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SIMPLIFIED DESIGN SEISMIC-RESISTANT METHOD FOR LOW-RISE LIGHT-FRAMED SHEAR WALL BUILDINGS

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Abstract

In this research work a prescriptive methodology for preliminary design of light-frame shear wall buildings up to 6 floors against seismic loads is proposed. The research is based on the analysis of structural models of archetypes oriented to residential buildings with eccentricity controlled by a suitable selection of strength and stiffness with respect to the structural axes. Through pseudo 3-D analysis of structural models of archetypes, and by identifying their dynamic behavior it is possible to define simplified models that allow the realization of parametric studies in relatively short time, which makes it attractive to use fragility curves for making decisions regarding design parameters. In order to achieve optimal performance for various seismic records, structural axes of variable resistance in height were considered. The drift was selected as a principal response parameter of the models, which should not exceed certain values associated with several performance levels. In addition, the variation of resistance in height was established trying to achieve a uniform drift response of the structure. In this way, all resistant lines actively participate in the energy dissipation. A proportional resistance for each level was obtained with respect to the first level response, and therefore proportional to the weight of the building. An important result of this work is that it is possible to suggest a simple way to design the system with seismic design coefficients, as an alternative to using the R-factor that reduces the elastic spectrum. The proposed methodology also incorporates control over the relative displacements between stories. Each seismic coefficient has been analyzed to meet performance targets proposed for timber structures, a condition that is verified in this study using fragility curves.

Keywords: seismic response, low-rise, wood, light-frame, shear wall, buildings, fragility curves.

1. Introduction

It is known from experience that timber buildings have good performance against earthquakes when they are design under proven requirements. Although this performance has been satisfactory in considering a life safety criterion, it may not be sufficient to limit damage in structural and non-structural elements. Today, the height of timber buildings in seismic zones is limited to approximately four stories in many countries. This is mainly due to the lack of understanding of the dynamic response of higher structures, but also due to non-structural limitations such as fire and considerations for non-structural damage. In particular, in Chile there is little knowledge about the seismic response, and both the seismic and timber design codes do not consider procedures for light-framed building with shear walls and diaphragms. Having Chile significant forest resources, there is a positive projection in the utilization of timber for low and middle-rise buildings in the near future, as long as the understanding of seismic resistant timber systems is improved and the building codes are modified to include light-framed shear walls buildings. In the last decade important efforts have been made to develop seismic design concepts based on timber structural performance that have only been applied to steel and reinforced concrete buildings. Nowadays there are displacement-based design procedures, but they still show complexity in their application, that result in resistance to adoption in traditional engineer offices. It can be said that the requirements of many design codes for timber structures are not yet based on a well comprehended seismic design philosophy. This means that designers are may not understand the mechanism of deformation and the load paths of timber structures during a seismic event. This combination of factors have limited the use of timber structures for construction of mid-rise buildings and hence have reduced the economic competitiveness of the timber industries relative to the steel and concrete.

In countries like Japan, United States, Canada and European countries among others, the study of timber as a construction material is very advanced and it is used in construction as a competitive alternative to other materials. For design purposes, t is commonly used a static analysis with equivalent lateral forces that follow the first mode shape, with satisfactory results (Breyer et al., 2007). However, before adopting any foreign design methodology, several issues should be addressed, such as mechanical behavior of Chilean structural timber used for shear walls and diaphragms, local manpower and technology, types of earthquake, etc.

The main objective of this research is to develop a prescriptive methodology for seismic design for light-framed shear wall buildings up to 6 floors in height. Shear walls stiffened by OSB panels, on one or both sides, were considered. The structural performance of different archetypes of residential buildings was evaluated. The archetypes do not have vertical nor horizontal irregularities, and eccentricity effects are controlled by a suitable selection of the strength and stiffness for the orthogonal structural axes. Through pseudo 3-D analysis these archetypes, and by identifying their dynamic behavior it is possible to define simplified models that allow the realization of parametric studies in relatively short time, which makes it attractive to use fragility curves for making decisions regarding design parameters. The expected outcome of this research work is to provide a simple tool to conduct the seismic design of timber buildings in areas of high seismicity, while ensuring a satisfactory performance objective. This methodology can be considered as complementary to the current and future design codes.

An interesting characteristic of the proposed methodology is that the seismic response modification factor (R-factor), that reduces the elastic spectrum, is not required to be defined. A seismic coefficient, defined later in this article as the ratio between the strength of the first floor and the seismic weight of the building, is selected at the beginning of the design, which at the same time defines the distribution of strength in height. Then, the archetype must be verified in order to meet performance targets proposed for timber structures, a condition that is verified in this study by using fragility curves. In this way, the method also incorporates control over the relative displacement between floors. Since a large number of seismic records with different magnitude, intensities and types of soil were considered, there would be no need to use a spectrum design for defined from a specific building code.

Through the iterative analyses of the archetypes, it was found that the structural resistance for each of the upper levels can be proportioned in relation to the first floor resistance, providing an acceptable performance for buildings up to 6 floors. The modeling also insures that all lines of resistance for each floor level actively participate in the energy dissipation.

2. Design according to Chilean Code NCh 433 for Seismic Design

The design requirements using the Equivalent Lateral Force Procedure (ELFP) explained in the Chilean Seismic Design of Buildings Code, NCh433.Of96 modified in 2009 (INN, 2009), considers forces lower than those expected at a major earthquake, which forces the structure to respond in the nonlinear range. Therefore, the structure should be designed to have sufficient ductility to remain structurally safe when forced to into the inelastic range during a major earthquake. Currently the Chilean Code uses a response modification factor (R-factor) of 5.5 if the ELFP is used, but this factor does not have a clear basis on how it was established in Chilean legislation.

The method used to calculate the horizontal forces at floor level consists of three steps. First, the base shear acting on the structure is calculated. A percentage of this base shear is then redistributed through the different levels. Finally, the forces on the individual elements are determined (panels, diaphragms, beams, etc.) which are generated as a result of the seismic forces from each floor level above the level being evaluated. The Chilean code only controls the level of displacement for reduced seismic loads. It does not require calculating the inelastic response, i.e., lateral displacements are calculated based on design response spectrum not reduced by the R-factor, with the exception of reinforced concrete buildings where the code was modified after the 2010 Maule's . The code does not provide any tool to estimate the nonlinear displacements and deformation capacity of timber buildings. In fact, the drift limit given in the code for reduced seismic loads, 2%, applies to all structural systems regardless of the material. This value is known to be too low for timber structures, in particular for light-framed shear wall buildings, which can undergo larger drifts without significant damage.

3. Criteria for evaluating seismic performance

In order to evaluate the seismic performance of timber buildings and define the α parameter, for light-framed shear wall buildings the following drift limits are proposed:

- For frequent earthquakes, drift should be less than 0.5% to 1%.
- For design level earthquakes, drift should be less than 2%.
- For maximum considered earthquakes, drift should be less than 3%.
- The collapse condition will be reached at a drift equal 4%.

These limits must be approved by an expert committee, because values that have been established in this work are based upon the drift limits used in other international codes. The lower drift ratios proposed for frequent and design earthquakes are intended to limit the economic losses produced, mainly because of the relatively high frequency of major earthquakes in Chile.

Following the ATC-63 methodology, Dechent (Dechent et al, 2014) suggested an R-factor equal to 6.5 for light-framed shear wall buildings up to five floors, which provides a base shear coefficient higher than 0.1. However, in order to propose a prescriptive design method for a certain type of timber building, where structural regularity is mandatory, it is believed that a better way to evaluate seismic performance can be achieved through fragility curves.

4. Constructive System and Modeling Capacity

The timber construction system based wood light frames is often used in North America, Europe, Japan (among other countries) in residential construction of low and medium height. The seismic-resistant system consists of shear walls and horizontal diaphragms made of structural plywood or OSB panels. The general configuration of a structural wall with a reinforcement panel consists of the following components:

- Double studs at the ends of the wall.
- Intermediated studs spaced at a predetermined distance (less than 610 mm).
- Single or double sill and top plates.
- Sheathing panels of the same or different thickness (plywood or OSB).
- A pattern of nails or screws at the panel edges and interior, at a predetermined distance.
- Special anchors made of tie-down brackets and anchor bolts.

It is worth noting that the lateral load is mostly resisted by the sheathing panel. Folz and Filiatrault (2001) have shown through mathematical models and tests that the contribution of studs to the lateral capacity is negligible.

In buildings with shear walls based on panels, which correspond to the analyzed structures, the following load path occurs: lateral load (seismic) acts on the horizontal diaphragm of each floor, where much of the mass is concentrated. The diaphragm transfers this load to the shear walls as a distributed traction along the top plate at the connection zone. The force is then transmitted from the top plates to the sheathing panel through the connection with nails or screws. Finally, the loads are transferred to the bottom plate, then to the anchors and foundations by shear, tensile, and compression (Breyer et al., 2007).

Wood-frame shear walls are the essential component of the lateral-force-resisting system on light-frame shear wall buildings. A large number of pushover and cyclic experiments on specimens of timber shear walls have been conducted showing that the force-displacement response at the top is highly nonlinear (Salenikovich 2000, Gatto and Uang, 2001; Pardoen et al., 2003). Several numerical models have been developed in the last two decades, being the one proposed by Folz and Filiatrault (Folz and Filiatrault, 2001) one of the most widespread and internationally accepted. This model incorporates the hysteretic behavior of wood shear walls.

In the seismic design of wood light frame buildings, shear walls are anchored to foundations and to each other on the upper floors to prevent overturning and ensure a desired mode of deformation, where the shear wall acts as a mechanism which deforms as a parallelogram to the extent that the top of the wall is moved relative to the bottom. The numerical model must be able to predict the load-displacement response at the top of the wall induced by static or dynamic loading.

4. Resistance Distribution Definition for an Optimum Performance

The distribution of resistance for the structure for optimal horizontal resistance performance for each floor is estimated so that the structure does not experience a concentration of drift at one level. In other words, it is assumed that a good seismic performance is obtained when all shear walls in the building are actively involved in resisting the seismic forces and provide energy dissipation. After conducting nonlinear dynamic analyses of simplified models subjected to several seismic records, a resistance distribution in height is is proposed in this work (Dechent et al, 2014). Although this study has yet to be extended to a wider range of seismic records that account for various types of soil, the results show that acceptable performance can be achieved. Analyses have been performed on a plane concentrated mass model capable only of shear deformations, varying the resistance per floor until reaching an average response close to the target displacements considered as optimal. It is believed that in low-rise buildings, the component for shear deformation will dominate over flexure, therefore, the results should not change considerably if flexural deformation mechanisms are incorporated. The Sapwood program has been used for performing multiple nonlinear dynamic analyses such as time-history analysis (Pei and van de Lindt, 2007). Since the objective of this work is to achieve a prescriptive design, the condition that structures are regular and have a low eccentricity has been imposed. This should be expected in structures well designed from the structural point of view, and even in structures that show certain levels of eccentricity, but with a proper balance of strength and stiffness along the different structural axes.

A lumped mass model was considered for the model, which was defined with equal mass on each floor except the highest floor, which had only half the mass of the floors assigned. This condition can be based on the existence of a lightweight finish concrete on all floors except the roof. This is a construction requirement that has been imposed in recent years in Chile. This finish concrete represents about 50% of the weight of the building. With the plan dimensions of the building known, an estimate of the mass of the floors using construction-based experience was made. Then the lateral resistance for each floor was assigned according to the parameter α (shown in Equation 1), which is defined as the shear capacity of the first level wall system, relative to the seismic weight of the building.

$$\alpha = \frac{Q_y}{W} \tag{1}$$

Where, Q_y corresponds to the shear capacity at the base of the structure and W is the seismic weight, estimated by Equation (2):

$$W = (m + m + ... + \frac{m}{2})g \tag{2}$$

The resistance of the first level can be determined once α is defined. Then, the resistance of the upper floors relative to the bottom floor resistance is varied until a condition of uniform drifts at all floors for the different records analysed is achieved. These analyses have led to a preliminary proposal of the distribution of resistance over the height of the building, which is shown in Figure 1.

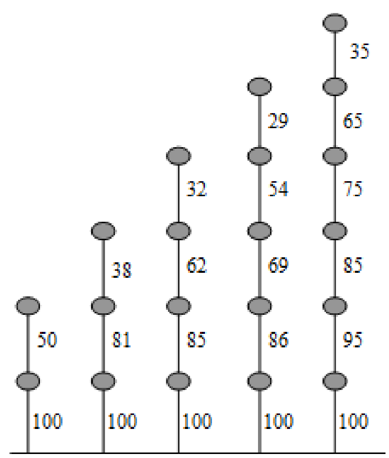

Fig. 1 – Proposed distribution (%) of floor resistance, for buildings from two to six floors in height.

Capacity values analyzed in the models were: α = 0.10, 0.20, 0.30, 0.40, 0.50, 0.70, 1.0. Once the resistance to be provided to the building is selected, and knowing design values of shear strength per unit length of wood shear walls (similar to Table 2306.3.1. International Building Code, for example), the designer can determine a suitable combination of wall lengths that fit the architectural floor plan for the project.

5. Selection of Seismic Records for Archetype Evaluation

In this preliminary phase of the research, 53 seismic records were selected to perform the nonlinear analyses of the archetypes, all of them obtained on soil type C according to the classification table of Article 6 of the

Supreme Decree 61 (2011) of the NCh 433 Chilean seismic code. The records represented subduction earthquakes of varying magnitudes. This type of soil was chosen because observations of historic earthquakes above 7.5 Mw in Chile resulted in damage to buildings built on these soils. Later phases of the research will include similar analyses using seismic records recorded on other types of soils. In Figure 2 and Figure 3 the fragility curves of a six-story building are presented, for earthquakes above 7.5 Mw and below 5.5 Mw, respectively. These curves show the effect of the magnitude of ground motion, even when the records are artificially scaled to similar PGA. The records to be considered for evaluating the archetypes will correspond to earthquakes with Mw>7.5, which should lead to conservative results for small values of PGA.

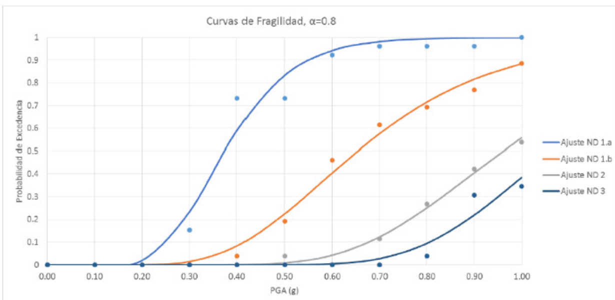

Fig. 2 – Fragility Curves for Mw≥7.5

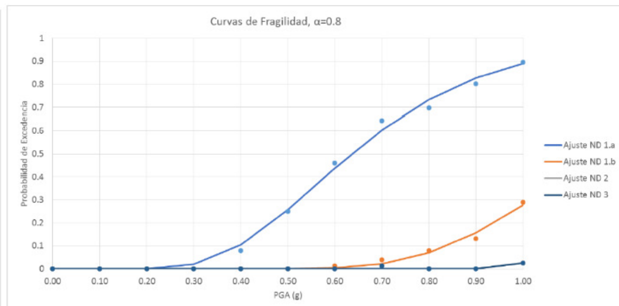

Fig. 3 – Fragility Curves for Mw≤5.5

6. Drift and Hysteresis Results

Representative results of the nonlinear dynamic analysis are presented for a four-story archetype subjected to the Llolleo 1985´s seismic record. Figure 4 shows the drift distribution for a model with uniform resistance of the floors, while Figure 5 shows the same results for a model with optimized resistance of upper the floors. From Figure 4, it can be observed that the first floor presents the highest drift, which means that the upper floors walls have less deformation. Therefore, the upper walls do not contribute to energy dissipation as much as they could, which is more significant for lower capacity of the models. On the other hand, from Figure 5 it can be observed that the drift is relatively uniform in height, avoiding a drift concentration at the firs level. In this way, an optimum seismic performance is obtained according to the criteria defined at the beginning of this article.

In order to explain the improvement of the shear wall behavior when changing from a model with uniform floor resistance to an optimized floor resistance distribution, the hysteretic curves of the first and fourth floors are presented, for a four-story model with α=0.2, subjected to Llolleo 1985´s seismic record. The response hysteresis of the original model, with uniform distribution of resistance in height is shown in Figure 6. A considerable difference of behavior can be noticed between the walls of the first and fourth levels, which reveals that the upper walls almost do not contribute to energy dissipation. In contrast, the response hysteresis of the model with optimized distribution of resistance in height is shown in Figure 7. In this case, the hysteresis response of the first floor presents a little reduction compared to the original model, while the hysteresis response of the fourth floor gets stretched, which indicates and increase in energy dissipation at the upper floor.

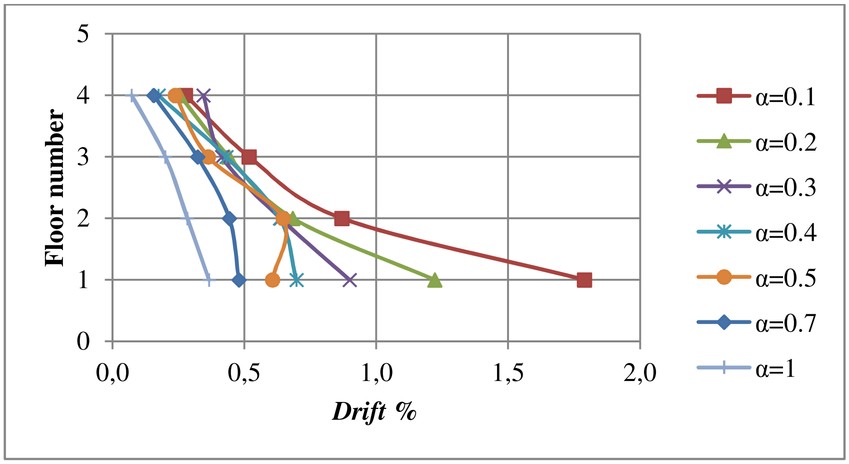

Fig. 4 – Distribution of maximum drift for a building with uniform shear capacity in height.

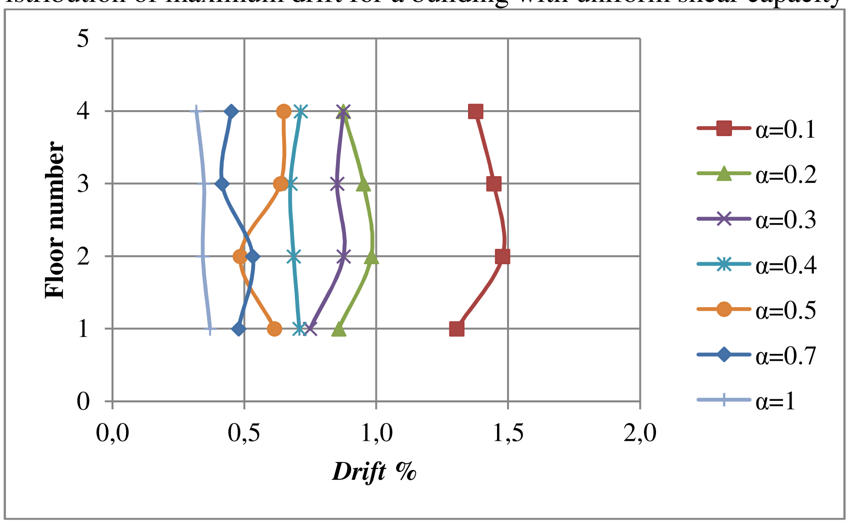

Fig. 5 – Distribution of maximum drift for a building with optimized shear capacity in height.

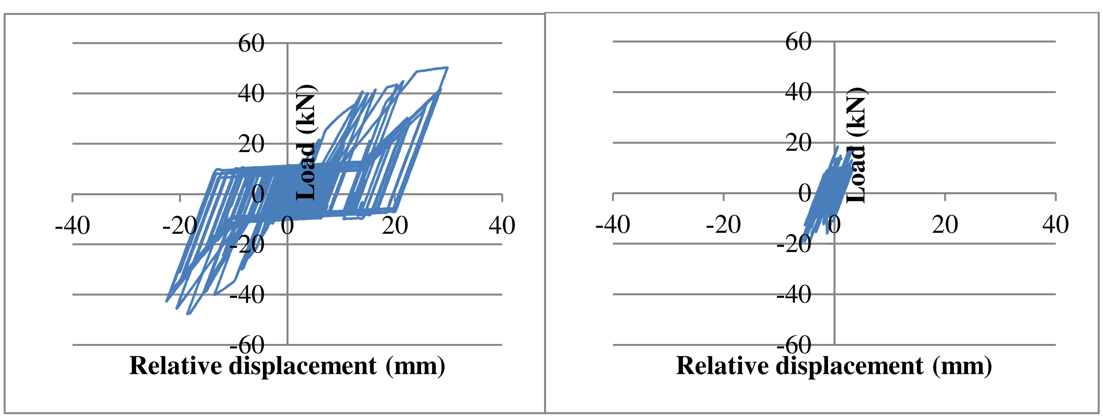

Fig. 6 – Response hysteresis of the model with uniform resistance in height, α=0.2. First floor at left and fourth floor at right.

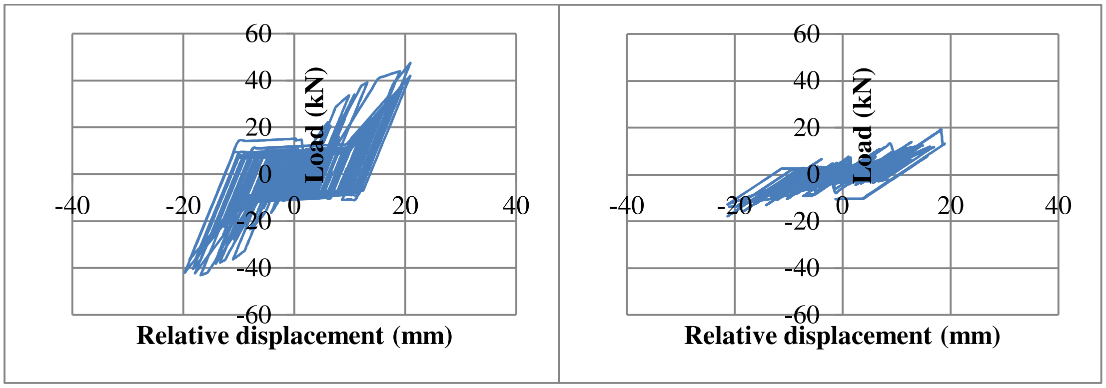

Fig. 7 – Response hysteresis of the model with optimized resistance in height, α=0.2. First floor at left and fourth floor at right.

For a four-story archetype with the optimized resistance distribution proposed in Figure 1, the parameter α was varied from 0.1 to 1 (as shown in Figures 4 and 5), obtaining seven different models. Then, each model was subjected to 17 seismic records, ending up with 119 nonlinear dynamic analyses. A drift larger than 4% at any floor was obtained only for three models with α=0.1, for five seismic records. This result suggests that building with such small shear capacity (10% or less of the seismic weight) should be avoided, since the collapse condition may be reached. Nevertheless, for design purposes, this condition is controlled by the limitation given in the Chilean Code NCh433Of.96 of a minimum seismic coefficient of 8%, which would be equivalent to have a structure with a shear capacity of approximately 24%

7. Application of the Methodology to an Archetype

The architecture plan for the building to be designed using the proposed prescriptive method is shown in Figure 6. This is a typical archetype for social housing in some regions of Chile. All walls with an aspect ratio less than 2 are selected as structural walls. Three resistance levels, defined for values of α of 0.3, 0.5 and 0.7, were considered for the building models. For each value of α, the required resistance of the first level is determined using the seismic weight, and then the strength distribution scheme proposed in Figure 1 is used to define the resistance of the upper floors. Therefore, all the analyzed models had the same resistance distribution in height, which is expected to provide a relatively uniform drift at all floors. Then, the incremental nonlinear dynamic analyses for several seismic records was conducted for the three different models.

The fact that for design there will be variations between the actual resistance and the suggested values due to the physical configurations of the walls, should not have a significant impact on the results, because of the inertial effects that absorb certain differences in the results and do not strongly affect the concept presented in this article. The resistance for each floor is distributed to the corresponding walls with the aid of resistance tables for shear walls, where stiffness and strength can be estimated for different nailing patterns and sheathing thickness.

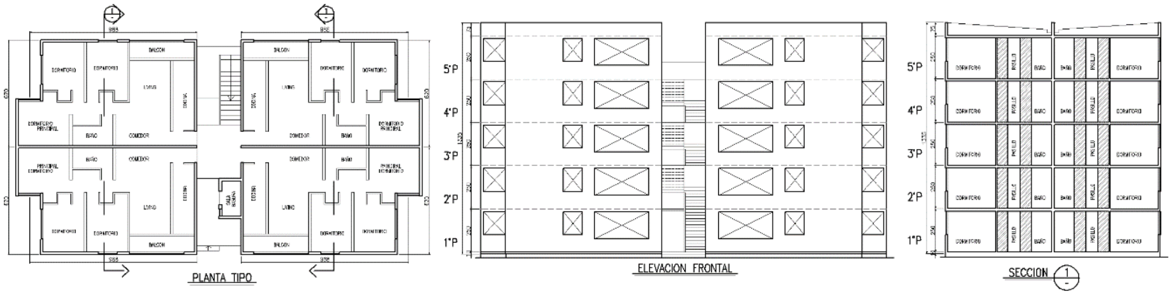

Fig. 8 – Architecture plan of a typical residential building in Chile.

Drift vs. PGA curves and fragility curves obtained from the incremental nonlinear dynamic analysis are presented in Figures 9, 10 and 11, for the three levels of resistance (α=0.3, 0.5 and 0.7). Fragility curves were developed for drift levels of 0.5%, 1%, 2% and 3%. These fragility curves were developed for earthquake records with magnitudes greater than 7.5. This results in conservative curves for low PGA event due to the minor energy level that such records hold, even when they artificially achieve these levels of PGA.

From the drift vs. PGA curves, it can be observed that as the resistance (given by the α-factor) of the building is increased, the drift is reduced for a larger number of records. It was noticed that there are some records that produce extremely large values of drift when certain level of acceleration is exceeded. These records should be analyzed in detail in order to determine whether they should be eliminated from the study due to special features that are outside of the considered group of ground motions. It is also demonstrated through the fragility curves that as the factor α is increased, the brittleness of the structure is reduced.

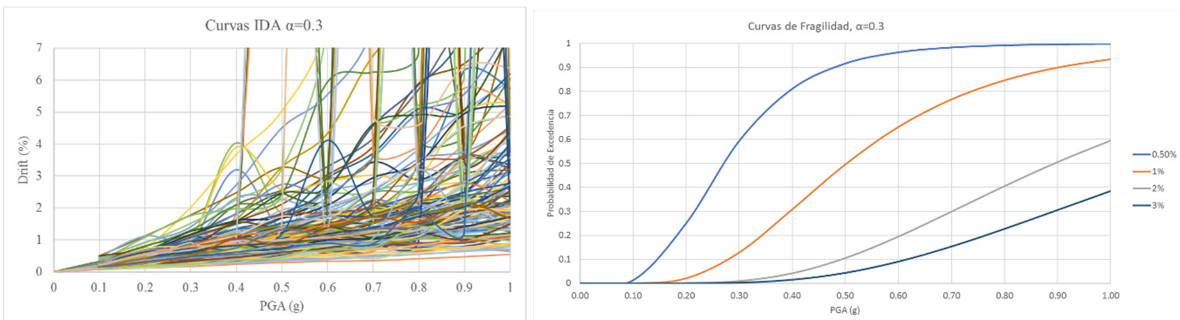

Fig. 9 – (a) Drift vs. PGA for α=0.3 (b) Fragility curves (Cumulative frequency vs. PGA)

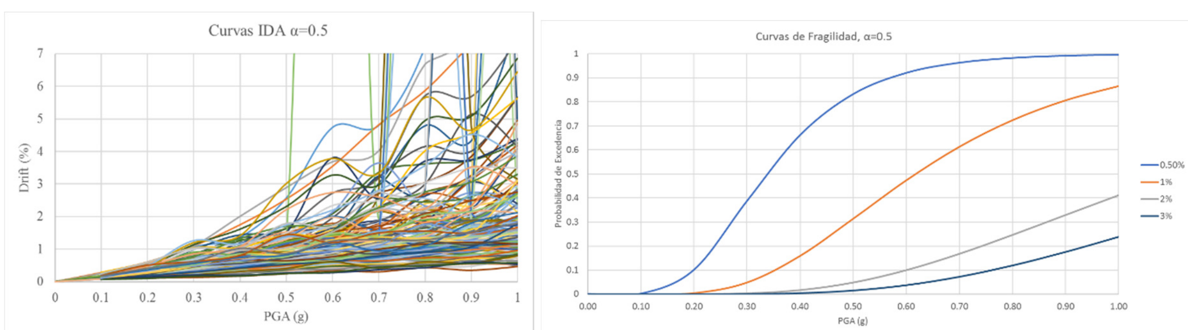

Fig. 10 – (a) Drift vs. PGA for α=0.5 (b) Fragility curves (Cumulative frequency vs. PGA)

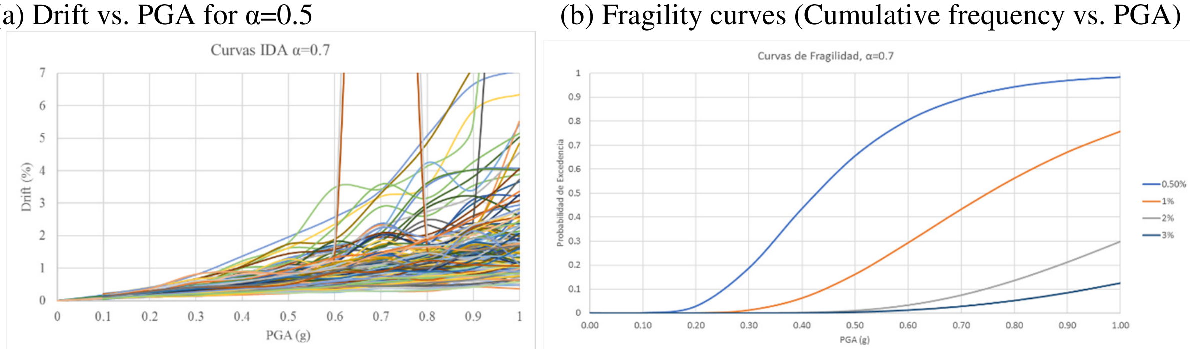

Fig. 11 – (a) Drift vs. PGA for α=0.3 (b) Fragility curves (Cumulative frequency vs. PGA)

With this design methodology, the proposed seismic performance limits defined in Section 3 are satisfied. This design method could now be considered for adoption into the standard NCh433. The designers can complete the design of buildings in a simple and straight forward manner. The design methodology should be verified for irregularity cases and other soil types.

8. Conclusions

A preliminary distribution of resistance over the height of the building was proposed for residential light-framed shear wall buildings of up to six stories. In this way, once the resistance to be provided by the building is selected, and knowing design values of shear strength per unit length of wood shear walls, the designer will be able to easily determine a suitable combination of wall lengths that fit the architectural floor plan for the project. This methodology could lead to a prescriptive design method for seismic loading, in the meantime it is valid only for regular structures with controlled eccentricity.

For a four-story building, it was verified that using this optimized distribution of resistance over height, rather than a uniform resistance distribution, leads to relatively uniform drift at all levels, avoiding drift concentrations at the first floor. Through comparison of hysteresis curves of shear walls of the first and fourth floors, it was also verified that using the optimized distribution will help to dissipate more energy at upper levels.

The initial results of this research, that is, drift vs. PGA curves and fragility curves obtained from incremental nonlinear dynamic analysis of a building archetype subjected to several seismic records of earthquakes with Mw larger than 7.5, suggest that providing a shear resistance at the first floor of about 30% of the seismic weight of the building, may be sufficient to achieve a good performance from a safety and serviceability point of view for most of the possible earthquakes. In order to provide this minimum resistance, a seismic design coefficient of about 0.1 to 0.15 would be needed.

8. Acknowledgements

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