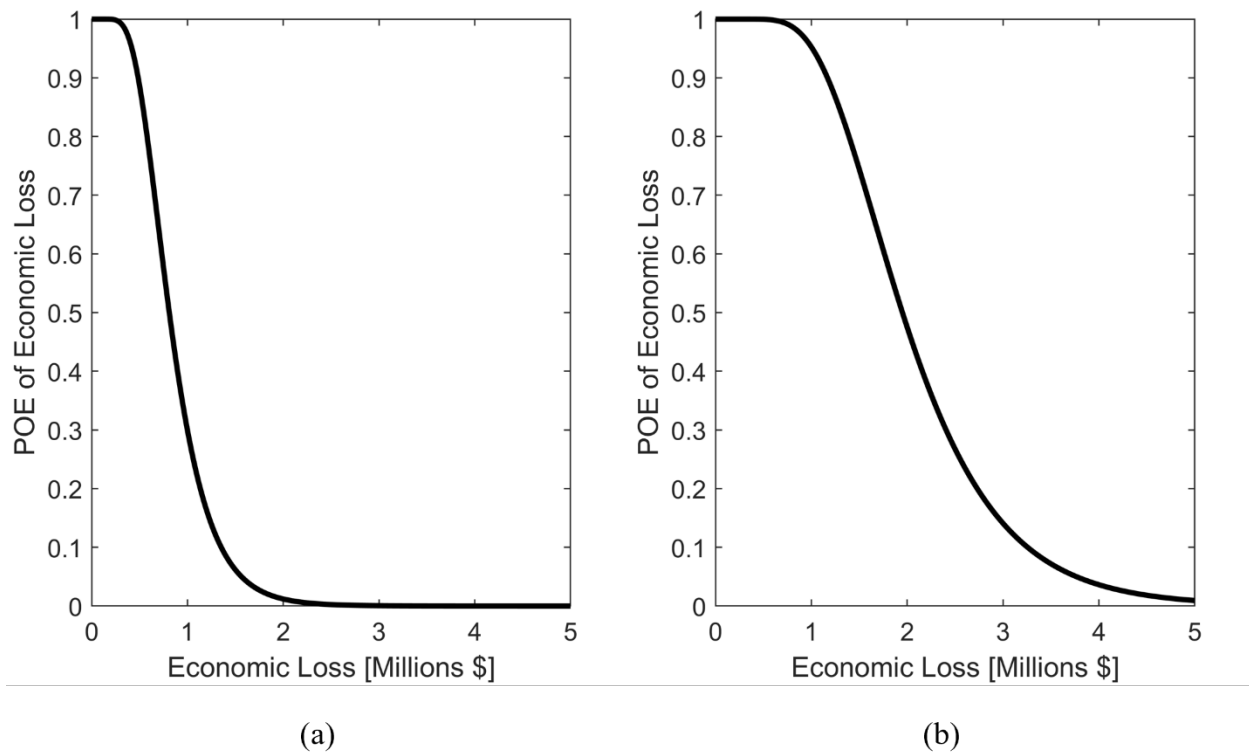


Figure 8.5 Loss functions for the total collapse case for the (a) 4-story and (b) 6-story case-study buildings.

8.3 Loss functions for other components.

Loss functions used in this section are obtained using information from the SISPAC cost manual (SISPAC 2022), which provides information on approximate percentages that a given component or group of components represents as a fraction of the construction total. The present section develops loss functions for the previously defined structural and non-structural components.

From the SISPAC cost manual, for a middle-class reinforced concrete building with heights varying from 5 to 8 stories tall, it can be extracted that the structural components of a building represent roughly 44% of the total construction cost. Thus the total cost of all structural components for the 4-story building and for the 6-story building are \$356,000 and \$858,000, respectively.

For the 4-story ductile building, keeping in mind that the median IDR values for DS_2 , and DS_3 (2.50% and 3.50% respectively) are well beyond the established IDR limit (2.0%) used to deem the structure to have reached collapse, the mean cost assigned for each DS corresponds to 16.66%, 60%, and 100% (for slight, moderate, and severe damage, respectively) of the total mean cost assigned to structural components for this building. For the 6-story non-ductile building, the mean cost assigned for each DS corresponds to 25%, 50%, 75%, and 100% (for light cracking, severe cracking, shear failure, and axial failure, respectively) of the total mean cost assigned to structural components for this building.

Similarly, from the SISPAC cost manual, for a middle-class reinforced concrete building with heights varying from 5 to 8 stories tall, it can be extracted that the non-structural components of a building represent roughly 50% of the total construction cost. In this category of non-structural components, cost associated with preliminary actions, flooring, roofing, hydraulic installation, mechanical installation, finishing activities, the content of the building, and others are being lumped together. To consider the cost associated with the content of the building, 40% of the total cost of construction is added to the median total cost of all non-structural components. Thus the total cost of all non-structural components for the 4-story building and for the 6-story building are \$729,000 and \$1,755,000, respectively. Furthermore, the mean cost assigned for each DS corresponds to 25%, 50%, 75%, and 100% (for slight, moderate, extensive, and complete damage, respectively) of the total mean cost assigned to non-structural components for each case study building.

Loss functions for the structural and non-structural components are assumed to have a lognormal distribution, following previous works of loss estimation of buildings (Aslani 2005). For the four sets of loss functions presented here, a large value of dispersion ($\beta = 0.40$) is chosen to represent the lack of certainty in the cost estimates and the variability associated with it. Finally, the loss functions for the structural and non-structural components for the 4-story ductile building are presented in Figures 8.6. Similarly, the loss functions for the structural and non-structural components for the 6-story non-ductile building are presented in Figure 8.7.

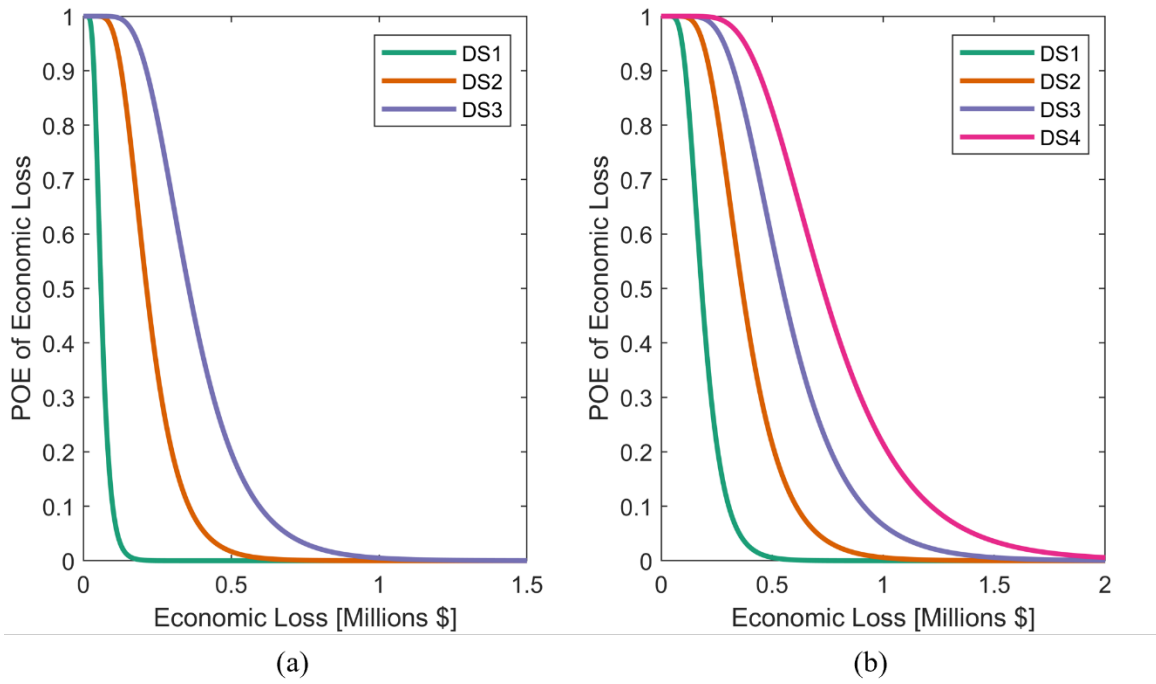


Figure 8.6 Loss functions for other components for the 4-story ductile building: (a) structural components and (b) non-structural components.

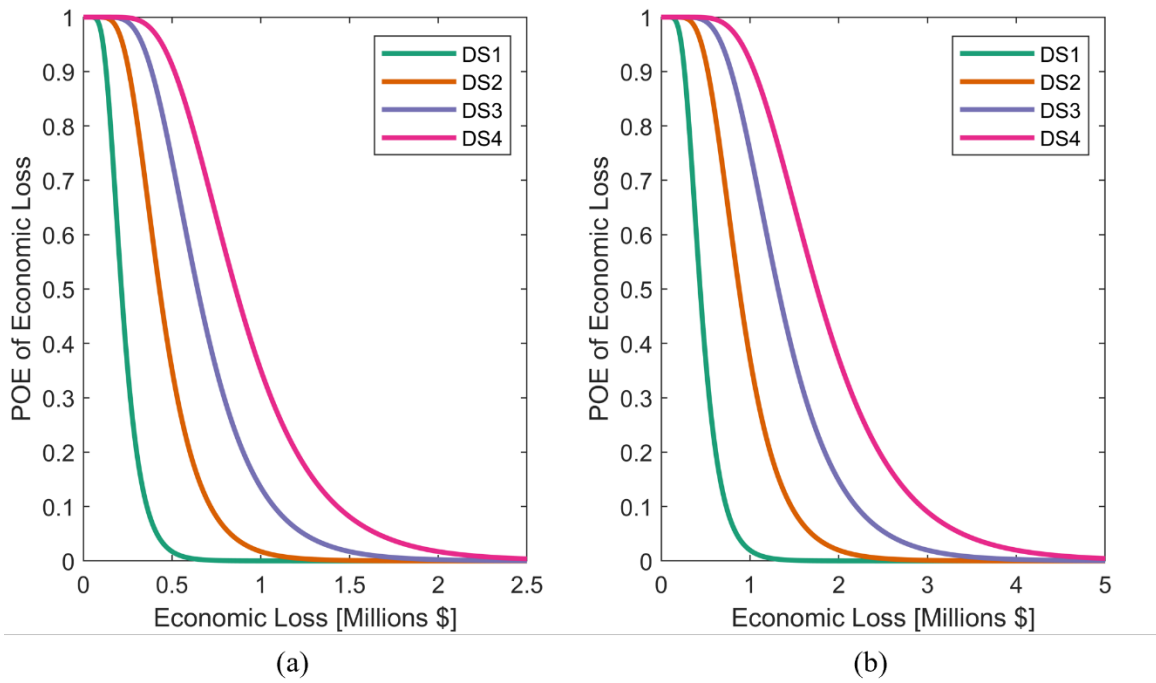


Figure 8.7 Loss functions for other components for the 6-story non-ductile building: (a) structural components and (b) non-structural components.

8.4 Loss estimation of the case study buildings

After all four phases of the PEER PBEE methodology have been performed, the final step is to combine them using the total probability theorem discussed in Equation (3.1) of section 3.3, herein repeated for convenience:

$$\lambda(DV) = \iiint G\langle DV|DM \rangle dG\langle DM|EDP \rangle dG\langle EDP|IM \rangle d\lambda(IM) \quad (3.1)$$

Several researchers have proposed discrete alternatives to Equation (3.1) (Aslani 2005, Günay and Mosalam 2013). In this section, the discrete approach to determine the loss curve of a structure proposed by Günay and Mosalam (2013) is followed and applied. In Equation (4a) of their manuscript, an equation to estimate the probability of exceedance (POE) of a given decision variable (DV) level (DV^n) can be found by:

$$P(DV^n) = \sum_m P(DV^n|IM_m) p(IM_m) \quad (4a)$$

where $p(IM_m)$ represents the probability of occurrence of a given intensity measure level and can be obtained as a function of a hazard curve assuming that the occurrence of earthquakes in a period of time t follows a Poisson model (Günay and Mosalam 2013); hazard curves for the site of interest were computed in Section 5.4.4. Furthermore, using the total probability theorem, $P(DV^n|IM_m)$, which is the POE of a certain DV level at a specific intensity measure (IM) can be computed as the sum of two mutually exclusive and collectively exhaustive states that the building can be in after a given ground motion, namely after a given ground motion a building either reached collapse (C) or it did not reach collapse (NC), as follows:

$$P(DV^n|IM_m) = P(DV^n|NC, IM_m) \cdot p(NC|IM_m) + P(DV^n|C) \cdot p(C|IM_m) \quad (4b)$$

where $p(C|IM_m)$ and $p(NC|IM_m)$ are the probability of collapse and non-collapse for a determined intensity measure level IM_m ; these two probabilities were computed in Section 6.1.5 and presented in Table 6.1 and Table 6.2, respectively, for the 4-story and the 6-story case study buildings. $P(DV^n|C)$ is the POE of a certain DV level conditioned on that collapse has occurred, for this purpose, a loss function for the collapse case was developed in Section 8.2. $P(DV^n|NC, IM_m)$ is the POE of a certain DV level given that collapse did not occur at a specific intensity level. $P(DV^n|NC, IM_m)$ can be computed as the sum of the losses in individual components for all possible damage states and intensity measures as follows:

$$P(DV^n|NC, IM_m) = \sum_j \sum_i \sum_k P(DV_j^n|DM_k) \cdot p(DM_k|EDP_j^i) \cdot p(EDP_j^i|IM_m) \quad (4c) - (4e)$$

where $P(DV_j^n | DM_k)$ is the POE of a DV level for a specific component of the structure given that damage state k is attained, $P(DV_j^n | DM_k)$ was developed in Sections 8.1 and 8.3 for the URM infill walls and other components and was presented by loss functions. Moreover, $p(DM_k | EDP_j^i)$ is the probability that a given component is at a specific damage state given an edp ; these probabilities were obtained in Sections 7.1 and 7.2 for all the components of the structure. Finally, $p(EDP_j^i | IM_m)$ represents the probability that an edp level occurs for a specific component j given that an intensity measure level IM_m occurred. These probabilities were developed during the structural analysis phase in Section 6.1.5 and were depicted in Figure 6.19 to Figure 6.26.

8.4.1 Loss estimation for the 4-story archetype building

Using the methodology described in Section 8.4, one could use the total probability theorem to compute the loss curve of our 4-story ductile archetype building. The contribution of each hazard level to the probability of exceedance of a given Economic Loss can be calculated with Equation (4b); the deaggregation of the POE of economic losses by hazard level is presented in Figure 8.8 for the four set of fragility and loss functions previously defined. Next, each contribution can be weighted according to the probability of occurrence of each of those hazard levels $p(IM_m)$ and added according to Equation (4a) to determine the POE of a given DV level. As it was already mentioned, if the occurrence of earthquakes is assumed to be a Poisson model, the POE of an intensity measure IM_m in T years can be computed as:

$$POE(IM_m) = 1 - e^{-\lambda(IM_m)T} \quad (8.1)$$

where $\lambda(IM_m)$ is the MAF of exceedance of a given IM (the hazard curve). The probability of occurrence $p(IM_m)$ can be found as suggested by Güney and Mosalam (2013) with Equation (8.2). Figure xx shows a graph of IM vs $p(IM_m)$.

$$p(IM_m) = POE(IM_m) - POE(IM_{m+1}) \quad (8.2)$$

The loss curves for the POE of Economic Losses obtained with the four definitions of fragility and loss functions are presented in Figure 8.9. Similarly, it is possible to compute the Mean Annual Frequency (MAF) of exceedance of different levels of Economic Losses by slightly modifying Equation (3.1) to Equation (8.3). The MAF of exceedance of different levels of Economic Losses for the 4-story ductile archetype building is presented in Figure 8.10.

$$\lambda(DV^n) = \sum_m P(DV^n | IM_m) \lambda(IM_m) \quad (8.3)$$

From Figure 8.9 and Figure 8.10, it can be concluded that there is not a drastic change in the outcome of the loss estimation for the 4-story case study building if different definitions of fragility and loss functions are considered, probably because the loss contributions of other

structural and nonstructural components are governing. Furthermore, from Figure 8.10, it can be seen that losses equivalent to 50% of the total construction cost (i.e., $0.5 \times \$810,000 = \$405,000$) occur on average every 590 years, which is not that far from the return period of the design-based earthquake (DBE). Additionally, on Figure 8.9 and Figure 8.10, the loss estimation curve for the bare frame case is presented. From these, it can be seen there is not a significant change in the loss curves of the bare frame vs the infilled frame. One possible explanation for this could be that because the infills are only a small portion of the building and the building and repairing cost are relatively low for infill walls, specially in a country like Colombia where labor cost is low.

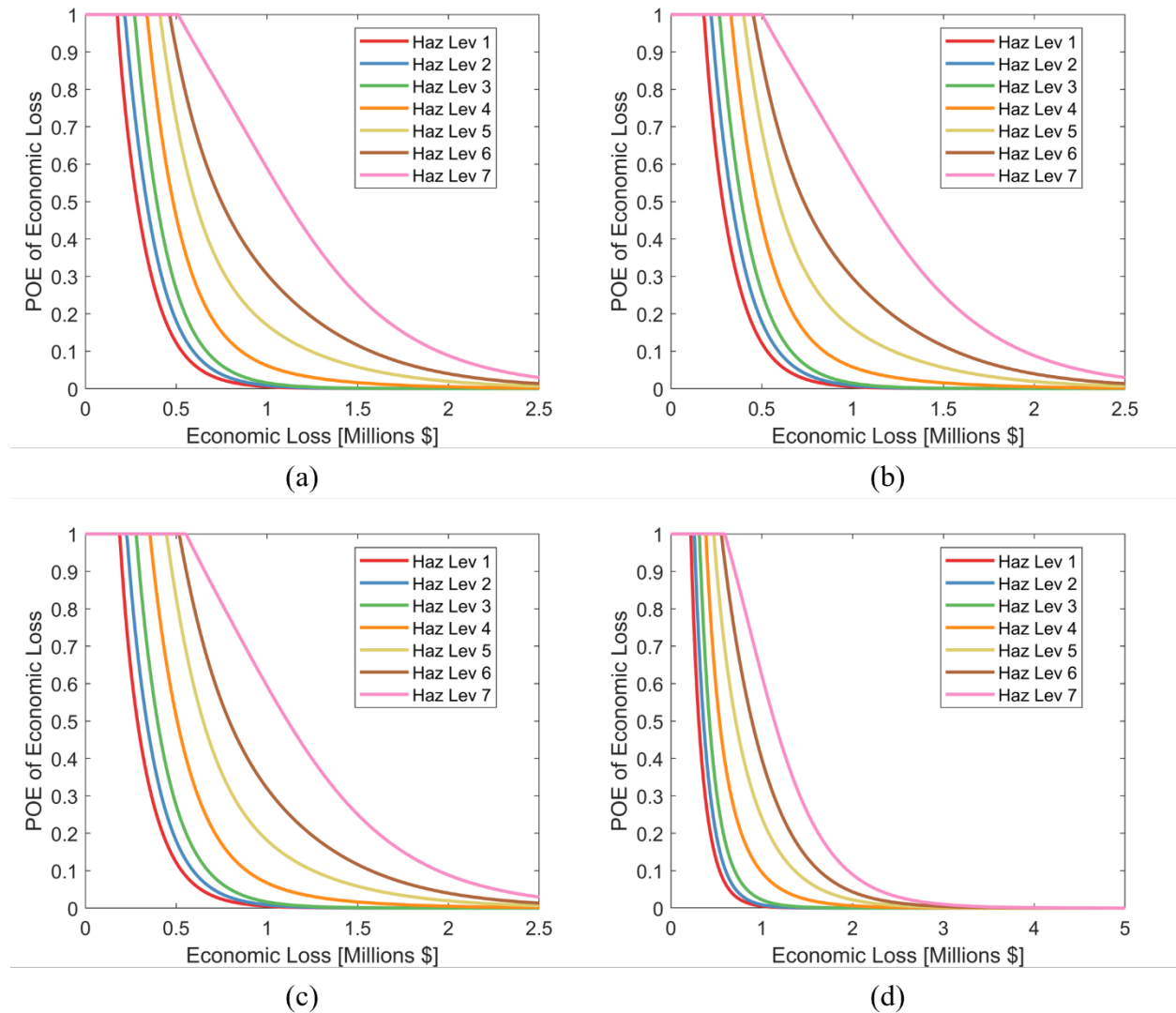


Figure 8.8 4-story ductile archetype building loss curves disaggregated by hazard level contribution obtained using fragility and loss functions by (a) Cardone and Perrone (2015), (b) Sassun et al. (2016), (c) Chiozzi and Miranda (2017), and (d) Del Gaudio et al. (2019).

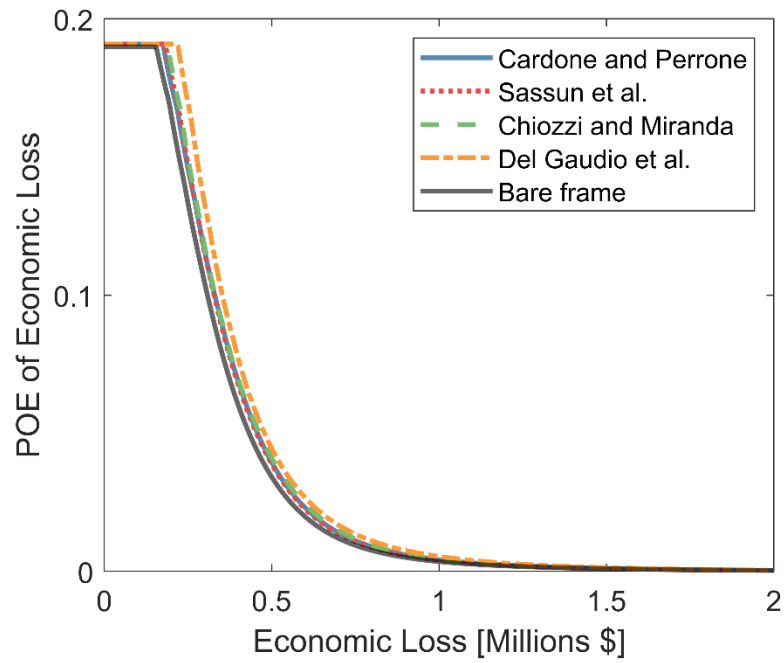


Figure 8.9 Loss curve for 4-story ductile archetype building.

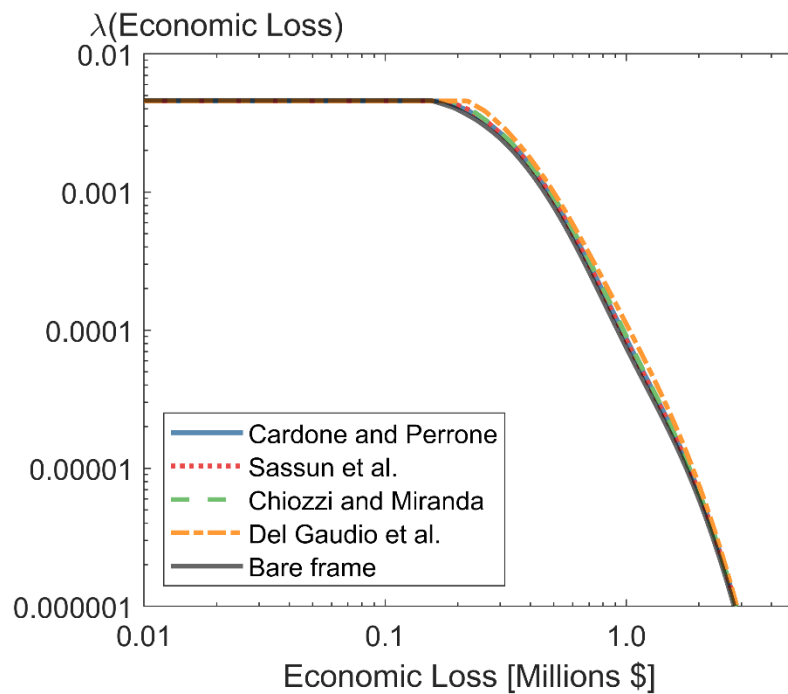


Figure 8.10 Mean Annual Frequency (MAF) of exceedance different levels of Economic Losses for the 4-story ductile archetype building.

8.4.2 Loss estimation for the 6-story archetype building

Similar results are presented for the 6-story non-ductile archetype building. The contribution of each hazard level to the probability of exceedance of a given Economic Loss is presented in Figure 8.11 for the four set of fragility and loss functions previously defined. And the loss curves for the POE of Economic Losses obtained with the four definitions of fragility and loss functions are presented in Figure 8.12. Similarly, the MAF of exceedance different levels of Economic Losses for the 6-story non-ductile archetype building is presented in Figure 8.13. In this case, again, Figure 8.10 it can be seen that there is not a large variability in the outcome of the loss estimation for the 6-story case study building if different definitions of fragility and loss functions are considered. Additionally, on Figure 8.12 and Figure 8.13, the loss estimation curve for the bare frame case is presented. From these, it can be seen that the loss estimation is less for the bare case. One possible difference, with the 4-story building, is that the infills used in the 6-story building had considerable less strength than the ones used in the 4-story building, which would make this set of infills more likely to fail and experience a certain damage state more quickly.

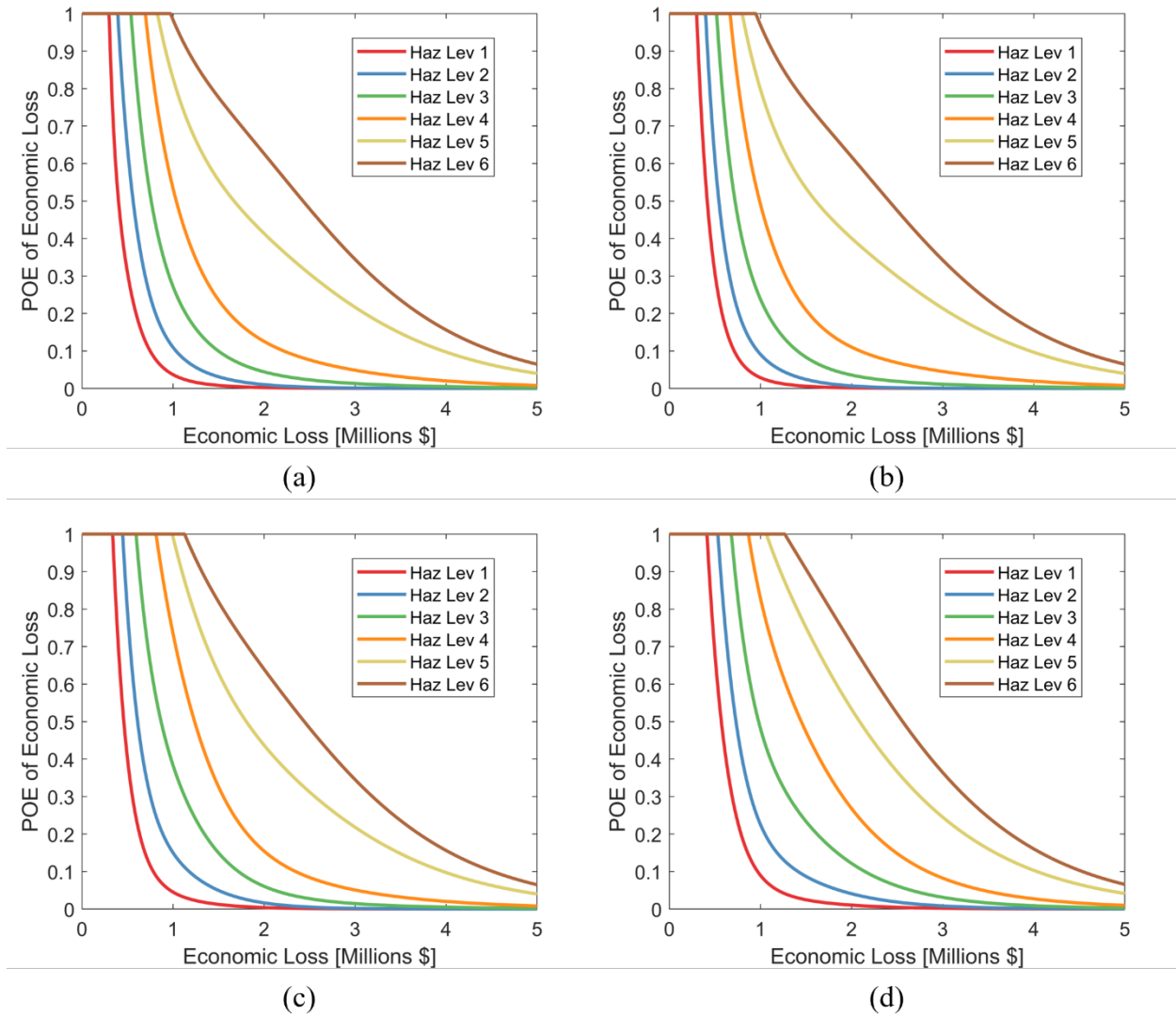


Figure 8.11 6-story non-ductile archetype building loss curves disaggregated by hazard level contribution obtained using fragility and loss functions by (a) Cardone and Perrone (2015), (b) Sassun et al. (2016), (c) Chiozzi and Miranda (2017), and (d) Del Gaudio et al. (2019).

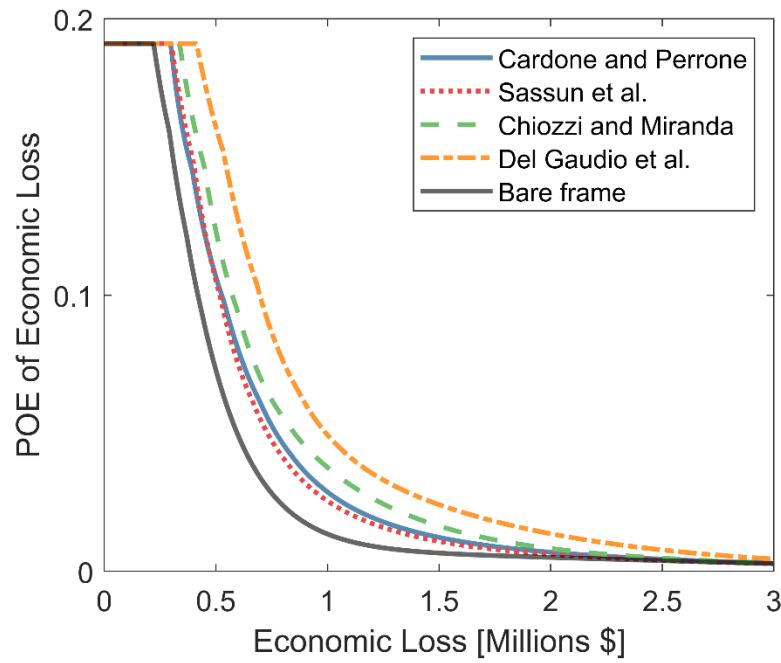


Figure 8.12 Loss curve for 6-story non-ductile archetype building.

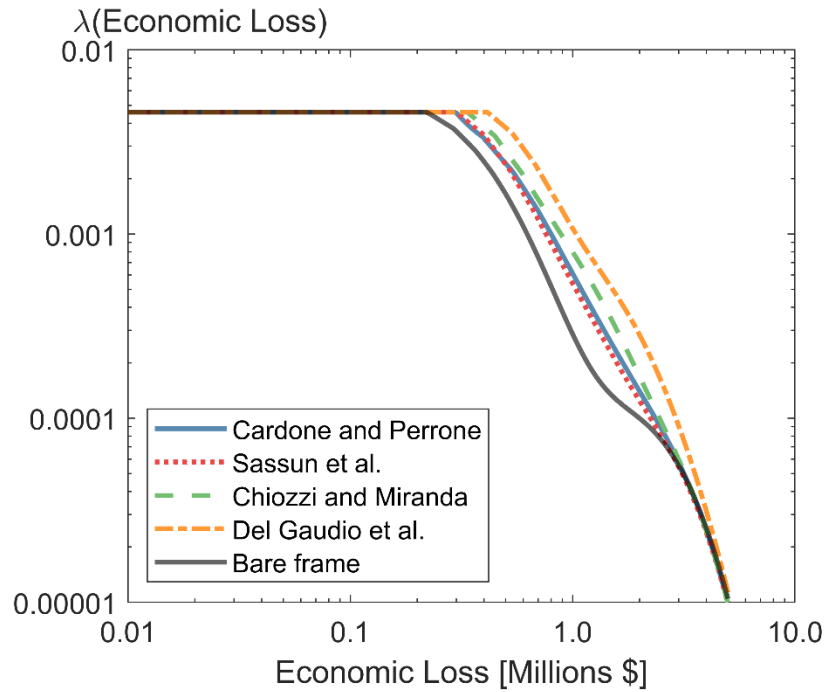


Figure 8.13 Mean Annual Frequency (MAF) of exceedance different levels of Economic Losses for the 6-story non-ductile archetype building.

Finally, a comparison between the loss estimation of the two case study buildings is presented in Figure 8.14 and Figure 8.15 for the infilled and bare cases, respectively. It can be seen

that for both cases, infilled and bare frames, the probability of exceeding a given threshold of economic loss is much smaller for the 4-story ductile case study, as expected, a ductile building performs better under seismic loads when compared to a non-ductile building, which translates into less economic losses.

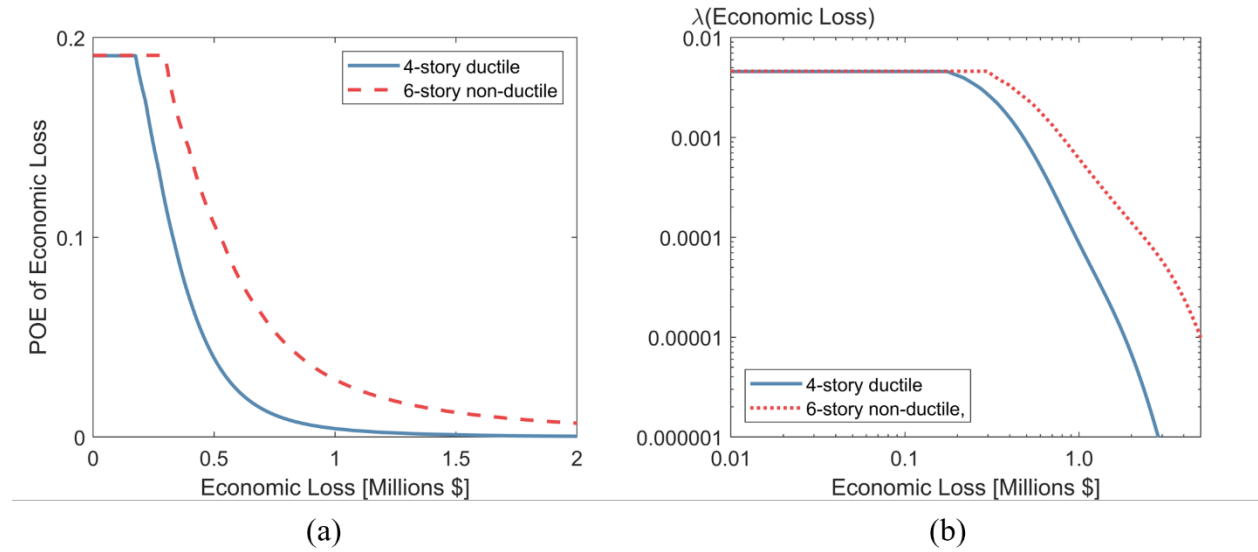


Figure 8.14 Comparison of the (a) loss curve and (b) MAF of exceedance of different economic losses for the infilled case of the 4-story ductile building and the 6-story non-ductile building.

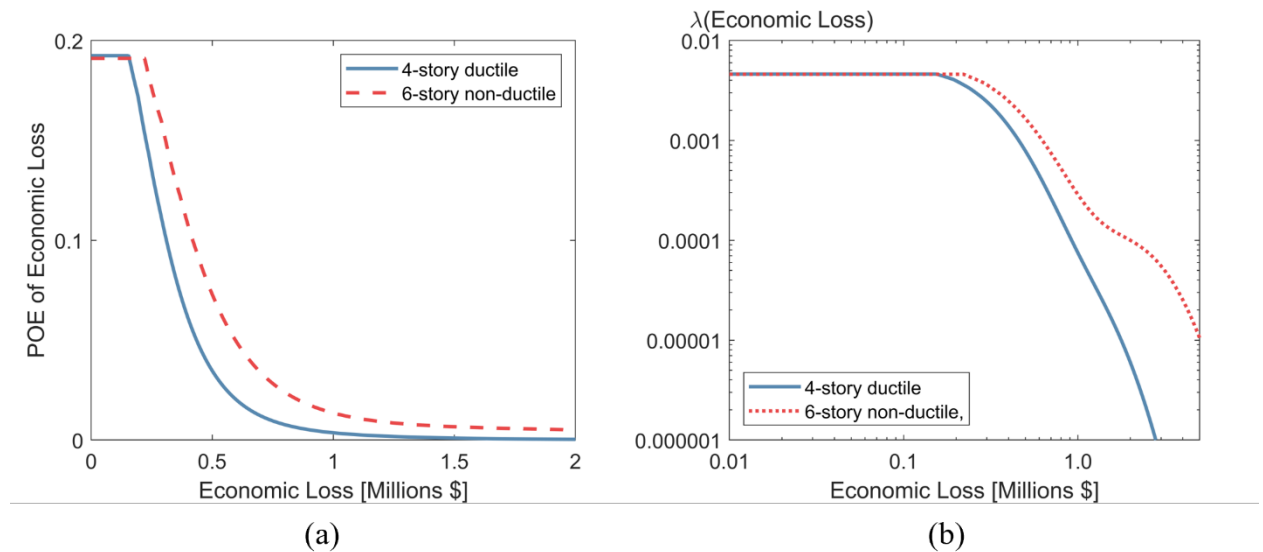


Figure 8.15 Comparison of the (a) loss curve and (b) MAF of exceedance of different economic losses for the bare frame case of the 4-story ductile building and the 6-story non-ductile building.

9. Conclusions and Future Research

9.1 Summary

Assessing the structural behavior of RC frames with URM infill walls is a very complex problem that depends on 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, and others. To further motivate the need to improve our understanding of this structural typology, Chapter 1 reviewed three seismic events, namely the 1999 Armenia (Colombia) earthquake, the 2016 Muisne (Ecuador) earthquake, and the 2017 Puebla (Mexico) earthquake that exposed the vulnerability of RC frame buildings with URM infills when subjected to seismic forces. RC frames with URM infill walls are widely used in earthquake resistant construction in seismically active regions around the world. These three seismic events and many others worldwide served as a motivation for this study.

A thorough literature review was done in Chapter 2 to determine the state of the art of numerical modeling of URM walls, its applications, structural performance, and interaction with the surrounding frame; an overview of the PEER PBEE methodology was also presented in Chapter 3. This exhaustive review showed evidence of areas that need improvement within the adoption of the PBEE methodology for RC frame buildings with URMs.

Overall, an application of the PEER PBEE methodology, on RC frames with URM infill walls was used to evaluate the performance of this type of structure for several reasons. First, it quantifies the performance of this type of building in earthquakes with varying intensities using a consistent set of decision variables. Second, this quantification allows highlighting particular deficiencies that need to be addressed to improve the performance of these buildings and identifying these deficiencies in the order of importance. Finally, existing buildings of this type may need a retrofit, and new design methodologies, including protective systems, need to be developed to design new buildings. Probabilistic PBEE methodologies can guide these designs to minimize losses. Therefore, Chapters 4 to 8 of the dissertation focused on adopting the PEER PBEE methodology for evaluating RC frames with infill walls.

To study RC frames with infill walls using PBEE, two building archetypes of RC frames with URM infill walls were designed in Chapter 4, one according to the latest Colombian seismic provisions and another one intended to represent a building designed and constructed around 1985, one year after the first seismic building code was implemented in Colombia. Chapters 5 to 8 used these two archetypes as case studies of a hazard-consistent methodology to evaluate economic losses and performance through the PEER-PBEE methodology. The design of the buildings followed typical Colombian engineering practices. The three seismic events studied in Chapter 1 inspired the archetype buildings selected in Chapter 4.

Chapter 5 describes the seismic hazard for Armenia, Colombia, where the archetype buildings, previously designed in Chapter 4, are located. This chapter presented an overview of

the tectonic setting of the area, as well as a review of the main earthquakes that have struck the region. A simplified seismic hazard model is proposed and implemented. The hazard model is used to perform a probabilistic seismic hazard analysis (PSHA), and the results of the simplified model are validated against the latest available seismic hazard for Colombia. Furthermore, hazard curves, Uniform Hazard Spectra (UHS), and deaggregation of the hazard were computed and presented. Then, a Conditional Scenario Spectra (CSS) was used as a tool to select a set of realistic ground-motion records with rates of occurrence that reproduce the hazard at the site of interest. The chapter ended with a proposed way to estimate *edp-risk* curves for a given structure and sites of interest using the results obtained from the CSS. A novel contribution of this Chapter was the extension of the CSS methodology to account for different tectonic settings in the computation of fragility curves and seismic risk. This would allow presenting the seismic risk assessment in terms of the annual rate of exceedance of specific EDPs and deaggregated by tectonic setting.

Chapter 6 studied the behavior of the two archetype buildings under realistic suites of ground motions that represent the hazard at the site and was assessed by means of Nonlinear Time History (NLTH) analyses; the numerical models were developed using the software package OpenSees. Detailed information about the nonlinear model implemented in OpenSees was outlined. Specifically, two key modeling aspects were considered in the numerical models of URM infill walls: 1) In-Plane/Out-of-Plane (IP/OOP) interaction of infill walls, 2) account of infills failure through the implementation of a progressive collapse algorithm. The performance of the proposed model for URM was studied. For both archetype buildings, nonlinear static analyses (pushover) were performed in both horizontal directions. Moreover, NLTH analyses were performed, and relevant statistics are summarized; specifically, the probability of different engineering demand parameters (*edp*), and probability of collapse and non-collapse for a given intensity measure were computed. Chapter 6 also obtained fragility functions, which are plots of $P\{EDP > edp | IM = im\}$ versus *IM*, for several *edps*. Finally, *edp-risk* was computed for both case study buildings in a non-traditional way, using the results obtained from the CSS.

Chapter 7 reviewed the literature for previously developed fragility functions that estimate levels of damage in unreinforced masonry infill (URM) infill walls as a function of in-plane (IP). Four sets of fragility-component functions for URM infill walls are presented. The idea was to compare how the use of one set of fragility or another impacts the loss estimation of the case study buildings, which to the best of the knowledge of the author, had not been done until now. Chapter 7 also described the fragility function for all other components of the two case study buildings. All the different damage states used in the fragility functions presented in Chapter 7 were chosen based on specific repairs needed to restore the component to its pre-damage condition, which helped streamline the loss estimation process in the next chapter.

Chapter 8 developed loss functions for the four different sets of fragility functions of URM infill walls, previously developed in Chapter 7. These different sets of loss functions were used to estimate the performance of the two case study buildings in terms of the decision variable of interest and to quantify the variability in the final loss estimation due to the epistemic uncertainty of the masonry infill fragility and loss functions. Moreover, Chapter 8 described the loss functions used for all other components of our case-study building. A loss function for the case of the complete collapse was also generated.

Finally, at the end of Chapter 8, after all four phases of the PEER PBEE methodology had been completed, the hazard, structural, damage, and loss results were combined using the total probability theorem to estimate the performance of the two case study buildings in terms of economic losses. One key finding was that, at least for these two buildings, there does not seem to be considerable variability in the loss estimation due to the uncertainty in the damage and loss functions for the URM infill wall. One reason to explain this is that for these buildings, which only had URM infill walls in the perimeter RC frame, there are not enough infills to create a significant impact in the loss estimation of the whole building and the losses due to structural and nonstructural components are larger. From Chapter 8, it was also possible to measure the impact of including or not including the infills in the loss estimation process, for the 4-story ductile building which had high strength infill walls, there was not a significant change in the loss curves of the bare frame case compared to the infilled case. On the other hand, for the 6-story non-ductile building which had low strength infill walls, the inclusion of the infills in the loss estimation process did increase the expected losses when compared to the bare frame case. Additionally, it was noted that for both cases, infilled and bare frames, the probability of exceeding a given threshold of economic loss is much smaller for the 4-story ductile case study compared to the 6-story building, as expected, a ductile building performs better under seismic loads when compared to a non-ductile building, which translates into less economic losses.

9.2 Future Research and Areas of Improvement

As with any research endeavor, there can always be improvements. Some future work is suggested:

1. Regarding the numerical model used in this dissertation to represent the URM infill panels and the element removal algorithm, an improvement to the infill panels' IP/OOP interaction curve can be sought. Perhaps, experimental work available in the literature with enough information can be explored to estimate better the failure curve used in section 6.1.2.
2. One question that remains to be answered regarding the behavior of RC frames with infill walls is: does the inclusion of the OOP actions and their interaction with the IP actions change the overall behavior and possible failure modes of the structure? If the answer is yes, an attempt to include such modeling in the engineering practice should be made. This is already present in ASCE-41 (2017) for evaluation and retrofit of buildings, which can potentially be expanded to building codes for new designs.
3. Regarding the methodology's damage analysis stage, reliable fragility functions that allow the estimation of levels of damage in unreinforced masonry infill (URM) infill walls, as a function of in-plane (IP) and out-of-plane (OOP) actions need to be developed. These fragility functions could potentially be derived from experimental data available in the literature.
4. Future work of this study will focus on using downtime as decision variables since downtime and economic losses are key to extending the concept of this

methodology to city-scale problems in the frame of achieving a resilient society. For that, loss functions in terms of downtime need to be developed for URM infill walls and all other structure components.

5. A larger pool of archetype buildings needs to be explored to explore the impact on the epistemic and aleatory uncertainty present in URM infill walls' damage and loss functions. It would be interesting to choose buildings where the URM infill walls make up a more significant percentage of the floor plan and, therefore, would represent a larger percentage of the cost of the building.

Ultimately, the author hopes that the application presented in this dissertation of the PEER PBEE methodology to RC frames with URM infill walls can advance the current practice of how this type of and other structures are assessed.

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Appendix A. Conditional Scenario Spectra (CSS) Metadata

Metadata for the Conditional Scenario Spectra performed and described in **Section 5.4.5**, the data is presented by source mechanism (i.e., Interface, Intralab, and Crustal), in Table A.1 to Table A.3.

Table A.1 Metadata of CSS for interface earthquakes.

Hazard Level	ID	RSN	EqID	Mag (Mw)	Rrup (km)	Vs30 (m/s)	Scale Factor	Rate
1	1	6001800	6000149	8.8	130.7	574	0.495	1.14E-03
	2	4032588	4000095	8.3	159.1	254	1.276	1.12E-03
	3	4028592	4000068	8.3	47.6	600.3	1.468	8.95E-04
	4	6001804	6000149	8.8	116.9	622	0.850	5.11E-04
	5	4040369	4000001	9.1	52.6	398.5	1.065	1.19E-03
	6	4000348	4000001	9.1	44.5	200.7	1.401	1.47E-03
	7	4000836	4000001	9.1	165.6	277.4	2.370	1.43E-03
2	8	6001800	6000149	8.8	130.7	574	0.642	1.04E-04
	9	4032588	4000095	8.3	159.1	254	1.653	6.46E-04
	10	4028592	4000068	8.3	47.6	600.3	1.898	1.70E-04
	11	6001804	6000149	8.8	116.9	622	1.096	1.37E-04
	12	4040369	4000001	9.1	52.6	398.5	1.371	3.88E-04
	13	4000348	4000001	9.1	44.5	200.7	1.796	1.07E-03
	14	4000224	4000001	9.1	80.1	364.2	0.767	1.65E-04
3	15	4000836	4000001	9.1	165.6	277.4	3.018	8.56E-04
	16	4032588	4000095	8.3	159.1	254	2.089	2.84E-04
	17	4028592	4000068	8.3	47.6	600.3	2.395	6.75E-05
	18	6001804	6000149	8.8	116.9	622	1.381	1.42E-04
	19	4040369	4000001	9.1	52.6	398.5	1.724	9.79E-05
	20	4000348	4000001	9.1	44.5	200.7	2.250	4.27E-04
	21	4000224	4000001	9.1	80.1	364.2	0.960	4.61E-05
4	22	4022913	4000068	8.3	60.5	230.2	0.794	3.61E-06
	23	4000836	4000001	9.1	165.6	277.4	3.763	2.76E-04
	24	6001800	6000149	8.8	130.7	574	1.009	3.84E-05
	25	4032588	4000095	8.3	159.1	254	2.590	1.12E-04
	26	4028592	4000068	8.3	47.6	600.3	2.965	4.85E-06
	27	6001804	6000149	8.8	116.9	622	1.707	5.51E-05
	28	4040369	4000001	9.1	52.6	398.5	2.128	2.71E-05
5	29	4000348	4000001	9.1	44.5	200.7	2.771	1.75E-04
	30	4000224	4000001	9.1	80.1	364.2	1.180	6.43E-06
	31	4000836	4000001	9.1	165.6	277.4	4.613	1.25E-04
	32	4032588	4000095	8.3	159.1	254	3.160	4.18E-05

	33	4028592	4000068	8.3	47.6	600.3	3.614	8.35E-06
	34	6001804	6000149	8.8	116.9	622	2.079	3.50E-06
	35	4040369	4000001	9.1	52.6	398.5	2.589	1.98E-05
	36	4000348	4000001	9.1	44.5	200.7	3.362	9.34E-05
	37	4000224	4000001	9.1	80.1	364.2	1.430	5.01E-06
	38	4000836	4000001	9.1	165.6	277.4	5.579	6.48E-05
6	39	4032588	4000095	8.3	159.1	254	3.807	1.71E-05
	40	4028592	4000068	8.3	47.6	600.3	4.349	7.61E-06
	41	6001804	6000149	8.8	116.9	622	2.500	8.53E-07
	42	4040369	4000001	9.1	52.6	398.5	3.109	3.55E-06
	43	4000348	4000001	9.1	44.5	200.7	4.031	3.63E-05
	44	4000224	4000001	9.1	80.1	364.2	1.713	4.47E-06
	45	4022913	4000068	8.3	60.5	230.2	1.412	2.12E-07
	46	4000836	4000001	9.1	165.6	277.4	6.670	3.18E-05
7	47	4032588	4000095	8.3	159.1	254	4.536	4.49E-06
	48	4028592	4000068	8.3	47.6	600.3	5.178	7.57E-06
	49	4040369	4000001	9.1	52.6	398.5	3.696	9.32E-07
	50	4000348	4000001	9.1	44.5	200.7	4.784	1.13E-05
	51	4022913	4000068	8.3	60.5	230.2	1.673	1.13E-06
	52	4000836	4000001	9.1	165.6	277.4	7.896	6.69E-06
8	53	6001800	6000149	8.8	130.7	574	2.094	8.87E-06
	54	4032588	4000095	8.3	159.1	254	5.354	8.56E-06
	55	4028592	4000068	8.3	47.6	600.3	6.108	5.74E-06
	56	6001804	6000149	8.8	116.9	622	3.505	3.94E-06
	57	4040369	4000001	9.1	52.6	398.5	4.354	4.60E-06
	58	4000348	4000001	9.1	44.5	200.7	5.627	1.19E-05
	59	4000836	4000001	9.1	165.6	277.4	9.270	1.86E-06

Table A.2 Metadata of CSS for intraslab earthquakes.

Hazard Level	ID	RSN	EqID	Mag (Mw)	Rrup (km)	Vs30 (m/s)	Scale Factor	Rate
1	1	1002829	1000142	7.2	139.6	455	0.617	2.98E-04
	2	7006610	7000049	6.9	268	523	10.685	3.72E-05
	3	5001503	5000150	6.7	100.8	1000	6.104	7.48E-04
	4	4032460	4000092	7.6	107.3	260.2	0.174	8.47E-04
	5	3001493	3000184	7.2	89.9	517	1.712	1.46E-03
	6	7006355	7000049	6.9	107.2	344.3	2.746	9.01E-04
	7	6001150	6000061	7.8	136.4	605	0.820	5.72E-04
	8	2000893	2000014	6.6	41.3	362	0.509	1.32E-03
	9	7005081	7000045	6.6	145.6	409.8	5.736	1.30E-03
2	10	1002829	1000142	7.2	139.6	455	0.956	1.91E-04

	11	7006610	7000049	6.9	268	523	16.516	1.67E-05
	12	5001503	5000150	6.7	100.8	1000	9.335	1.64E-04
	13	4032460	4000092	7.6	107.3	260.2	0.264	3.33E-04
	14	3001493	3000184	7.2	89.9	517	2.590	2.71E-04
	15	7006355	7000049	6.9	107.2	344.3	4.144	3.37E-04
	16	6001150	6000061	7.8	136.4	605	1.234	2.16E-04
	17	2000893	2000014	6.6	41.3	362	0.765	1.92E-04
	18	6000991	6000167	7.1	264	489	9.410	3.84E-04
	19	7005060	7000045	6.6	245.9	223.7	7.433	2.54E-04
	20	7005081	7000045	6.6	145.6	409.8	8.428	7.30E-04
3	21	1002829	1000142	7.2	139.6	455	1.395	1.69E-04
	22	7006610	7000049	6.9	268	523	24.055	2.18E-06
	23	2000902	2000014	6.6	39.1	427	2.690	3.06E-05
	24	5001503	5000150	6.7	100.8	1000	13.486	7.29E-05
	25	2000030	2000004	6.8	64.6	399	1.162	6.69E-07
	26	4032460	4000092	7.6	107.3	260.2	0.381	7.71E-05
	27	3000103	3000038	6.5	93.1	586	1.860	2.84E-05
	28	3001493	3000184	7.2	89.9	517	3.711	9.94E-05
	29	7006355	7000049	6.9	107.2	344.3	5.926	9.44E-05
	30	6001150	6000061	7.8	136.4	605	1.760	1.25E-04
	31	2000893	2000014	6.6	41.3	362	1.089	3.35E-06
	32	6001143	6000061	7.8	206.4	432	1.360	1.59E-05
	33	7006078	7000048	7	40.7	493.5	0.859	7.26E-07
	34	6000991	6000167	7.1	264	489	13.320	1.96E-04
	35	7005081	7000045	6.6	145.6	409.8	11.809	3.27E-04
	36	1002957	1000142	7.2	253.9	332	1.304	1.31E-06
4	37	1002829	1000142	7.2	139.6	455	1.944	3.73E-05
	38	7006610	7000049	6.9	268	523	33.477	4.32E-07
	39	5001503	5000150	6.7	100.8	1000	18.649	1.92E-05
	40	4032460	4000092	7.6	107.3	260.2	0.525	1.27E-05
	41	3000103	3000038	6.5	93.1	586	2.560	1.77E-05
	42	3001493	3000184	7.2	89.9	517	5.100	4.68E-05
	43	7006355	7000049	6.9	107.2	344.3	8.130	3.90E-05
	44	6001150	6000061	7.8	136.4	605	2.411	5.17E-05
	45	2000893	2000014	6.6	41.3	362	1.490	5.76E-05
	46	7006078	7000048	7	40.7	493.5	1.171	9.15E-07
	47	6000991	6000167	7.1	264	489	18.130	5.77E-05
	48	7005081	7000045	6.6	145.6	409.8	15.947	1.31E-04
	49	1002957	1000142	7.2	253.9	332	1.759	3.23E-05
5	50	1002829	1000142	7.2	139.6	455	2.615	1.96E-05
	51	7006610	7000049	6.9	268	523	44.967	4.41E-06
	52	5001503	5000150	6.7	100.8	1000	24.925	1.35E-05
	53	4032460	4000092	7.6	107.3	260.2	0.699	8.00E-06

	54	3000103	3000038	6.5	93.1	586	3.409	6.92E-08
	55	3001493	3000184	7.2	89.9	517	6.782	6.39E-06
	56	7006355	7000049	6.9	107.2	344.3	10.798	1.61E-05
	57	6001150	6000061	7.8	136.4	605	3.199	2.01E-05
	58	2000893	2000014	6.6	41.3	362	1.974	4.50E-05
	59	6000991	6000167	7.1	264	489	23.931	2.05E-05
	60	7005081	7000045	6.6	145.6	409.8	20.919	7.02E-05
	61	1002957	1000142	7.2	253.9	332	2.304	1.84E-06
6	62	1002829	1000142	7.2	139.6	455	3.419	8.32E-06
	63	7006610	7000049	6.9	268	523	58.736	1.82E-06
	64	5001503	5000150	6.7	100.8	1000	32.427	6.08E-06
	65	2000030	2000004	6.8	64.6	399	2.784	1.12E-08
	66	4032460	4000092	7.6	107.3	260.2	0.908	2.30E-08
	67	3001493	3000184	7.2	89.9	517	8.788	3.03E-06
	68	7006355	7000049	6.9	107.2	344.3	13.978	5.11E-06
	69	6001150	6000061	7.8	136.4	605	4.137	1.09E-05
	70	2000893	2000014	6.6	41.3	362	2.550	9.67E-06
	71	6000991	6000167	7.1	264	489	30.823	8.55E-06
	72	7005060	7000045	6.6	245.9	223.7	24.205	2.45E-06
	73	7005081	7000045	6.6	145.6	409.8	26.808	2.73E-05
	74	1002957	1000142	7.2	253.9	332	2.950	4.66E-06
7	75	1002829	1000142	7.2	139.6	455	4.370	2.54E-06
	76	7006610	7000049	6.9	268	523	75.013	4.86E-06
	77	5001503	5000150	6.7	100.8	1000	41.278	1.95E-06
	78	4032460	4000092	7.6	107.3	260.2	1.154	1.79E-07
	79	3000103	3000038	6.5	93.1	586	5.615	9.32E-06
	80	7006355	7000049	6.9	107.2	344.3	17.721	3.12E-07
	81	2000893	2000014	6.6	41.3	362	3.227	2.89E-06
	82	6001143	6000061	7.8	206.4	432	4.011	4.38E-06
	83	7006078	7000048	7	40.7	493.5	2.522	1.16E-06
	84	6000991	6000167	7.1	264	489	38.916	2.16E-06
	85	7005081	7000045	6.6	145.6	409.8	33.709	9.26E-06
	86	1002957	1000142	7.2	253.9	332	3.706	5.79E-08
8	87	1002829	1000142	7.2	139.6	455	5.483	8.60E-06
	88	7006610	7000049	6.9	268	523	94.052	3.08E-06
	89	2000902	2000014	6.6	39.1	427	10.351	1.05E-07
	90	5001503	5000150	6.7	100.8	1000	51.614	7.09E-06
	91	4032460	4000092	7.6	107.3	260.2	1.441	3.24E-06
	92	7006355	7000049	6.9	107.2	344.3	22.083	4.73E-06
	93	6001150	6000061	7.8	136.4	605	6.526	2.23E-06
	94	6001143	6000061	7.8	206.4	432	4.988	1.09E-08
	95	6000991	6000167	7.1	264	489	48.331	1.47E-06
	96	7005081	7000045	6.6	145.6	409.8	41.722	4.58E-06

97	1002957	1000142	7.2	253.9	332	4.584	4.00E-06
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Table A.3 Metadata of CSS for Crustal earthquakes.

Hazard Level	ID	RSN	EqID	Mag (Mw)	Rrup (km)	Vs30 (m/s)	Scale Factor	Rate
1	1	1614	138	7.1	11.5	481	1.089	2.15E-04
	2	1681	149	5.9	15.4	405.2	1.025	9.98E-04
	3	1091	127	6.7	23.6	996.4	0.762	1.10E-04
	4	530	101	6.1	10.8	207.5	0.776	8.17E-05
	5	1512	137	7.6	8.2	443	0.319	2.54E-04
	6	668	113	6	20.4	270.2	0.785	3.23E-04
	7	317	73	5.9	7.8	191.1	0.820	1.18E-03
	8	458	90	6.2	11.5	221.8	0.592	1.24E-04
	9	2385	171	5.9	21.1	624.9	2.200	9.15E-04
	10	639	113	6	15.2	349.4	0.690	3.46E-04
	11	705	113	6	16.3	308.6	1.564	1.30E-04
	12	57	30	6.6	22.6	450.3	0.649	3.68E-04
	13	589	113	6	14.7	550	0.620	2.96E-04
	14	649	113	6	20.3	361.2	1.958	1.13E-04
	15	250	61	5.9	16	345.4	0.427	1.78E-04
	16	409	80	5.8	11.1	376.1	0.800	3.76E-04
	17	238	58	5.9	18.1	345.4	2.918	1.91E-04
	18	1549	137	7.6	1.8	664.4	0.335	2.06E-04
	19	139	46	7.3	13.9	659.6	0.625	7.98E-04
2	20	1614	138	7.1	11.5	481	1.632	1.01E-04
	21	549	103	6.2	17.2	271.4	0.806	1.82E-04
	22	1681	149	5.9	15.4	405.2	1.527	1.09E-04
	23	1091	127	6.7	23.6	996.4	1.134	1.39E-04
	24	3473	175	6.3	11.5	443	0.620	4.61E-04
	25	529	101	6.1	4	345.4	0.337	8.79E-05
	26	1512	137	7.6	8.2	443	0.471	6.27E-07
	27	668	113	6	20.4	270.2	1.159	1.61E-04
	28	317	73	5.9	7.8	191.1	1.207	2.18E-04
	29	180	50	6.5	4	205.6	0.404	2.68E-04
	30	458	90	6.2	11.5	221.8	0.867	3.53E-04
	31	548	103	6.2	21.9	271.4	0.963	3.85E-05
	32	2385	171	5.9	21.1	624.9	3.211	6.05E-05
	33	639	113	6	15.2	349.4	1.004	8.86E-05
	34	705	113	6	16.3	308.6	2.275	6.38E-05
	35	57	30	6.6	22.6	450.3	0.943	2.55E-07
	36	589	113	6	14.7	550	0.901	1.37E-04

	37	649	113	6	20.3	361.2	2.841	5.40E-05
	38	1491	137	7.6	7.7	467.5	0.844	7.82E-05
	39	250	61	5.9	16	345.4	0.618	1.34E-04
	40	161	50	6.5	10.4	208.7	1.018	1.24E-04
	41	322	76	6.4	24	271.4	0.738	2.55E-07
	42	238	58	5.9	18.1	345.4	4.202	8.63E-05
	43	1549	137	7.6	1.8	664.4	0.482	3.96E-05
	44	139	46	7.3	13.9	659.6	0.898	2.60E-04
3	45	1614	138	7.1	11.5	481	2.333	1.97E-05
	46	549	103	6.2	17.2	271.4	1.150	3.48E-07
	47	1091	127	6.7	23.6	996.4	1.613	4.43E-07
	48	3473	175	6.3	11.5	443	0.877	3.30E-04
	49	529	101	6.1	4	345.4	0.476	1.48E-04
	50	1512	137	7.6	8.2	443	0.665	5.79E-05
	51	668	113	6	20.4	270.2	1.635	7.09E-05
	52	317	73	5.9	7.8	191.1	1.700	1.43E-05
	53	150	48	5.7	3.1	663.3	0.764	3.91E-05
	54	458	90	6.2	11.5	221.8	1.216	1.05E-04
	55	2385	171	5.9	21.1	624.9	4.491	1.15E-05
	56	639	113	6	15.2	349.4	1.402	2.29E-05
	57	705	113	6	16.3	308.6	3.173	5.06E-05
	58	57	30	6.6	22.6	450.3	1.314	3.25E-05
	59	589	113	6	14.7	550	1.255	7.74E-05
	60	649	113	6	20.3	361.2	3.955	1.90E-05
	61	312	71	5.9	24.9	274.5	2.962	5.74E-08
	62	1491	137	7.6	7.7	467.5	1.173	2.79E-06
	63	250	61	5.9	16	345.4	0.858	1.98E-05
	64	161	50	6.5	10.4	208.7	1.412	7.63E-05
	65	322	76	6.4	24	271.4	1.022	3.58E-05
	66	238	58	5.9	18.1	345.4	5.814	5.58E-08
	67	1549	137	7.6	1.8	664.4	0.667	3.93E-05
	68	139	46	7.3	13.9	659.6	1.241	8.28E-05
4	69	1614	138	7.1	11.5	481	3.209	1.00E-05
	70	1681	149	5.9	15.4	405.2	2.977	2.78E-05
	71	1091	127	6.7	23.6	996.4	2.208	8.04E-06
	72	1752	155	6.1	17.7	274.5	1.146	6.97E-08
	73	3473	175	6.3	11.5	443	1.196	1.10E-04
	74	1512	137	7.6	8.2	443	0.906	1.60E-07
	75	668	113	6	20.4	270.2	2.226	3.30E-05
	76	317	73	5.9	7.8	191.1	2.310	1.13E-05
	77	150	48	5.7	3.1	663.3	1.038	7.02E-05
	78	458	90	6.2	11.5	221.8	1.647	3.40E-05

	79	2385	171	5.9	21.1	624.9	6.069	4.43E-07
	80	1602	138	7.1	12	326	0.487	3.49E-07
	81	639	113	6	15.2	349.4	1.891	1.01E-05
	82	705	113	6	16.3	308.6	4.279	7.24E-06
	83	57	30	6.6	22.6	450.3	1.771	2.19E-08
	84	589	113	6	14.7	550	1.690	1.20E-05
	85	649	113	6	20.3	361.2	5.325	5.26E-06
	86	1491	137	7.6	7.7	467.5	1.577	6.75E-06
	87	250	61	5.9	16	345.4	1.152	6.28E-06
	88	161	50	6.5	10.4	208.7	1.896	1.96E-05
	89	1198	137	7.6	11	544.7	1.295	1.55E-05
	90	238	58	5.9	18.1	345.4	7.788	9.55E-06
	91	1549	137	7.6	1.8	664.4	0.893	8.40E-07
	92	139	46	7.3	13.9	659.6	1.661	2.50E-05
5	93	1614	138	7.1	11.5	481	4.278	8.10E-06
	94	549	103	6.2	17.2	271.4	2.103	3.30E-05
	95	1681	149	5.9	15.4	405.2	3.957	2.80E-06
	96	1091	127	6.7	23.6	996.4	2.934	3.50E-07
	97	530	101	6.1	10.8	207.5	2.952	2.54E-06
	98	3473	175	6.3	11.5	443	1.582	3.43E-05
	99	529	101	6.1	4	345.4	0.858	1.84E-05
	100	1512	137	7.6	8.2	443	1.198	1.64E-05
	101	668	113	6	20.4	270.2	2.942	1.63E-07
	102	150	48	5.7	3.1	663.3	1.370	1.87E-05
	103	458	90	6.2	11.5	221.8	2.169	3.01E-05
	104	2385	171	5.9	21.1	624.9	7.977	8.46E-06
	105	639	113	6	15.2	349.4	2.482	2.03E-06
	106	705	113	6	16.3	308.6	5.613	1.16E-06
	107	57	30	6.6	22.6	450.3	2.323	5.25E-06
	108	589	113	6	14.7	550	2.215	5.81E-06
	109	649	113	6	20.3	361.2	6.976	7.89E-07
	110	1491	137	7.6	7.7	467.5	2.065	9.44E-06
	111	1203	137	7.6	16.1	233.1	1.289	6.45E-06
	112	250	61	5.9	16	345.4	1.507	1.17E-06
	113	161	50	6.5	10.4	208.7	2.479	1.04E-06
	114	409	80	5.8	11.1	376.1	2.809	1.09E-06
	115	1198	137	7.6	11	544.7	1.692	6.94E-08
	116	322	76	6.4	24	271.4	1.788	5.80E-07
	117	238	58	5.9	18.1	345.4	10.163	1.08E-08
	118	139	46	7.3	13.9	659.6	2.165	1.53E-05
6	119	1614	138	7.1	11.5	481	5.561	3.52E-07
	120	549	103	6.2	17.2	271.4	2.730	8.32E-06

	121	1681	149	5.9	15.4	405.2	5.130	1.15E-07
	122	1752	155	6.1	17.7	274.5	1.967	6.94E-08
	123	1530	137	7.6	6.1	494.1	2.440	6.56E-09
	124	3473	175	6.3	11.5	443	2.045	9.77E-06
	125	529	101	6.1	4	345.4	1.108	8.93E-06
	126	1512	137	7.6	8.2	443	1.547	1.45E-05
	127	668	113	6	20.4	270.2	3.798	4.86E-06
	128	317	73	5.9	7.8	191.1	3.932	1.13E-08
	129	150	48	5.7	3.1	663.3	1.766	9.53E-06
	130	458	90	6.2	11.5	221.8	2.791	1.35E-05
	131	1511	137	7.6	2.8	615	1.370	1.10E-08
	132	2385	171	5.9	21.1	624.9	10.249	5.28E-06
	133	1602	138	7.1	12	326	0.820	5.53E-06
	134	639	113	6	15.2	349.4	3.185	6.54E-06
	135	57	30	6.6	22.6	450.3	2.979	2.92E-07
	136	589	113	6	14.7	550	2.840	6.54E-07
	137	1491	137	7.6	7.7	467.5	2.644	2.98E-06
	138	1203	137	7.6	16.1	233.1	1.650	2.18E-08
	139	161	50	6.5	10.4	208.7	3.171	6.85E-07
	140	1198	137	7.6	11	544.7	2.163	2.70E-06
	141	322	76	6.4	24	271.4	2.285	9.33E-09
	142	1549	137	7.6	1.8	664.4	1.486	1.06E-08
	143	139	46	7.3	13.9	659.6	2.764	9.53E-06
7	144	319	73	5.9	6.5	193.7	1.345	1.10E-08
	145	1614	138	7.1	11.5	481	7.081	3.73E-06
	146	549	103	6.2	17.2	271.4	3.473	8.35E-07
	147	1681	149	5.9	15.4	405.2	6.518	2.32E-08
	148	1091	127	6.7	23.6	996.4	4.830	2.32E-08
	149	1752	155	6.1	17.7	274.5	2.496	9.07E-07
	150	3473	175	6.3	11.5	443	2.591	3.37E-06
	151	529	101	6.1	4	345.4	1.404	3.90E-06
	152	1512	137	7.6	8.2	443	1.959	5.66E-06
	153	668	113	6	20.4	270.2	4.808	1.50E-06
	154	317	73	5.9	7.8	191.1	4.973	1.83E-09
	155	150	48	5.7	3.1	663.3	2.233	2.71E-06
	156	458	90	6.2	11.5	221.8	3.524	5.14E-06
	157	2385	171	5.9	21.1	624.9	12.924	1.45E-06
	158	639	113	6	15.2	349.4	4.013	1.84E-06
	159	705	113	6	16.3	308.6	9.070	3.27E-09
	160	57	30	6.6	22.6	450.3	3.751	1.39E-06
	161	649	113	6	20.3	361.2	11.250	3.15E-06
	162	1491	137	7.6	7.7	467.5	3.325	6.49E-06

	163	161	50	6.5	10.4	208.7	3.984	2.19E-09
	164	1198	137	7.6	11	544.7	2.716	1.80E-09
	165	238	58	5.9	18.1	345.4	16.291	4.70E-09
	166	1549	137	7.6	1.8	664.4	1.865	1.31E-07
	167	319	73	5.9	6.5	193.7	1.686	1.12E-06
	168	1614	138	7.1	11.5	481	8.863	3.09E-06
	169	549	103	6.2	17.2	271.4	4.344	2.25E-07
	170	1681	149	5.9	15.4	405.2	8.144	3.60E-06
	171	1752	155	6.1	17.7	274.5	3.114	2.18E-06
	172	3473	175	6.3	11.5	443	3.230	1.22E-06
	173	529	101	6.1	4	345.4	1.750	1.22E-06
	174	668	113	6	20.4	270.2	5.989	2.11E-09
	175	317	73	5.9	7.8	191.1	6.189	1.92E-06
	176	150	48	5.7	3.1	663.3	2.778	1.08E-08
	177	988	127	6.7	23.4	278	3.424	6.93E-08
	178	458	90	6.2	11.5	221.8	4.379	8.16E-07
	179	1511	137	7.6	2.8	615	2.148	1.13E-08
8	180	548	103	6.2	21.9	271.4	4.826	1.98E-08
	181	2385	171	5.9	21.1	624.9	16.044	1.96E-07
	182	639	113	6	15.2	349.4	4.979	1.79E-09
	183	705	113	6	16.3	308.6	11.249	1.08E-08
	184	57	30	6.6	22.6	450.3	4.651	1.66E-07
	185	649	113	6	20.3	361.2	13.941	2.27E-08
	186	312	71	5.9	24.9	274.5	10.420	1.20E-08
	187	1491	137	7.6	7.7	467.5	4.119	2.08E-06
	188	250	61	5.9	16	345.4	3.000	1.27E-06
	189	409	80	5.8	11.1	376.1	5.582	1.72E-09
	190	1198	137	7.6	11	544.7	3.359	4.70E-06
	191	322	76	6.4	24	271.4	3.548	2.15E-09
	192	238	58	5.9	18.1	345.4	20.143	1.81E-09
	193	1549	137	7.6	1.8	664.4	2.305	2.69E-06
	194	139	46	7.3	13.9	659.6	4.284	2.31E-06

Appendix B. Implementation of an Element Removal Algorithm for URM infill walls in OpenSees.

This section presents snippets of the relevant scripts implemented in OpenSees for the correct implementation and use of the element removal algorithm for URM infill walls. All the source code for OpenSees, including the scripts mentioned here can be found in the OpenSees repository on GitHub: <https://github.com/OpenSees/OpenSees>. Example applications and further explanations of the infill wall model and element removal can be found in Günay and Mosalam (2010).

Appendix B.1 Element.h

In this snippet, the input parameters of the Removal Recorder are shown inside the source code that declares all the variables related to elements in OpenSees.

111	////////// Addig element removal - MMT //////////
112	// virtual int getRemCriteria(int CompID,int critID, double &value) (opcerr<<"Element::getRemCriteria() - not implemented for element "<<this->getTag()<<endl; return -1;);
113	// new
114	virtual int getRemCriteriawall(int compID,int critID, const char* filenamewall, int nTagbotn11, int nTagmidn11, int nTagtopn11, int globgrav11, double &value, double &checkvalue1, Domain *theDomain) { opcerr<<"Element::getRemCriteriawall() - not implemented for element "<<this->getTag()<<endl; return -1;};
115	//
116	////////// End element removal

Appendix B.2 ForceBeamColumn3d.h

In this snippet, the input parameters for the Removal Recorder are shown inside of the source code that declares all the variables needed for a *ForceBeamColumn3d* element.

89	////////// Addig element removal - MMT //////////
90	// int getRemCriteria(int CompID,int critID, double &critValue);
91	// new
92	int getRemCriteriawall(int compID,int critID, const char* filenamewall, int nTagbotn11, int nTagmidn11, int nTagtopn11, int globgrav11, double &value, double &checkvalue1, Domain *theDomain);
93	//
94	////////// End element removal

Appendix B.3 RemoveRecorder.ccp

In this snippet, all the possible cases that the Remove Recorder can handled are defined and then the user is asked to identify what kind of removal are they trying to perform. It can be seen from

lines 502 to 506 that *case 7* is the one that refers to the element removal of URM infill walls. Other cases refer to the removal of other elements, e.g., columns, which are not currently available in the standard version of OpenSees and only available in a local version.

```

435 RemoveRecorder::checkEleRemoval(Element* theEle, int
    &theComponent,const Vector criteria)
436 {
437     int secFlag = 0;
438
439     for (int k=0; k<numRules; k++) {
440         double currentValue = 0;
441         double checkwallvalue1=0;
442         // changed
443         // if (int(criteria[2*k] != 0 && criteria[2*k] != 7)) {
444         // if (theEle->getRemCriteria(theComponent,
445         int(criteria[2*k]), currentValue) != 0) {
446         // opserr<<" RemoveRecorder::checkRemoval() - failed to
447         // retrieve information from element "<<theEle->getTag()<<endl;
448         // return -1;
449         // }
450         // }
451         // new
452         if (int(criteria[2*k] == 7)) {
453         // const char* filenameinf;
454         //
455         theEle->getRemCriteriaWall(theComponent,
456         int(criteria[2*k]), fileNameinf, nTagbotn, nTagmidn, nTagtopn,
457         globgrav, currentValue, checkwallvalue1, theDomain);
458         // if (theEle->getRemCriteria(theComponent,
459         int(criteria[2*k]), currentValue) != 0) {
460         // opserr<<" RemoveRecorder::checkRemoval() - failed to
461         // retrieve information from element "<<theEle->getTag()<<endl;
462         // return -1;
463         // }
464         //
465         switch (int(criteria[2*k])) {
466
467             case 0:
468                 // no removal,for comparison;
469                 break;
470
471             case 1:
472                 // min strain (maximum compressive), notice the
473                 inequality direction
474                 if (currentValue <= criteria [2*k+1])
475                 secFlag ++;
476                 break;
477
478             case 2:
479                 // max strain
480                 if (currentValue >= criteria [2*k+1])
481                 secFlag ++;
482                 break;
483
484             case 3:

```

```

479         // axial failure damage index
480         if (currentValue >= criteria [2*k+1])
481             secFlag ++;
482         break;
483
484     case 4:
485         // flexural failure damage index
486         if (currentValue >= criteria [2*k+1])
487             secFlag ++;
488         break;
489
490     case 5:
491         // axial limit state
492         if (currentValue >= criteria [2*k+1])
493             secFlag ++;
494         break;
495
496     case 6:
497         // shear limit state
498         if (currentValue >= criteria [2*k+1])
499             secFlag ++;
500         break;
501     // new
502     case 7:
503         // infill wall element
504         if (currentValue >= checkwallvalue1)
505             secFlag ++;
506         break;
507     //
508     default:
509         // unknown removal criteria switch
510         opserr<<"RemoveRecorder::checkRemoval() - Failed to
identify removal criterion "<<criteria [2*k]<<endl;
511         return -1;
512         break;
513     }
514 }
515 }
516
517 opserr<<"secFlag = "<<secFlag<<endl;
518
519 return secFlag;
520 }

```

Appendix B.4 TclRecorderCommands.cpp

```

94 ///////////////////////////////////////////////////MMT: Adding element removal
95 #include <RemoveRecorder.h>
96 ///////////////////////////////////////////////////

117 // new
118 TCL_Char *fileNameInf = 0;

```

Appendix B.5 ForceBeamColumn3d.cpp

In this snippet, the actual check to determine whether the URM infill wall has exceeded the collapse criteria or not is performed.

From lines 2474 to 2485, starting from the previously defined *globgrav11* (which is the global axis of the model in the direction of the gravity), the remaining two directions are established in the common case in which the gravity is defined in the Y-direction (*globgrav11==2*), the direction *indwdir1* refers to the X-direction and the *indwdir2* refers to the Z-direction.

The lines 2486 to 2493 use the coordinates of node I (*crdIa1*) and node J (*crdJa1*) to obtain the unit vector (direction) along the in-plane and the out-of-plane direction of the infill panel. Then the lines 2495 to 2508 obtain the displacements along the IP and OOP direction for the bottom, middle and top nodes of the diagonal element that represents the URM infill wall.

Line 2510 calculates the total IP displacement of the infill wall as:

$$\Delta_{IP} = |u_{IP_{Top\ node}} - u_{IP_{Bottom\ node}}|$$

Line 2511 calculates the total OOP displacement of the middle point of the diagonal with respect to the chord that connects these two points as:

$$\Delta_{OOP} = \left| u_{OOP_{Middle\ node}} - \frac{u_{OOP_{Top\ node}} - u_{OOP_{Bottom\ node}}}{2} \right|$$

Finally, the while loop that starts at line 2529 checks the previously found values Δ_{IP} and Δ_{OOP} against the values in the collapse criteria curve which is given in a separate file, interpolating linearly between two consecutive points in the curve to see whether the displacements of the URM infill wall exceed the collapse criteria and if so it initiates the removal algorithm.

```

2459 Int
2460 ForceBeamColumn3d::getRemCriteriaWall(int compID, int critID, const
char* filenameWall, int nTagbotn11, int nTagmidn11, int nTagtopn11,
int globgrav11, double &value, double &checkvalue1, Domain
*theDomain)
2461 {
2462
2463     if (critID == 7) {
2464         //
2465         double oofwallresp;
2466         // determine the in plane horizontal deformation axis
2467         // and the out of plane horizontal deformation axis
2468         Node *theNode1a = theDomain->getNode(nTagbotn11);
2469         Node *theNode3a = theDomain->getNode(nTagtopn11);
2470         const Vector &crdIa1 = theNode1a->getCrds();
2471         const Vector &crdJa1 = theNode3a->getCrds();

```

```

2472     int indwdir1;
2473     int indwdir2;
2474     if (globgrav11==1) {
2475         indwdir1=1;
2476         indwdir2=2;
2477     }
2478     else if (globgrav11==2) {
2479         indwdir1=0;
2480         indwdir2=2;
2481     }
2482     else if (globgrav11==3) {
2483         indwdir1=0;
2484         indwdir2=1;
2485     }
2486     double dirla1=crdJa1(indwdir1)-crdIa1(indwdir1);
2487     double dir2a1=crdJa1(indwdir2)-crdIa1(indwdir2);
2488     double dirsumdum=sqrt(dirla1*dirla1+dir2a1*dir2a1);
2489     double dirlinp=dirla1/dirsumdum;
2490     double dir2inp=dir2a1/dirsumdum;
2491
2492     double dirloop=-dir2inp;
2493     double dir2oop=dirlinp;
2494
2495     Node *theNode1 = theDomain->getNode(nTagbotn11);
2496     const Vector &theResponsewall = theNode1->getTrialDisp();
2497     double
2498     valbotinfn=theResponsewall(indwdir1)*dirlinp+theResponsewall(indwdir
2499     2)*dir2inp;
2500     double
2501     valbotoutfn=theResponsewall(indwdir1)*dirloop+theResponsewall(indwdir
2502     2)*dir2oop;
2503
2504     Node *theNode2 = theDomain->getNode(nTagmidn11);
2505     const Vector &theResponsewall2 = theNode2->getTrialDisp();
2506     double
2507     valmidinfn=theResponsewall2(indwdir1)*dirlinp+theResponsewall2(indwdir
2508     2)*dir2inp;
2509     double
2510     valmidoutfn=theResponsewall2(indwdir1)*dirloop+theResponsewall2(indw
2511     dir2)*dir2oop;
2512
2513     Node *theNode3 = theDomain->getNode(nTagtopn11);
2514     const Vector &theResponsewall3 = theNode3->getTrialDisp();
2515     double
2516     valtopinfn=theResponsewall3(indwdir1)*dirlinp+theResponsewall3(indwdir
2517     2)*dir2inp;
2518     double
2519     valtopoutfn=theResponsewall3(indwdir1)*dirloop+theResponsewall3(indw
2520     dir2)*dir2oop;
2521
2522     value = sqrt(pow((valtopinfn-valbotinfn),2.0));
2523     double valoutchck=valmidoutfn-(valtopoutfn+valbotoutfn)/2.0;
2524     oofwallresp = sqrt(pow(valoutchck,2.0));
2525
2526     //
2527     double outplanevaldat; // variable for input value
2528     double inplanevaldat; // variable for input value
2529     double outplanevaldat1; // variable for input value

```

```

2517         double inplanevaldat1; // variable for input value
2518         ifstream indata;
2519
2520         if (filenameewall!=NULL) {
2521             //
2522             indata.open(filenameewall); // opens the file
2523             if(!indata) { // file couldn't be opened
2524                 cerr << "Error: file for infill wall could not be
opened" << endl;
2525                 exit(1);
2526             }
2527             checkvalue1=0.0;
2528             int counterdum=0;
2529             while ( !indata.eof() ) { // keep reading until end-of-
file
2530                 counterdum=counterdum+1;
2531                 indata >> outplanevaldat >> inplanevaldat ; // sets EOF
flag if no value found
2532                 if (counterdum!=1) {
2533                     if (oofwallresp >= outplanevaldat1 && oofwallresp <=
outplanevaldat) {
2534                         checkvalue1= inplanevaldat1+(oofwallresp-
outplanevaldat1)/(outplanevaldat-outplanevaldat1)*(inplanevaldat-
inplanevaldat1);
2535                         break;
2536                     }
2537                 }
2538                 indata >> outplanevaldat1 >> inplanevaldat1;
2539                 if (oofwallresp >= outplanevaldat && oofwallresp <=
outplanevaldat1) {
2540                     checkvalue1= inplanevaldat+(oofwallresp-
outplanevaldat)/(outplanevaldat1-outplanevaldat)*(inplanevaldat1-
inplanevaldat);
2541                     break;
2542                 }
2543             }
2544             indata.close();
2545         }
2546         return 0;
2547     }
2548 }

```

Appendix C. Summary of fiber properties.

This section presents a summary of the fiber properties used in the different types of infill walls from the two archetype buildings studied throughout this dissertation. Furthermore, the resulting P-M interaction curve and the displacement capacity curves are presented for each fiber section.

Appendix C.1 Fiber properties of 4-story case study building.

Table C.1 Fiber properties of first floor longitudinal infill walls.

Fiber ID	F_y	A_i	z_i	σ_i	ϵ_y
1	11.2208	0.0033376	0.73163	3361.9	0.00044826
2	21.5232	0.0059458	0.38142	3619.9	0.00048265
3	30.4017	0.0080756	0.27003	3764.6	0.00050195
4	42.0072	0.010756	0.19543	3905.3	0.00052071
5	80.1471	0.019071	0.10243	4202.5	0.00056033
6	80.1471	0.019071	-0.10243	4202.5	0.00056033
7	42.0072	0.010756	-0.19543	3905.3	0.00052071
8	30.4017	0.0080756	-0.27003	3764.6	0.00050195
9	21.5232	0.0059458	-0.38142	3619.9	0.00048265
10	11.2208	0.0033376	-0.73163	3361.9	0.00044826

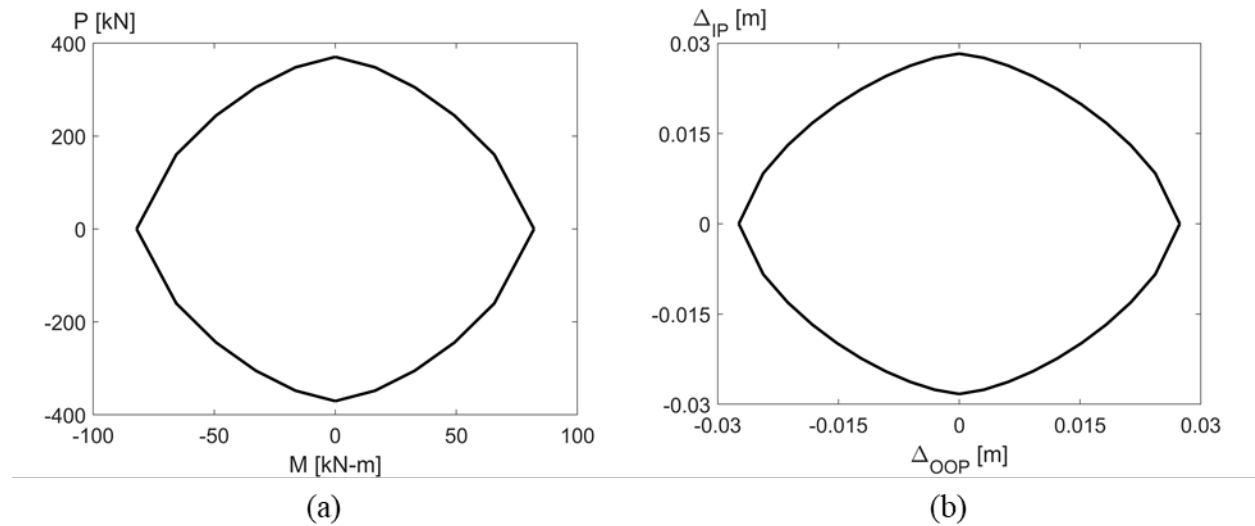


Figure C.1 (a) P-M interaction diagram and (b) Displacement capacity curve of first-floor longitudinal infill walls.

Table C.2 Fiber properties of longitudinal infill walls for floors 2-4.

Fiber ID	F_y	A_i	z_i	σ_i	ε_y
1	10.926	0.0013515	1.1128	8084.3	0.0010779
2	20.957	0.0036424	0.58012	5753.7	0.00076716
3	29.602	0.0061616	0.4107	4804.3	0.00064058
4	40.903	0.010079	0.29723	4058	0.00054107
5	78.039	0.026945	0.15579	2896.2	0.00038616
6	78.039	0.026945	-0.15579	2896.2	0.00038616
7	40.903	0.010079	-0.29723	4058	0.00054107
8	29.602	0.0061616	-0.4107	4804.3	0.00064058
9	20.957	0.0036424	-0.58012	5753.7	0.00076716
10	10.926	0.0013515	-1.1128	8084.3	0.0010779

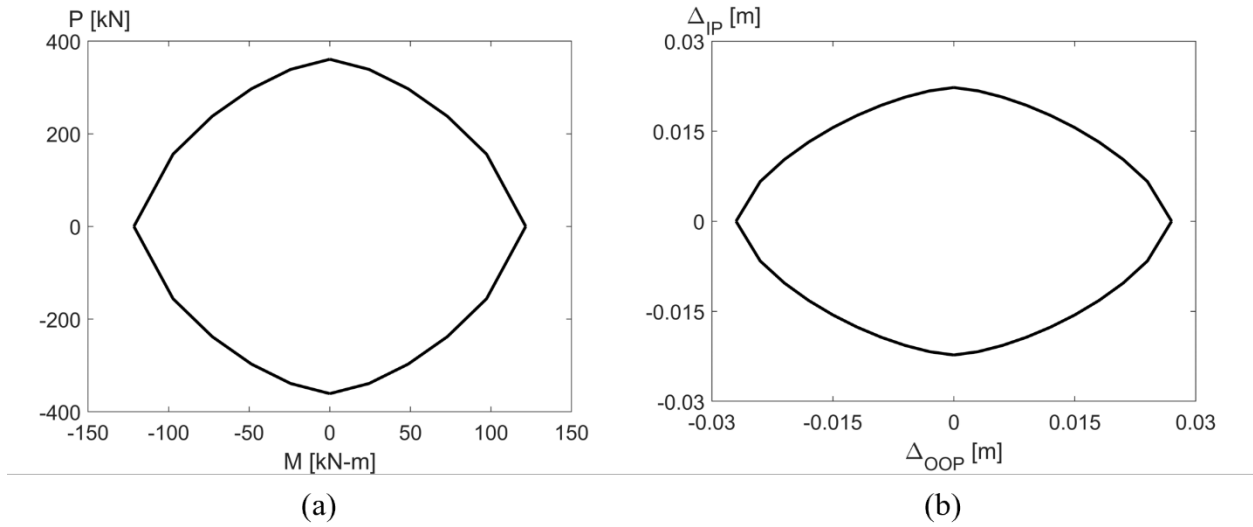


Figure C.2 (a) P-M interaction diagram and (b) Displacement capacity curve of longitudinal infill walls for floors 2-4.

Table C.3 Fiber properties of first floor transverse infill walls.

Fiber ID	F_y	A_i	z_i	σ_i	ϵ_y
1	9.5979	0.0024344	0.60969	3942.6	0.00052568
2	18.41	0.0047024	0.31785	3915.1	0.00052201
3	26.005	0.0066669	0.22503	3900.5	0.00052007
4	35.932	0.0092441	0.16286	3887	0.00051827
5	68.555	0.01776	0.085358	3860.1	0.00051468
6	68.555	0.01776	-0.085358	3860.1	0.00051468
7	35.932	0.0092441	-0.16286	3887	0.00051827
8	26.005	0.0066669	-0.22503	3900.5	0.00052007
9	18.41	0.0047024	-0.31785	3915.1	0.00052201
10	9.5979	0.0024344	-0.60969	3942.6	0.00052568

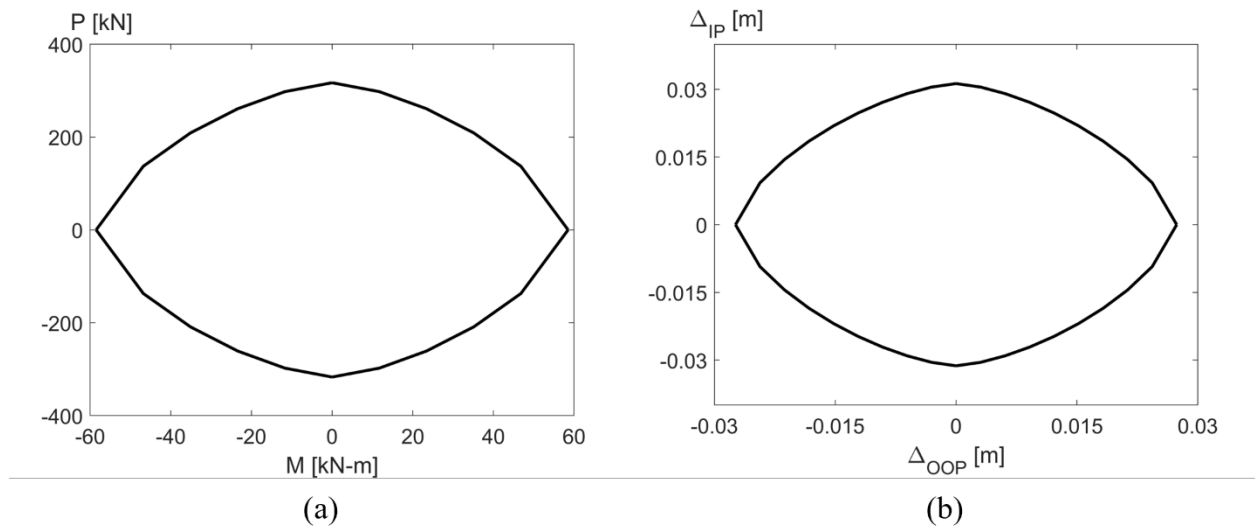


Figure C.3 (a) P-M interaction diagram and (b) Displacement capacity curve of first floor transverse infill walls.

Table C.4 Fiber properties of longitudinal infill walls for floors 2-4.

Fiber ID	F_y	A_i	z_i	σ_i	ε_y
1	9.2653	0.0009099	0.92729	10183	0.0013577
2	17.772	0.0027081	0.48343	6562.5	0.00087501
3	25.104	0.0048287	0.34225	5198.9	0.00069318
4	34.687	0.0082978	0.24769	4180.2	0.00055736
5	66.18	0.024477	0.12982	2703.8	0.0003605
6	66.18	0.024477	-0.12982	2703.8	0.0003605
7	34.687	0.0082978	-0.24769	4180.2	0.00055736
8	25.104	0.0048287	-0.34225	5198.9	0.00069318
9	17.772	0.0027081	-0.48343	6562.5	0.00087501
10	9.2653	0.0009099	-0.92729	10183	0.0013577

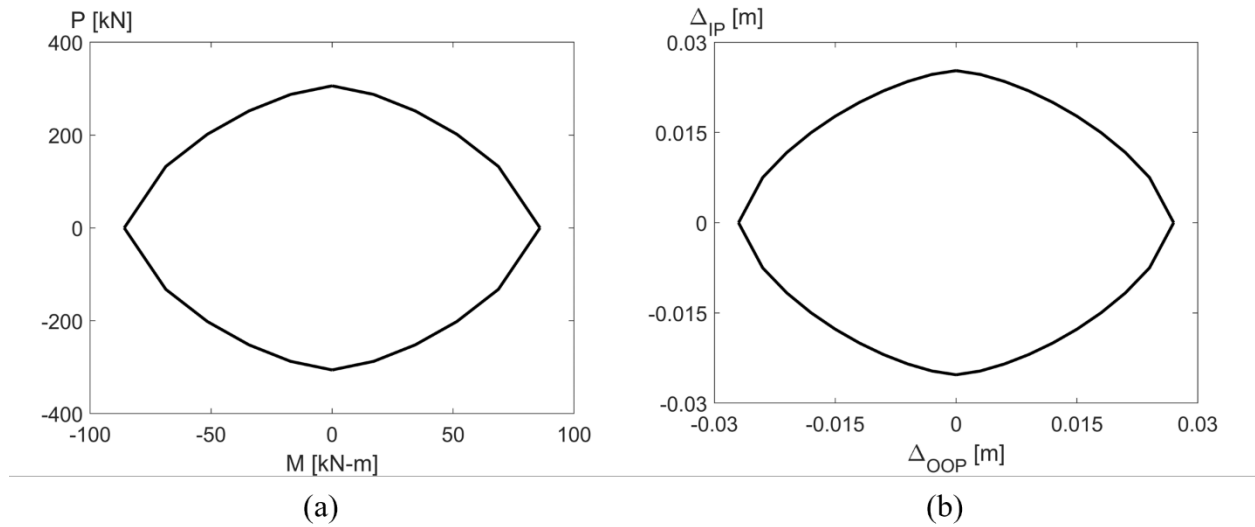
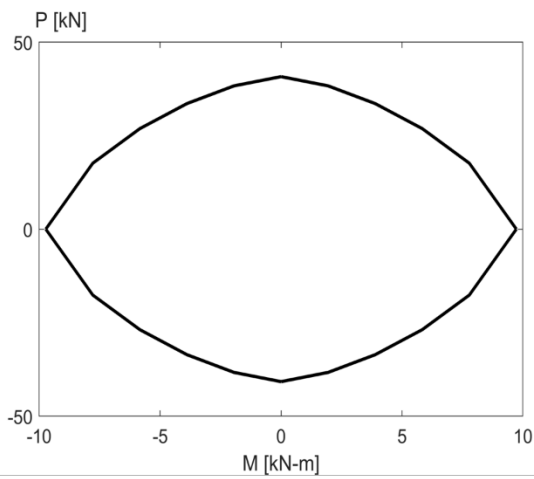


Figure C.4 (a) P-M interaction diagram and (b) Displacement capacity curve of transverse infill walls for floors 2-4.

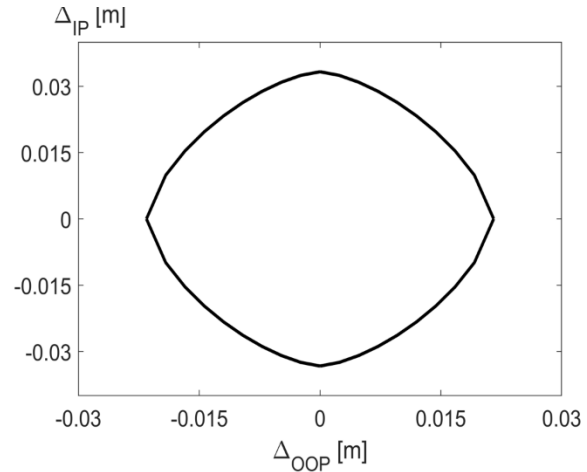
Appendix C.2 Fiber properties of 6-story case study building.

Table C.5 Fiber properties of first floor infill walls.

Fiber ID	F_y	A_i	z_i	σ_i	ϵ_y
1	1.2346	0.0015267	0.78784	808.6684	0.53911224
2	2.3681	0.0041254	0.41073	574.0252	0.38268345
3	3.3449	0.0069884	0.29078	478.6443	0.31909621
4	4.6218	0.0114469	0.21044	403.7639	0.26917592
5	8.8182	0.0306810	0.11030	287.4145	0.19160964
6	8.8182	0.0306810	-0.11030	287.4145	0.19160964
7	4.6218	0.0114469	-0.21044	403.7639	0.26917592
8	3.3449	0.0069884	-0.29078	478.6443	0.31909621
9	2.3681	0.0041254	-0.41073	574.0252	0.38268345
10	1.2346	0.0015267	-0.78784	808.6684	0.53911224



(a)

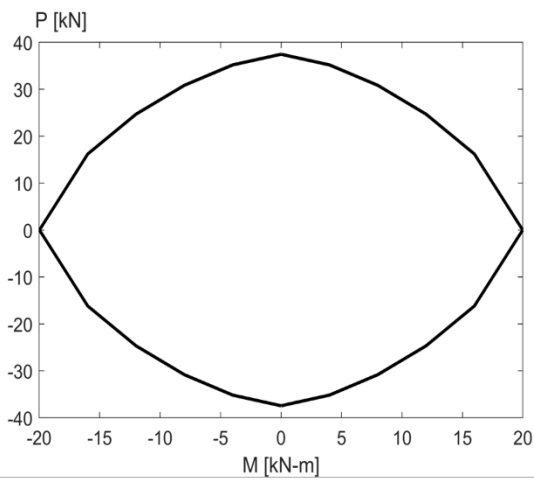


(b)

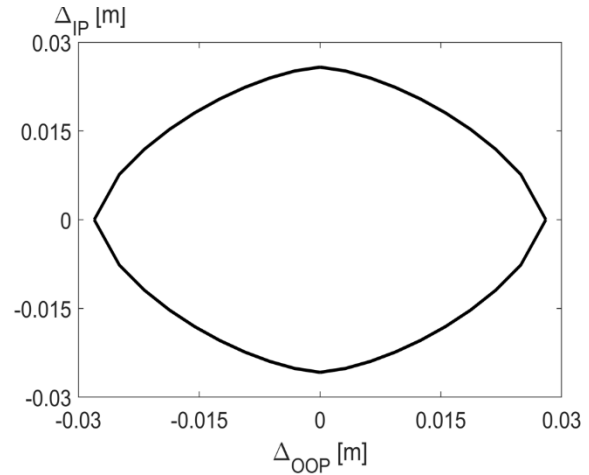
Figure C.5 (a) P-M interaction diagram and (b) Displacement capacity curve of first floor infill walls.

Table C.6 Fiber properties of infill walls for floors 2-4.

Fiber ID	F_y	A_i	z_i	σ_i	ϵ_y
1	1.1339	0.0000175	1.76176	64819.7	43.2131572
2	2.1749	0.0002452	0.91847	8868.7	5.912433919
3	3.0721	0.0009945	0.65024	3089.1	2.059367577
4	4.2448	0.0036886	0.47059	1150.8	0.767202646
5	8.0988	0.0506042	0.24665	160.0	0.106695227
6	8.0988	0.0506042	-0.24665	160.0	0.106695227
7	4.2448	0.0036886	-0.47059	1150.8	0.767202646
8	3.0721	0.0009945	-0.65024	3089.1	2.059367577
9	2.1749	0.0002452	-0.91847	8868.7	5.912433919
10	1.1339	0.0000175	-1.76176	64819.7	43.2131572



(a)



(b)

Figure C.6 (a) P-M interaction diagram and (b) Displacement capacity curve of infill walls for floors 2-4.