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ANALYTICAL FRAGILITY CURVES OF HIGH-RISE REINFORCED CONCRETE SHEAR WALL BUILDINGS

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Abstract

Chile, as other countries along the Pacific rim, is threatened continuously by various extreme natural phenomena. The 2010 Mw 8.8 Maule earthquake, the 2014 Mw 8.1 Pisagua earthquake, the 2015 Mw 8.3 Illapel earthquake, and their subsequent tsunamis, generated comprehensive structural damage in the built environment. After the 2010 Chilean earthquake, close to 2% of the estimated 2,000 reinforced concrete (RC) buildings taller than 9 stories suffered substantial damage due to the ground motion. Consequently, calculating the probability of RC buildings of exceeding a given damage state during potential future seismic events is of paramount importance. For this purpose the definition of reliable fragility functions for these structures is required. The construction of fragility curves is part of the process of defining the seismic vulnerability of these systems. Herein, analytical fragility curves are built using a numerical model of a 20-story shear-wall prototype building. First, the two-dimensional prototype building was defined and it was modeled in OpenSEES to simulate the nonlinear seismic response of such building. The prototype building was defined based on representative characteristics of actual buildings (i.e. story height, wall thickness, and seismic weight per unit area). For simplicity, rectangular cross-sections were considered for the walls and rectangular beams were used to simulate the bending behavior of the slabs. For each intensity measure of the earthquake, the seismic variability is accounted for using a set of 28 ground motions. The fragility curves obtained in this investigation are intended to further understand the seismic behavior of RC shear-wall buildings and they may be used in damage evaluation and risk assessment studies.

Keywords: reinforced concrete walls; buildings; nonlinear analysis; incremental dynamic analysis; fragility

1. Introduction

Within different types of typical reinforced concrete (RC) buildings built in Chile [1], RC shear-walls buildings are one of the most commonly used. In this type of buildings, a structural system configured primarily by RC walls resist both the lateral and vertical loads, and the wall density varies between 1.5 and 3.5% [2, 3]. For buildings constructed before 1985 the stiffnesses in the plan layout have a marked regularity, presenting symmetry in one or the two perpendicular axes, and most of the RC walls are continuous in height. This latter characteristic evolved to a slight discontinuity in the wall cross-section along the height when buildings built after 1985 are inspected [4]. Fig. 1 illustrates a typical floor-plan of a RC shear-wall building in Chile constructed after 1985.

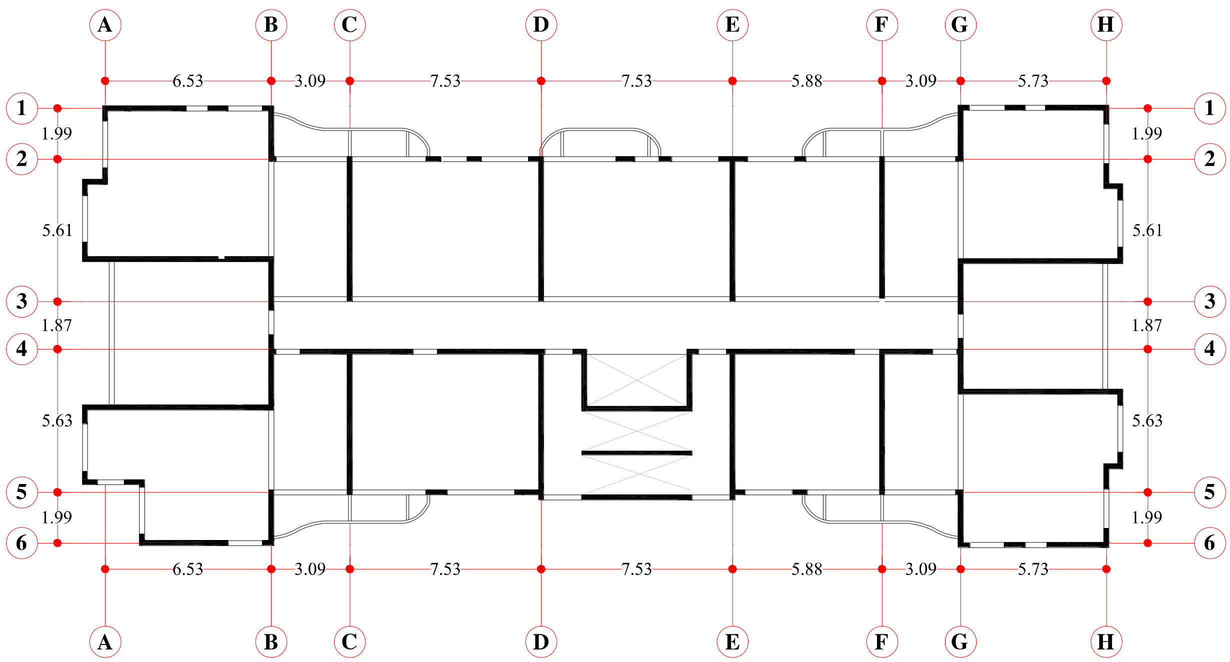


Fig. 1 – Plan view of a typical RC shear wall building in Chile (dimensions in meters)

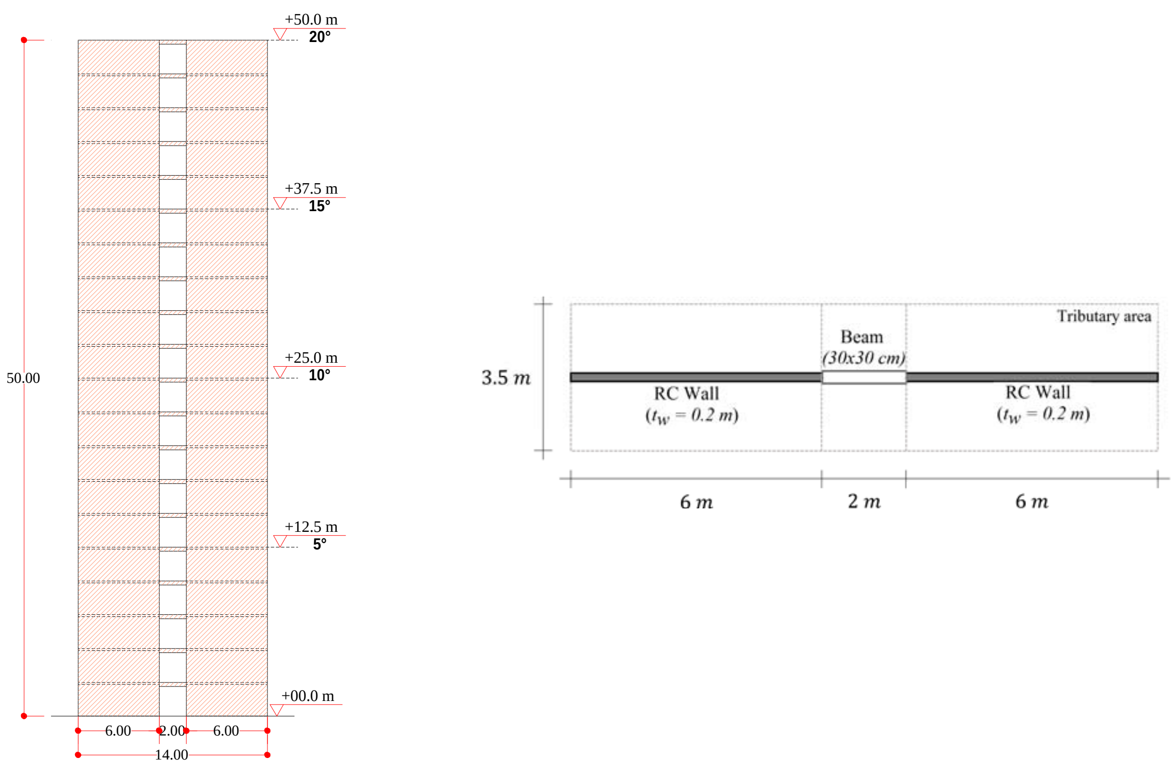
From the past Chilean severe earthquakes (1985, 2010, 2014 and 2015), it has been observed that RC shear-wall buildings have performed well. This fact has led to an increase of confidence on the use of RC walls, but there is a lack of understanding on how the entire building will behave inelastically under severe conditions. Although after the large 2010 earthquake a good performance of RC walls was observed in general, unexpected issues were observed [5, 6, 7], especially in high-rise buildings with high levels of axial loads.

Since RC shear-walls buildings are the preferred structural system in Chile for residential uses, study their seismic risk is of a great importance, given the seismic hazard levels expected in Chile. This work is a first step towards an extensive research program aimed to evaluate the vulnerability and risk of Chilean RC shear-wall buildings. To achieve this goal, fragility curves of a 20-story RC shear-wall building are developed in this paper. The fragility curves were estimated with incremental dynamic analyses [8] using a simplified two-dimensional (2D) model of a prototype building. For the ground motions, the two horizontal components of 14 stations that recorded 2010 Chile earthquake were used. The damages state was evaluated from the response of the walls in the first story, and the four damage states defined by HAZUS technical manual [9] for RC walls were used. As part of the analyses, the inter-story drift of the first floor is used to identify the damage state of the entire building frame.

2. Prototype building

The fragility curves of this study were obtained using a 20-story RC prototype shear-wall building. Since the incremental dynamic analysis consist on running several time-history analyses, a simplified 2D model was used to represent the seismic behavior of a shear-wall building. The 2D prototype building used for this study is shown in Fig. 2. The search of a simple representation of shear-wall buildings must find a balance between low computational cost and accurate results. The prototype building consists of two walls connected by slender beams. These rectangular beams are used for simplicity to simulate the bending stiffness and strength of the slabs. Rectangular cross-sections were considered for the walls in this study, but T-shape walls will be considered in future studies.

The considered length of each wall is 6 m, the thickness 0.2 m, and the free space between the walls is 2 m. The tributary area for the analyzed slice of the building is 49 m^2, which implies a wall density in the direction of analysis 4.9%. This wall density is in the upper range of Chilean buildings [10], whereas the mean wall density of damaged buildings after 2010 earthquake was found to be 2.9 % [2, 3]. An inter-story height of 2.5 m was selected to represent usual construction practice, and the total height of the prototype building is 50 m. The bending stiffness of the slab is simulated with a 0.3 m wide and 0.3 m depth-coupling beam connecting the two walls.

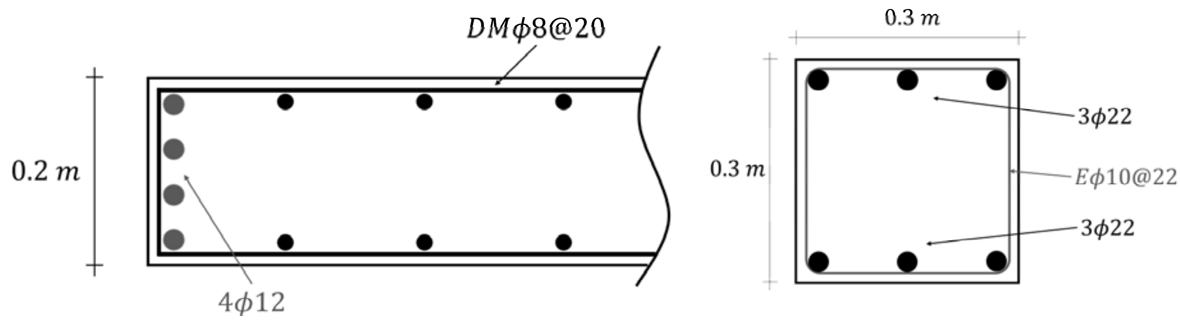


(a) (b)

Fig. 2 – Plan and elevation views of prototype building: (a) Elevation, (b) Plan view with tributary area

Common material properties were used for designing the prototype buildings. A specified compressive strength of f'_c = 25MPa was used for concrete and a specified yield strength and ultimate strength of 420 MPa and 630 MPa, respectively, was used for the steel.

The design of the prototype building (Fig. 2) was carried out following the Chilean codes NCh433 [11], DS60 [12], and DS 61 [13]. An elastic response spectrum structural analysis was performed in ETABS [14] to design the reinforcement of walls and beams. The fundamental period of building obtained from the elastic analysis was 0.9 sec., which is similar to the periods found in real RC shear-walls buildings in Chile [2]. The ultimate design shear was 2012 kN which is equivalent to 19.6% of the seismic weight of the prototype building. The selected reinforcement for the walls and beams is shown in Fig. 3. Both walls were designed with 4 ϕ12 mm bars as longitudinal boundary reinforcement, and ϕ8 mm bars spaced at 20 cm as longitudinal and transverse web reinforcement, along the height of the building. The resulting web reinforcement ratio is 0.0025 for both longitudinal and transverse direction. This ratio is equivalent to the minimum reinforcement ratio required by ACI 318 [15]. It is important to notice that in this study, special boundary elements were not considered in the walls to represent construction practice in Chile before the 2010 earthquake. The reinforcement detailing of the beams are shown in Fig. 3b.



(a) (b)

Fig. 3 – Structural elements cross-sections: (a) RC wall, (b) Coupling beam

3. Numerical non-linear model

The prototype building described in Fig. 2 and designed according to the previous section is modeled in OpenSEES platform [16], where an incremental dynamic analysis is performed to obtain the fragility curves of the building.

3.1 Macroscopic element model

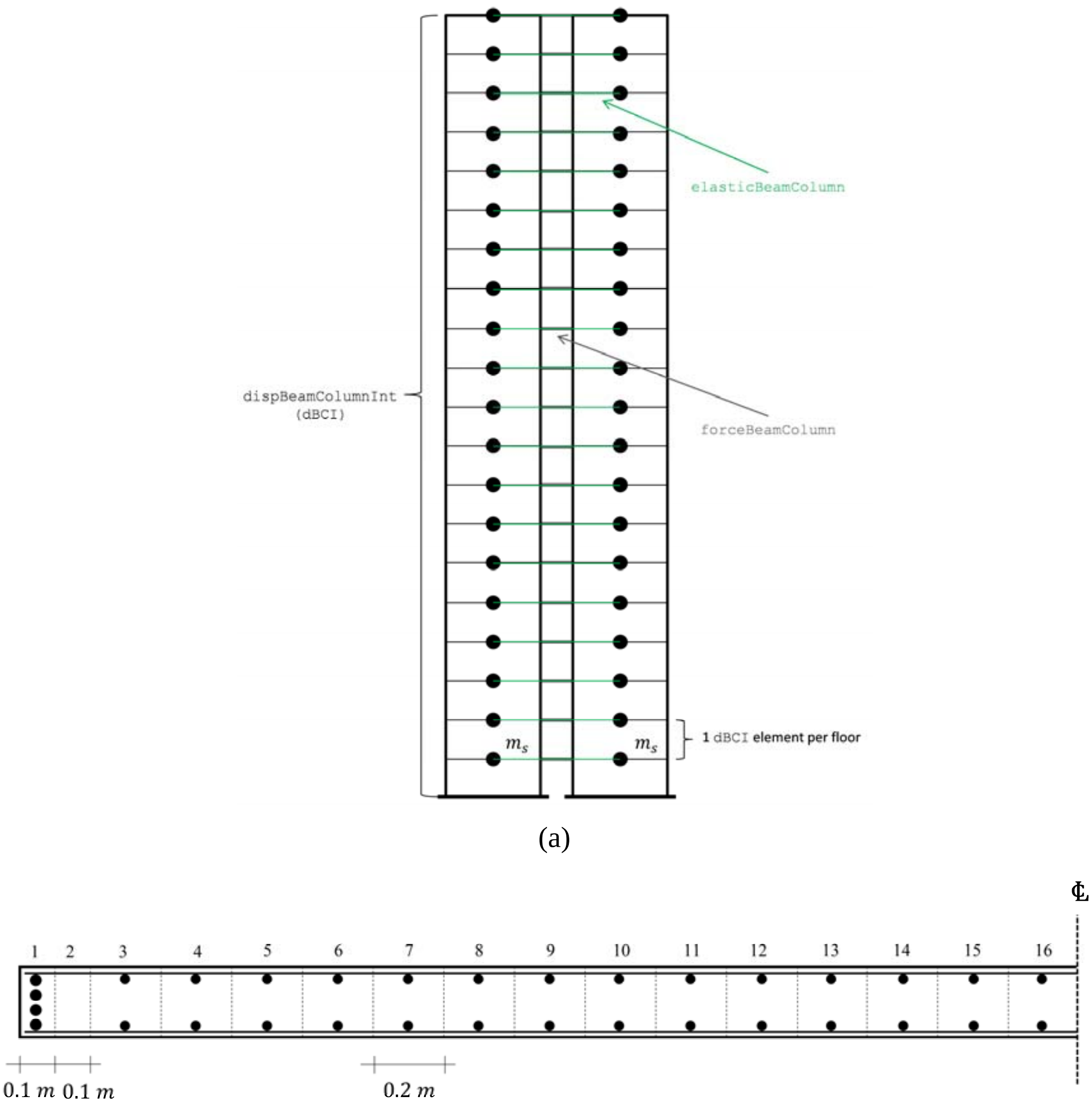
The prototype building was modeled with two-node frame elements to represent both the RC walls and beams. The beams are represented with “forceBeamColumn” element in OpenSEES. This element is defined on an iterative force-based formulation with the advantage to be able to use different integrations schemes, and consider both, lumped or distributed plasticity. For this work, three integration points and distributed plasticity was considered for beams elements, and they were modeled with two-nodes frame elements from the vertical axis of the right wall to the vertical axis of the left wall. The portion of the beams located inside the walls was assumed rigid, and the nonlinear behavior of the beam was considered within their material configuration.

RC walls are modeled with the “Flexure-Shear Interaction Displacement-Based Beam-Column” element (“dispBeamColumnInt” element) in OpenSEES, which is a displacement-based beam-column element with distributed plasticity capable of including interaction between flexural and shear components. This element is based on a study [17] where improvements were made to the MVLEM (Multi-vertical-line element model) developed by Vulcano [18]. Following the recommendations of Vulcano et al. [19] for this type of element, a center of rotation localized at 40% of the wall height, and three integration points were taken. Furthermore, 32 fibers were considered along the wall length, with smeared properties of the concrete and steel located at the center of the fiber. More information about this element can be found elsewhere [17, 20]. Searching for a balance between a computational cost and accuracy of the model Kircher et al. [21] recommends that the height of the wall elements should be approximately equal to the length of the plastic hinge likely to be form. Additionally, Paulay and Priestley [22] establish the length of the plastic hinge between $0.3l_w$ (with l_w = wall length) and $0.8l_w$, hence a suitable hinge length for the walls modeled in this study is $l_p = 0.5l_w = 3m$. Therefore, the height of the wall elements should be around $3m$, which results in one wall element per floor for the RC walls.

On each floor a rigid diaphragm was imposed to represent the displacement compatibility given by the slab. The rigid diaphragm was achieved by adding a rigid element ("elasticBeamColumn" with a larger cross-sectional area). P-Δ effects were not considered in vertical elements, since the expected roof drift was less than 1%. Finally, the two walls were fixed to the ground. Fig. 4 shows a schematic representation of the model built in OpenSEES.

3.2 Constitutive relationships for concrete and steel

The materials considered for the prototype building are of common use in OpenSEES simulations to model RC elements. Reinforcing steel bars are modeled using a uniaxial modified Menegotto–Pinto model (Steel02 model in OpenSEES) with isotropic strain hardening [23]. Concrete is modeled using a uniaxial constitutive model with tensile strength, which considers the effects of biaxial compression softening, and nonlinear tension stiffening, based on the Thorenfeldt curve (Concrete06 model in OpenSEES) [17, 24]. Since limited reinforcement confinement was considered in the design, to represent the Chilean practice before the 2010 earthquake, unconfined concrete was considered for both walls and beams. In future studies, concrete confinement may be considered in walls with larger amount of transverse reinforcement.



(a)

(b)

Fig. 4 – OpenSEES model of RC shear-wall building frame: (a) Elevation, (b) Plan view with considered fibers

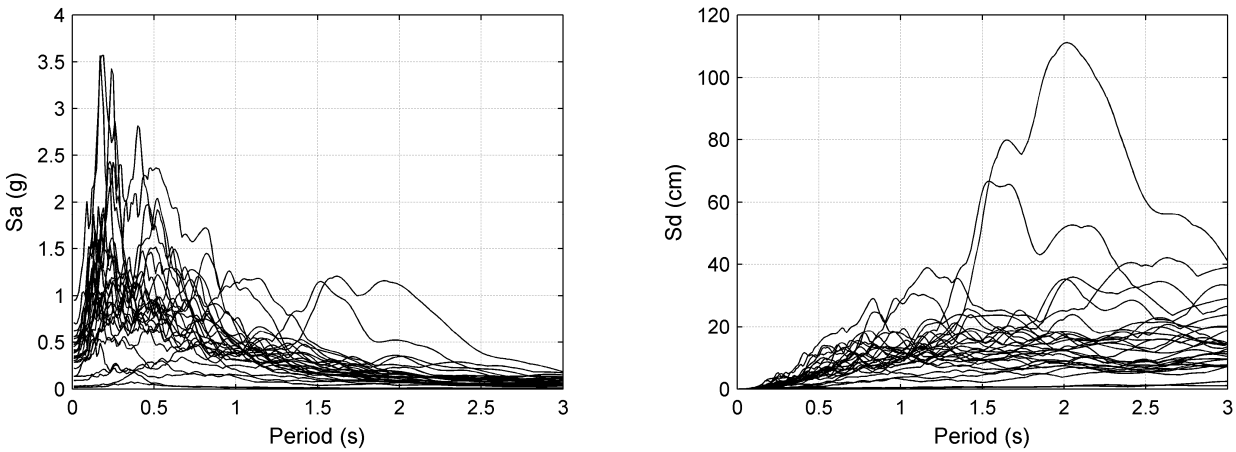
4. Ground Motions

The fragility curves were generated using 28 ground motions, shown in Table 1. These ground motions were obtained from recordings of 14 stations during the 2010 Chilean earthquake.

Table 1 – Selected ground motions

Station	GM Component	Δt	t_{max} (sec)	S_a (T_1, 5%) (g)
Angol	EW	0.01	180.0	0.45
	NS	0.01	180.0	0.30
Concepción Centro	L	0.005	141.7	0.56
	T	0.005	141.7	0.41
Constitución	L	0.005	143.3	0.66
	T	0.005	143.3	1.04
Copiapó	EW	0.01	70.0	0.02
	NS	0.01	70.0	0.02
Curicó	EW	0.01	180.0	0.49
	NS	0.01	180.0	0.38
Hualane	L	0.005	144.1	0.65
	T	0.005	144.1	0.64
Llolleo	L	0.005	124.6	0.37
	T	0.005	124.6	0.75
Maipú	EW	0.01	167.0	0.48
	NS	0.01	167.0	0.54
Matanzas	L	0.005	120.4	0.88
	T	0.005	120.4	0.39
Talca	L	0.005	148.0	0.34
	T	0.005	148.0	0.37
Valdivia	EW	0.01	79.0	0.32
	NS	0.01	79.0	0.23
Valparaíso UTFSM	L	0.005	72.0	0.13
	T	0.005	72.0	0.15
Viña Centro	EW	0.01	125.0	0.71
	NS	0.01	125.0	0.60
Viña Salto	EW	0.005	170.0	1.06
	NS	0.005	170.0	1.02

Since an incremental dynamic analysis is performed to obtain the fragility curves, the ground motions were scaled using the pseudo-acceleration at the building period (T_1=0.9sec). The ground motions where scaled to obtain a pseudo-acceleration value within the range of 0.05g to 1.2g to reflect different levels of intensity measure. Fig. 5 shows the unscaled pseudo acceleration response spectrum for all the ground motions used in this study, and Table 1 shows the ordinates of the spectral acceleration at the building period S_a (T_1, 5%).



(a) (b)

Fig. 5 – Response spectra for selected ground motions considering 5% damping: (a) Pseudo-acceleration response spectrum (b) Displacement response spectrum

5. Fragility Curves

The fragility functions of the prototype building were obtained from the results of an incremental dynamic analysis [8]. The probability of exceedance of each damage state was adjusted to lognormal fragility functions. The methodology used to obtain the fragility functions is explained in the following sections.

5.1 Damage state definitions

In order to build fragility curves, different levels of damage states should be established. In this study, the performance limit states extracted from the HAZUS Earthquake Model Technical Manual [9] for RC shear-walls buildings were selected. HAZUS is a tool developed by FEMA to estimate physical, economic and social impacts of natural disasters based on geographic information. According to HAZUS, four levels of damage are defined: Slight, Moderate, Extensive and Complete damage. These performance levels are defined conforming to types of buildings and buildings materials throughout the inter-story drift, which is assessed during the analysis. In this case, the classification of the inspected building is stated in the table “Model Building Types” (Table 5.1) of the HAZUS manual, where the type C2H for High-rise Concrete Shear Walls buildings for buildings with more than eight floors is selected. From here, the levels of inter-story drift are specified in the table “Structural Fragility Curve Parameters” (Table 5.9 of the HAZUS document). This table is divided in four categories, giving four different threshold values of the inter-story drift according to the seismic design level: High-Code, Moderate-Code, Low-Code and Pre-Code buildings. Following the characterization described in the HAZUS manual, and considering that the sections of the reinforced concrete walls selected for the analysis do not comprise the use of confinement at the wall edges, the Moderate-Code inter-story threshold values are used to define the damage state limits values. Table 2 shows the damage state limits considered in the analyses.

To assess if a certain limit state was achieved in each response history analysis, the maximum inter-story drift of the first floor is computed and compared with the limit values of Table 2. At each time step, the value of the inter-story drift is estimated from the displacement of the first floor divided by the first floor height, which is compared to the threshold inter-story drift values of Table 2. Given this comparison, the performance level or damage state (Slight, Moderate, Extensive, or Complete) can be established. Once one of the damage states is achieved during the ground motion duration, the corresponding intensity measure is saved to build the consequent fragility curve. After the first damage state is reached, the time-history analysis of the current ground motion keeps running until the last performance level is obtained, or the ground motion is finished. It is important to notice that for some earthquake records not all the performance limits are attained.

Table 2 – Inter-story drift threshold values (mm/mm) for C2H using moderate-code seismic design level according to HAZUS manual

Slight	Extensive	Moderate	Complete
0.0020	0.0042	0.0116	0.0300

5.2 Incremental dynamic analysis

The second step to determine the fragility functions is to conduct the incremental dynamic analysis (IDA) using the selected 28 ground motions listed in Table 1. The pseudo-acceleration at the building period was scaled from 0.0 g to 1.2 g with intensity increments of 0.05g. Therefore, a total of 700 time-history analyses were performed in OpenSEES. Values of the first floor and roof displacements were recorded at every time step for all the analyses. Fig. 6 shows the results of the IDA curves illustrating the maximum roof displacement for each intensity measure for the 28 ground motions.

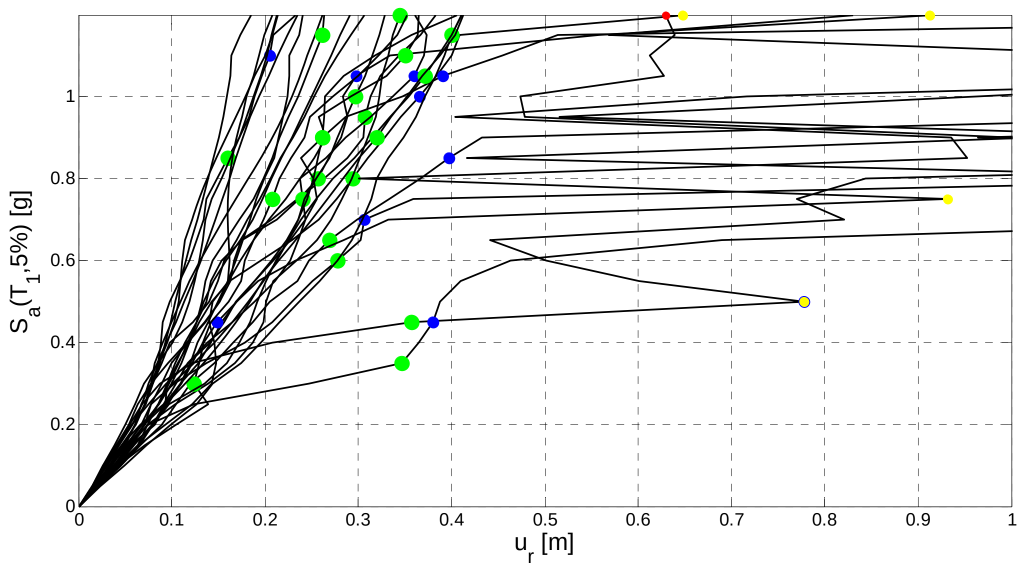


Fig. 6 – Incremental dynamic analysis curves for the roof displacement (u_r)

In Fig. 6, the green dots represent the intensity measure at which the *Slight* limit state is achieved. The blue, yellow, and red dots represent the intensity measure at which *Moderate*, *Extensive*, and *Complete* limits are attained, respectively.

5.3 Building fragility curves

The fragility curves were obtained from the results of the 700 time-history analyses. Since not all damage levels are achieved for each time-history analysis, the truncated incremental dynamic analysis procedure described by Baker [25] is used to construct the fragility curves. This method calibrates a lognormal candidate distribution using the maximum likelihood method to estimate the probability of attaining a certain damage state given the data obtained from the analyses, and provides an estimation of rest of the damage states for higher intensities measures (in this case $S_a(T_1)$).

The obtained fragility curves are shown in Fig. 7 for the four damage states established. The dots indicate the probability of exceeding certain limit states for a given intensity measure and the continuous line are the adjusted fragility functions. The mean and standard deviations of the fragility function associated to each limit states are summarized in Table 3. Fig 7 shows that a 50% probability of exceedance the *Complete* limit state is obtained for an intensity measure of $S_a(T_1)$=1.7g. Additionally, Fig. 7 presents a 50% probability of exceedance of the *Slight* limit state for an intensity measure of $S_a(T_1)$=1.0g. Moreover, for the Concepción ground motion at the L component, the $S_a(T_1)$ value of the considered structure is 0.56g (Table 2) and the corresponding probability of exceedance for *Complete* damage extracted from Fig. 7 is 0.1%. In the city of Concepción two buildings collapsed after the 2010 Chilean earthquake, which is comparable to the 0.1% predicted by Fig. 7, considering that the Chilean buildings have a smaller wall density than that assigned to the building prototype of this paper. A higher value of wall density is correlated with less damage; therefore the predicted 0.1% of probability of exceedance for the *Complete* damage state is found to be reasonable.

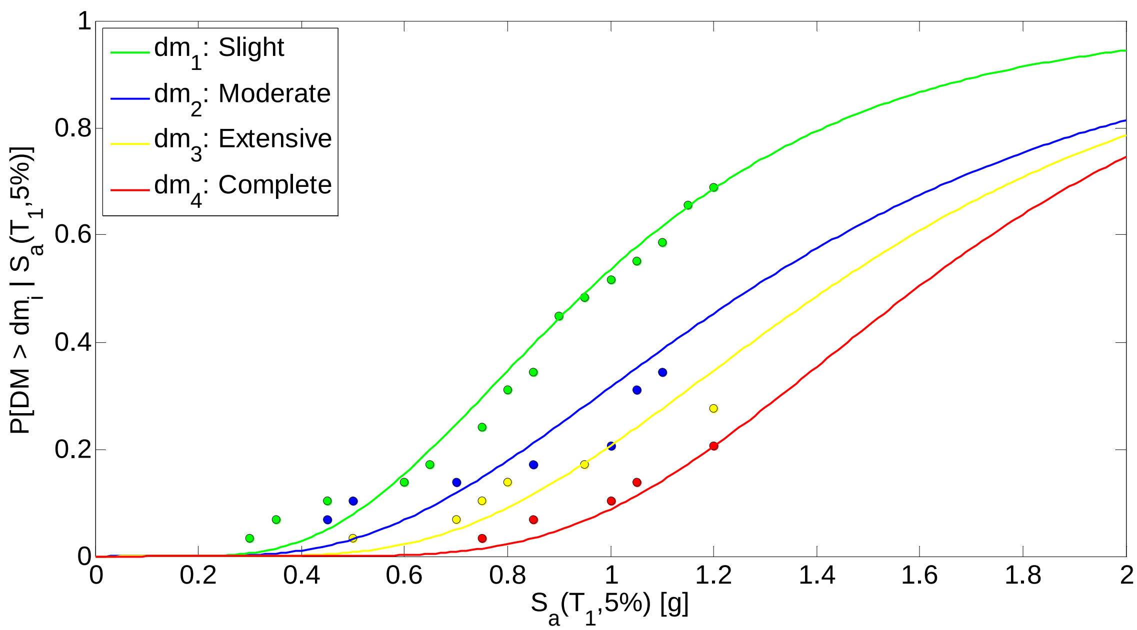


Fig. 7 – Fragility curves developed

Table 3 – Mean and standard deviations of the fragility functions

Limit state	Mean	Standard deviation
Slight	-0.041	0.460
Moderate	0.242	0.505
Extensive	0.352	0.430
Complete	0.466	0.344

6. Concluding remarks

In this paper a simplified procedure for defining fragility curves of RC shear wall buildings is presented. The methodology is based on conducting incremental dynamic analyses to a simplified prototype building that represents a transverse slice of a RC shear-wall building. The performance limit states are defined according to inter-story drift thresholds given by HAZUS technical manual. From the results of the incremental dynamic analysis, the fragility curves associated to four limit states were obtained.

Although the procedure presented is based on a simple representation of a RC shear-wall building, the obtained fragility curves give reasonable estimations of the performance of high-rise shear-wall buildings. Therefore, the proposed methodology may be used in the future to obtain fragility curves of RC shear-wall buildings with different configurations. To obtain representative fragility curves different building heights and configurations should be considered. For example different wall lengths, wall density, wall cross-sections (such as using T-shape walls), wall separations, beams or slabs cross-sections, and reinforcement detailing, among others, may be studied

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