ABSTRACT

Chilean buildings are periodically subjected to earthquakes and their satisfactory performance preventing collapse is widely known. Yet, as a result of the Mw 8.8 2010 Chile earthquake, one reinforced concrete building collapsed and many others were severely damaged, raising concerns about the collapse margin of these structures. Modifications to the Chilean codes were introduced after this earthquake, but Chilean codes are still prescriptive and assessments of the collapse risk of current Chilean buildings is limited. This study evaluates the collapse potential of a code-conforming reinforced concrete office building in Santiago, Chile, whose structural system has a core of two cantilever C-shaped walls surrounded by intermediate moment frames. The architectural layout was designed based on a statistical analysis of the building inventory in Santiago. Incremental dynamic analyses using 45 Chilean earthquakes were performed over a nonlinear model of the building to estimate its collapse fragility, which was combined with a site specific seismic hazard analysis to estimate the mean annual frequency of collapse ($\lambda_c$), and the probability of collapse in 50 years ($P_c(50)$). Results of $\lambda_c$ and $P_c(50)$ were $1.2 \times 10^{-4}$ and $0.6\%$, respectively, and deaggregation of $\lambda_c$ is dominated by small to medium earthquake intensities.
Collapse Assessment of a Chilean Code-Conforming Reinforced Concrete Office Building

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ABSTRACT

Chilean buildings are periodically subjected to earthquakes and their satisfactory performance preventing collapse is widely known. Yet, as a result of the Mw 8.8 2010 Chile earthquake, one reinforced concrete building collapsed and many others were severely damaged, raising concerns about the collapse margin of these structures. Modifications to the Chilean codes were introduced after this earthquake, but Chilean codes are still prescriptive and assessments of the collapse risk of current Chilean buildings is limited. This study evaluates the collapse potential of a code-conforming reinforced concrete office building in Santiago, Chile, whose structural system has a core of two cantilever C-shaped walls surrounded by intermediate moment frames. The architectural layout was designed based on a statistical analysis of the building inventory in Santiago. Incremental dynamic analyses using 45 Chilean earthquakes were performed over a nonlinear model of the building to estimate its collapse fragility, which was combined with a site specific seismic hazard analysis to estimate the mean annual frequency of collapse ($\lambda_c$), and the probability of collapse in 50 years ($P_c(50)$). Results of $\lambda_c$ and $P_c(50)$ were 1.2x10\textsuperscript{-4} and 0.6%, respectively, and deaggregation of $\lambda_c$ is dominated by small to medium earthquake intensities.

Introduction

Estimation and reduction of the collapse potential of building structures is a critical concern in earthquake engineering. Collapse of structures is the principal contributor to injuries and casualties due to earthquakes \cite{1} and also has a significant impact on economic losses and downtime \cite{2}, \cite{3}. In fact, with the introduction of the performance-based earthquake
engineering (PBEE) framework developed by the Pacific Earthquake Engineering Research (PEER) Center [4], collapse assessment is required to estimate the performance of an engineered facility in terms of: (i) injuries and casualties; (ii) direct economic losses; and (iii) downtime. Although most current building seismic design codes are still merely prescriptive and do not include PBEE concepts to estimate the collapse risk explicitly, they have incorporated provisions to avoid collapse during extreme ground motions since design codes are primarily aimed at protecting human life. This collapse prevention has been successfully observed in recent large earthquakes in 2010 and 2011 in New Zealand [5], [6], and in 2010, 2014 and 2015 in Chile [7]–[9], where the number of fatalities due to collapsed buildings was relatively low.

Particularly in Chile, the \( M_w \) 8.8 2010 Chile earthquake affected thousands of tall reinforced concrete building structures [7]. Even though only one modern tall reinforced concrete building, the Alto Río building, completely collapsed [10], 10% of approximately 100 tall reinforced concrete buildings located in Concepción (105 km from the epicenter) resulted severely damaged [11] inquiring the real performance under extreme loading that pre-2010-earthquake code-conforming reinforced concrete buildings present in Chile. Pre-2010-earthquake Chilean code-conforming reinforced concrete buildings were designed following the Chilean code NCh 430 of 2008 “Design of reinforced concrete structures” [12] and the Chilean code NCh 433 of 1996 “Earthquake resistant design of buildings” [13]. The Chilean code NCh 430 of 2008 is strongly based on the Building Code Requirements for Structural Concrete developed by the American Concrete Institute (ACI) [14], but in the Chilean code the requirements for confinement of boundary elements in structural walls were not mandatory and many buildings damaged during the 2010 Chile earthquake presented poor confinement of the structural walls [11]. Regarding the Chilean code NCh 433 of 1996, this is strongly oriented to collapse prevention, stating explicitly that “although presenting damage, building structures should avoid collapse when subjected to exceptionally intensive ground motions”. However, the same code does not define explicitly “exceptionally intensive ground motions” neither defines specific collapse performance objectives such as the target value of probability of collapse of 1% in 50 years according to the ASCE7-10 [15]. The lack of mandatory requirements for confinement of structural walls and the absence of quantitative performance objectives of the Chilean building codes, placed a significant doubt about the collapse potential of pre-2010-earthquake Chilean code-conforming reinforced concrete buildings. To the author’s knowledge, there are not studies estimating collapse fragility curves, mean annual frequencies of collapse \( (\lambda_c) \), and probabilities of collapse in 50 years \( (P_c(50)) \) of pre-2010-earthquake Chilean code-conforming reinforced concrete buildings.

Due to the structural damage, including the collapse of the Alto Río building, of pre-2010-earthquake Chilean code-conforming reinforced concrete buildings, two major modifications, DS 60 [16] and DS 61 [17], were introduced for the seismic design of post-2010-earthquake reinforced concrete structures. The DS 60 requires that the design and construction of the reinforced concrete buildings in Chile should follow the Building Code Requirements for Structural Concrete and Commentary developed by the ACI in 2008 [18]. The DS 61 changed the confinement requirements of shear walls making them closer to the USA standards. However, curvature ductility requirements at the critical section are different than those required in the ACI 318S-2008 code. The DS 61 changed the parameter required for soil classification as well as the quantification of the earthquake demands imposed over the building structures. The real impact of these modifications on the collapse margin of currently designed and constructed Chilean code-conforming reinforced concrete building structures has not been quantified yet.
This study evaluates the collapse potential of a post-2010-earthquake Chilean code-conforming reinforced concrete office building. It is important to define the building occupancy since developers of office buildings are more concerned about collapse than developers of residential buildings due to longer ownership, and because the structural system of office buildings in Chile is usually a dual system of moment resistant frames and shear walls whereas the structural system of residential buildings is based on shear walls. In particular, the objectives of this study are: (i) to design a representative Chilean code-confirming reinforced concrete office building; (ii) to characterize the site-specific seismic hazard for the building location and to select representative subduction ground motions for collapse assessment; (iii) to perform collapse assessment of the building estimating its collapse fragility curve, \( \lambda_c \), and \( P_c(50) \).

**Collapse Assessment**

Common measurements to evaluate the collapse performance of engineered structures are: (i) collapse fragility curves; (ii) \( \lambda_c \), and (iii) \( P_c(50) \). Collapse fragility curves provide the probabilities of collapse of a structure conditioned on specific values of ground motion intensities and this study uses the spectral acceleration conditioned on the fundamental period of vibration of the structure \( S_a(T_1) \) as the ground motion intensity measure. This study implements the method proposed by Ibarra and Krawinkler [19] to estimate the collapse fragility curve, which requires incremental dynamic analyses (IDAs) to obtain the minimum \( S_a(T_1) \) at which each ground motion collapses the structure. For each record, a group of nonlinear response history analyses (NLRHA) at increasing levels of \( S_a(T_1) \) are performed until collapse is reached. The current study focuses on sidesway collapse in the direction of minimal lateral stiffness, with collapse defined as lateral dynamic instability where the building displaces laterally until the lateral resistance of the structural system is overcome due to P-Δ effects that are increased by component cyclic deterioration. This study identifies the minimum value of \( S_a(T_1) \) at which an unbounded increment of the roof drift ratio (i.e., roof displacement divided by the building’s height) is observed from the corresponding NLRHA.

To numerically obtain the collapse fragility curve, the maximum likelihood method is implemented over the \( S_a(T_1) \) values that causes collapse for each record to estimate the parameters of a lognormal distribution function, which has been recommended by previous studies [19]–[21]. The value of \( \lambda_c \) represents the mean rate of collapse per year and combines the collapse fragility curve of the structure and the seismic hazard curve affecting the structure over the entire domain of possible values of \( S_a(T_1) \). Eq. 1 shows the calculation of \( \lambda_c \).

\[
\lambda_c = \int_0^\infty P(C|S_a) \cdot \left| \frac{d\lambda_{S_a}}{dS_a} \right| \cdot dS_a
\]  

where \( P(C|S_a) \) is the probability of collapse of the structure when subjected to a ground motion with a specific value of \( S_a(T_1) \), and \( \left| \frac{d\lambda_{S_a}}{dS_a} \right| \) is the absolute value of the derivative of the seismic hazard curve. The computation of Eq. 1 usually requires numerical integration.

The probability of one collapse in 50 years can be assumed as a Poisson process, and its computation is given in Eq. 2, where all the terms have been defined previously.

\[
P_c(50) = 1 - e^{(-\lambda_c \cdot 50)}
\]
Methodology for Definition and Design of the Code-Conforming Building

Statistical Studies

Prior to the structural design of the building, the characterization of the location, architectural layout, and structural system of the proposed building must be defined. To achieve this objective, a statistical study is performed to represent the current Chilean reinforced concrete office building inventory. Based on the statistical study, three parameters were collected for this testbed office building. First, the district in which this office building is located since this gives information about site characterization and ground motion selection. Second, the number of stories because the dynamic properties change from parameters such as the height of the building and third, the floor area, which influences the selection of the structural typology to be used for this testbed office building. Statistical data about reinforced concrete buildings constructed from 2002 until 2015 in the metropolitan area of Santiago was collected from the Chilean National Statistics Institution (INE) [22]. Fig. 1 shows the histogram of reinforced concrete office buildings constructed in Santiago from 2002 to 2015, considering only projects with five or more stories. Office buildings having fewer than five stories are small in frequency, significantly scattered around Santiago, and consequently they are not considered in this study.

![Figure 1. Histogram of reinforced concrete office buildings constructed from 2002 until 2015.](image)

As the districts of Providencia, Huechuraba, Las Condes, Vitacura and Santiago concentrated the office building projects, a specific analysis of the number of stories and floor areas was developed for these districts and its results are shown in Fig. 2.

![Figure 2. (a) Number of stories distribution by district showing percentiles 10, 25, 50, 75 and 90, and (b) floor area by district, showing percentiles 10, 25, 50, 75 and 90.](image)

Code-Conforming Building Design and Modeling

A reinforced concrete office building, designed according to current Chilean standards, was considered for this study. The structural configuration was based on an actual office building located in the district of Las Condes, Santiago. The structure has sixteen stories and three
basement levels. Following the typical design of office buildings in Chile, post-tensioned slabs (20 cm typical thickness, 22 cm in basement levels) were considered to obtain longer spans without interior beams. The force-resisting system comprises two core C-shaped walls for gravity and lateral loading and moment frames for gravity loading, located in the building perimeter. Fig. 3(a) presents the plan view of the typical story. The software ETABS [23] was used for analysis and design of this building. Dead load of 200 kg/m² and live load of 300 kg/m² (basement levels), 500 kg/m² (typical story) or 200 kg/m² (roof), were considered according to Chilean standard NCh 1537 of 2009 [24]. Determination of seismic loads and analysis followed the Chilean standard NCh 433 of 1996 modified in 2009 [13] and DS 61 [17]. Seismic analysis parameters were obtained for seismic zone 2 (effective acceleration 0.3g) and soil type B (fractured rock, dense soil Vs30 ≥500 m/s), both corresponding to Las Condes, Santiago. The building was classified as category II (importance factor I=1) and analyzed with a response modification factor R=7. The nominal material properties are compressive strength in cylindrical specimen f’c=35 MPa (called H40 in Chile) for concrete, and nominal tensile strength fu=630 MPa and nominal yielding strength fy=420 MPa for steel. Reinforced concrete design followed the ACI 318-08 [18] building code and Chilean standard DS 60 [16]. In continuous frame-wall buildings where walls resist more than 75% of the base shear, DS 60 allows to design the frame as an ACI 318 intermediate moment frame. This building meets this requirement and therefore details corresponding to intermediate moment frames were developed and shown in Fig. 3(b)-(d).

Analysis showed that walls required special boundary element confinement in two stories above and below the grade level. Ordinary boundary element confinement was provided in other stories. Table 1 presents the main dynamic analysis results obtained using the software Perform 3D [25] and Fig. 4 the reduced design elastic spectrum in both analysis directions.

**Table 1. Dynamic analysis results.**

<table>
<thead>
<tr>
<th>Mode</th>
<th>T (s)</th>
<th>Mnx(%)</th>
<th>Mny(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.020</td>
<td>44.00</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>1.569</td>
<td>0.00</td>
<td>2.74*</td>
</tr>
<tr>
<td>3</td>
<td>0.825</td>
<td>0.00</td>
<td>45.37</td>
</tr>
</tbody>
</table>

*PERFORM 3D does not provide effective mass participation for rotational modes
The software Perform 3D [25] was used for nonlinear analysis of the building. For wall modeling, Perform 3D employs the General Wall Element or the Shear Wall Element. The first element is intended for the analysis of complex reinforced concrete walls with irregular openings, which is not the case of this building, where both core walls are continuous over the building height. The second model is preferred for slender walls without openings where the important actions are shear force and axial-bending action in the horizontal cross section. This model is used herein. The horizontal cross section is discretized into concrete and reinforcing steel fibers, each one with a uniaxial constitutive relation that follows a single YURLX backbone curve. Gravity loads are distributed by tributary areas in vertical elements and for seismic loading, translational and rotational masses are assigned at each level in the corresponding center of mass, linked to every other node at that level by rigid diaphragm constraints. Beams and columns are divided into three segments for modeling purposes. The segments at the ends correspond to the expected plastic hinge region, and they are modeled using nonlinear fibers sections. Eq. 3, from Paulay and Priestley [26], is used to calculate the plastic hinge length, $L_p$.

$$L_p = 0.08L + 0.022d_bf_y$$

where $L$ is the distance to the inflection point in a beam or column, $d_b$ is the diameter of the longitudinal bar, both in mm, and $f_y$ is the yielding strength of longitudinal bars, in MPa. The middle segment is considered linear elastic. For steel, expected material properties ($f_u=800$ MPa and $f_y=480$ MPa) are considered for the YURLX backbone curve, which is an inelastic steel model without buckling. For concrete, the stress strain relation proposed by Saatcioglu and Razvi [27] is used. Confined concrete properties are considered only in fibers located within the special boundary elements, located at the walls stems at the first and second levels, above and below the grade. Every other concrete fiber is modeled as unconfined. Inelastic in-plane shear behavior in walls was modeled with an elastic-perfectly plastic stress-strain curve. Reinforced concrete structures are prone to experience strain localization caused by strain softening behavior [28]. This makes the structural response to depend on the model discretization. Regularization of concrete materials (confined and unconfined) is needed to obtain an objective structural response. In walls, the regularization approach introduced by Kashani et al. [29] is used. This is a constant compressive fracture energy approach applied to walls modeled with “Shear Wall Elements” in Perform 3D. According to this approach, since there is no localization of damage prior to achieving maximum strength, standard models may be employed to define the pre-peak portion of the concrete constitutive relation. To define the post-peak response, unconfined concrete is assumed to have no residual strength and confined concrete is assumed to have
residual strength equal to 20% of the confined compressive strength at a specified residual strain. Fig. 5(a) presents the regularized YURLX envelope used for confined and unconfined concrete in shear structural walls. Concrete material in frame elements is also regularized following recommendations given by Coleman and Spacone [28]. In frame elements, Perform 3D considers only one integration point per element segment. Therefore, the length value used to regularize this concrete material corresponds to the length of the segment where the plastic hinge is expected to form. Fig. 5(b) presents the regularized YURLX envelopes used for unconfined concrete in beams and columns.

Site, Seismic Hazard Analysis and Ground Motion Selection

Site Characterization

The soil conditions in Santiago consist primarily of stiff gravels and alluvial deposits from the Maipo and Mapocho rivers. Outcropping rocks are common within the basin (i.e. San Cristobal Hill, Renca Hill), and at the selected districts, the depth to bedrock varies between 200-500 m. In this area, the shear wave velocity derived from ambient noise and seismic refraction are approximately 420-720 m/s in the top 20 m, 1200-1500 m/s at depths up to 200 m, and 2000 m/s up to bedrock [30], [31]. The testbed building is located in the business district of Las Condes (-33.4124°S;70.5635°W) on a soil class B (very dense or stiff soil).

Seismic Hazard

The rate of exceedance of $S_a(T_1)$ values was estimated through a standard PSHA implemented in in the code SeismicHazard [32]. Although the risk of building collapse is largely controlled by subduction megathrust events, a simplified model of shallow crustal faults in Central Chile is included for the sake of completeness. In this study, the geometry of the interface/in slab sources and the magnitude recurrence models were obtained from Martin [34], while the shallow crustal faults were based on the geometry and seismicity reported by Leyton et al. [35]. The ground motion intensity for subduction earthquakes was determined using the ground motion prediction equations (GMPE) proposed by previous studies [36]–[38]. Epistemic uncertainty for subduction GMPEs was used in the analysis by assigning equal weights to each model. The resulting hazard curves are presented in Fig. 6(a).

Figure 5. YURLX envelopes for regularized concrete material in compression for use in Perform 3D for (a) shear walls, and (b) frame elements.
**Ground Motion Selection**

The ground motions used to perform the IDAs consists of 45 records from eight interface subduction earthquakes that occurred along the boundary between the Nazca and South American plates. The selected records correspond to earthquakes with moment magnitude (Mw) between 6.3 and 8.8, recorded at rupture distances up to 500 km, on outcropping rock or firm soil sites with Vs30>300 m/s, and with PGA greater than 0.15 g. Frequencies outside the range 0.1-25 Hz were filtered out from the records, and for each hazard level, the records were scaled to match the target S_a value. The 5% damped response spectra of the records scaled to 0.1g at the fundamental period of vibration of the testbed building (T=2.02 s) are shown in Fig. 6(b).

![Image](image_url)

**Figure 6.** (a) Seismic hazard curves for S_a(T_1) values, and (b) 5% damping response spectra of selected ground motions and geometric mean.

**Collapse Assessment Results**

Fig. 7(a) shows the collapse S_a(T_1) intensities from the 45 ground motions along with the proposed lognormal fragility function for the testbed building. The median collapse intensity of the building is 0.64 g and the logarithmic standard deviation of 0.38. This dispersion only includes record to record variability since the analyzes did not include modeling uncertainty. Using Eq. 1, the value of λ_c was estimated reaching 1.2x10^{-4}, which is a small value and consistent with previous estimations of λ_c developed for reinforced moment frame buildings located in seismic areas [39]. Using Eq. 2 to estimate P_c(50), this value is 0.60%, which is below the target of 1% probability of collapse in 50 years proposed by ASCE7-10 [15]. Both estimated values, λ_c and P_c(50), show that the performance of the post-2010-earthquake Chilean code-conforming reinforced concrete office building considered in this study is satisfactory.

Fig. 7(b) presents the deaggregation of λ_c for testbed building and it can be observed that the contribution to the value of λ_c is dominated by small to medium S_a(T_1) intensities. In particular, 70% of the value of λ_c is reached by S_a(T_1)=0.63 g. This deaggregation evidences that since most of the value of λ_c comes from small to medium S_a(T_1) intensities, an accurate representation of the fragility curve is very important for S_a(T_1) intensities below the median collapse capacity as suggested by Eads et al. [21].
Conclusions

This paper presents the design of a testbed post-2010-earthquake Chilean code-conforming reinforced concrete office building and its collapse assessment in terms of the estimation of the collapse fragility curve, $\lambda_c$ and $P_c(50)$. The testbed building has sixteen stories and three basement levels and its architectural layout was designed based on a statistical study of the buildings’ inventory of similar structures in Santiago. The building was designed for the district of Las Condes, Chile, according to the current Chilean seismic codes, IDAs over a set of 45 representative Chilean earthquakes were performed and a site specific seismic hazard curve for the testbed building was developed. Results of $\lambda_c$ and $P_c(50)$ were $1.2 \times 10^{-4}$ and 0.6%, respectively, and the estimated value of $P_c(50)$ is smaller than the target recommendation of ASCE 7-10. Moreover, the deaggregation of $\lambda_c$ is dominated by small to medium earthquake intensities and, therefore, an accurate representation of the fragility curve is very important for $S_a(T_1)$ intensities below the median collapse capacity. The results presented in this paper show that reinforced concrete office buildings that conform to the post-2010 earthquake code requirements in Chile would perform adequately, indicated by a collapse probability that is smaller than what is currently recommended by ASCE 7-1.

References


