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SEISMIC DESIGN FACTORS FOR WOOD-FRAME BUILDINGS IN CHILE

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Abstract

The Chilean timber industry has a significant presence in both the economic and labor sector throughout the country. Local pollution issues and the seismicity of the region make timber a sustainable material choice for constructing damage-tolerant buildings. However, as the Chilean seismic regulations have been developed mainly for reinforced concrete structures, they are quite demanding for mid-rise timber structures. This leads to buildings with a high density of walls, short periods, and low ductility demands, and hence eliminating the main advantages of the wood frame system compared to other building materials. In order to address this issue, this paper reports the main findings of a collaborative research project carried out by researchers of the Pontificia Universidad Católica de Chile and the University of Technology Sydney. The main research goals could be summarized as: (1) validating the current seismic regulations regarding timber buildings, and (2) proving the feasibility of using less conservative guidelines when designing wood-frame structures.

For this purpose, a holistic approach was adopted for this project at three different research stages. First, the behavior of materials, connections, and wall assemblies were examined under different load conditions in order to get information on mechanical properties, force-deformation behavior, and nonlinear response of the structural elements. Second, non-linear models were developed for connections, walls, and buildings, aiming at expanding the research capabilities beyond the lab results. And third, the seismic performance of 201 wood-frame buildings was evaluated through static and three-dimensional dynamic analyses, taking into account different seismic design requirements. Results showed that the current Chilean design factors (R = 5.5 and Δ_{max} = 0.002) lead to resilient buildings with very good seismic behavior, while less conservative factors (R = 6.5 and Δ_{max} = 0.004) also result in code-compliant structures that fulfill the primary objective of safeguarding life and preventing building collapse. Furthermore, the benefit-cost ratio increases, leading to buildings which are economically attractive and competitive compared to other structural systems.

Keywords: wood-frame structures, mid-rise buildings, regulations, seismic design factors, FEMA P-695 ground motions.

1. Introduction

Seismic design factors are valuable in designing modern earthquake-resistant structures. They provide a first approach to estimate strength and displacement demands on structural systems that are being designed using elastic methods but that are expected to behave nonlinearity under large deformations. Therefore, they represent a simple and useful tool for practitioners and researchers when designing structural systems subjected to essential lateral seismic loads. Even though the current state-of-the-art has provided several design factors for different applications [1,2], the most widely used factors in local and international standards are the R factor and the maximum allowable drift Δ_{max}.

The design philosophy behind the seismic design factors relies on the nonlinear deformation capacity of buildings. The R factor reduces the lateral design force calculated from a design spectrum based on the ductility of the structural system, allowing the building to suffer damage during strong earthquakes but assuring that a life-threatening state will not be reached. Therefore, the R factor determines the relative acceleration level that the structure is required to resist. On the other hand, the maximum allowable drift Δ_{max} limits the lateral deformation and inter-story drift, minimizing the structural and non-structural damage during earthquakes of high exceedance probability. The calibration of the values adopted for such design factors is usually based on detailing and expected performance. However, for newly defined or undefined systems, they are typically determined employing engineering criteria and qualitative comparisons to achieve equivalent or similar behavior with the factors of previously code defined systems. Although a good performance may be achieved for the buildings designed using these factors even during strong earthquakes, the lack of a robust and rational calibration methodology may lead to very conservative designs that are not economically viable within the real estate market.

Nowadays, to design mid- to high-rise buildings in Chile, the current normative stipulates that the requirements of the national standard for seismic design of buildings NCh 433 [3] must be met. For timber construction, the standard establishes an R factor equal to 5.5, and a maximum allowable drift Δ_{max} of 0.002h. This latter refers to the relative displacement of the center of mass of two consecutive stories and was adopted to guarantee the resilience of stiff systems such as reinforced concrete structures. However, this limit may be difficult to achieve by flexible systems like timber, resulting in very rigid timber structures with short periods and low ductility demand, wasting the inherent advantages of timber construction. The suitability of the Chilean seismic design factors regarding timber construction can be analyzed by contrasting them with those defined in international standards. As noted by Dolan et al. [4], the American ASCE 7-16 [5] standard defines an R factor equal to 6.5 for light-frame shear walls with wood structural panels. If the differences in the seismic demand (i.e., seismic design spectra) between both countries are not taken into account, the lower R factor in the NCh433 standard results in timber buildings being designed for accelerations 18% higher than an equivalent structure designed under the USA requirements. If the different design spectra are considered, the accelerations can be up to 2.75-3.0 times higher [4]. It is also of relevant interest to assess how other materials are considered with respect to timber in both standards. For instance, the R factors for reinforced concrete and reinforced masonry in the ASCE 7-16 [5] standard are 2 and 4, while in the NCh433 [3] standard are 4 and 7, respectively. This gives timber construction a market advantage in the USA, while reinforced concrete has a market advantage in Chile. Furthermore, it can be noted that concrete and masonry are required to resist 30% and 225% higher accelerations than timber in the USA, while they are required to resist 21% lower and 25% higher accelerations than timber in Chile, respectively. As noted by Dolan et al. [4], these differences illustrate a bias towards more familiar and better-known materials, highlighting the need for a rational quantification of the seismic factors in standards and codes.

For the maximum allowable drift Δ_{max}, the ASCE 7-16 requirement is 0.025h for all structures 4 stories and less, meaning that the Chilean code requires timber buildings to be over 10 times as stiff as in the USA. Therefore, the lateral design will be controlled by drift requirements that are difficult to meet in areas prone to high seismic accelerations [4]. On the other hand, the 0.002h Chilean requirement means an inelastic drift limit equal to 4.8 mm for a 2.4 m high wall, and, as noted by McMullin and Merrick [6], the damage of gypsum panels (i.e., non-structural elements) occurs at a drift of ~6 mm, meaning that the 0.002h limit requires the buildings to remain elastic. The elastic drift limit can be estimated as $\Delta_E = \Delta_{max}/C_d$, where C_d is the deflection amplification factor. Assuming the concept of "equal displacement" proposed by Newmark [7], the C_d factor is equal to the *R* factor. Thus, $\Delta_E = \Delta_{max}/C_d = 0.002h/5.5 = 0.00036h$. For a regular 2.4 m high wall, the elastic drift limit is $\Delta_E = 0.9$ mm, a low drift located in the initial elastic region of the force-displacement response [4]. These small deflections expected for a Δ_{max} of 0.002h result in overdesigning wood-frame walls; hence, the full potentials of timber construction may not harnessed as have been reported in previous research [6,7]. In the Chilean context, Santa María et al. [8] reported large cross-sections and sturdy walls while designing a six-story wood-frame building according to current Chilean seismic regulations. Recently, Cárcamo et al. [9] highlighted the need to incorporate rigid structures, such as concrete or CLT walls, in wood frame buildings in order to meet the maximum drift requirements. Aiming at optimizing the design of timber structures, Dechent et al. [10] developed a simplified design methodology for low-rise wood frame buildings which does not require the *R* factor when calculating the lateral design forces. The methodology employs a simplified model that allows carrying out parametric analyses of the design parameters employing fragility functions. Additionally, Dechent et al. [10] highlighted that the basis of the proposed seismic design factors in the current Chilean seismic standard is not quite clear.

Since a robust seismic design standard is crucial for the development and growth of the timber construction in Chile, this paper presents the results of a comprehensive research project aimed at quantifying the seismic design factors for wood-frame buildings through a rational approach. Following the guidelines of the FEMA P-695 methodology [1], the investigation embraces (1) testing of materials, connections, and wall assemblies, (2) calibration and nonlinear numerical modeling of wood frame wall elements under monotonic and cyclic loads, and (3) seismic performance evaluation of a comprehensive set of wood frame buildings across the country through conducting several 3D simulations. This project was a collaborative effort of researchers from the University of Technology Sydney and the Pontifical Catholic University of Chile, and the results aim at providing new guidelines towards efficient seismic design of wood frame buildings in Chile.

2. Experimental program

As shown in Fig. 1, wood frame walls consist of: (1) a 1200 mm to 2400 mm timber frame made up of 38×135 mm studs spaced at 400 mm on center, (2) 9 to 15 mm thick oriented strand board (OSB) panels on one or both sides of the wall to provide lateral capacity, (3) sheathing-to-framing S2F connections (i.e., steel nails) to attach the panels to the frame, and (4) steel hold-downs to prevent overturning of the wall. Therefore, wood frame walls are not monolithic structures and their mechanical behavior depends on the individual properties of each of their components.

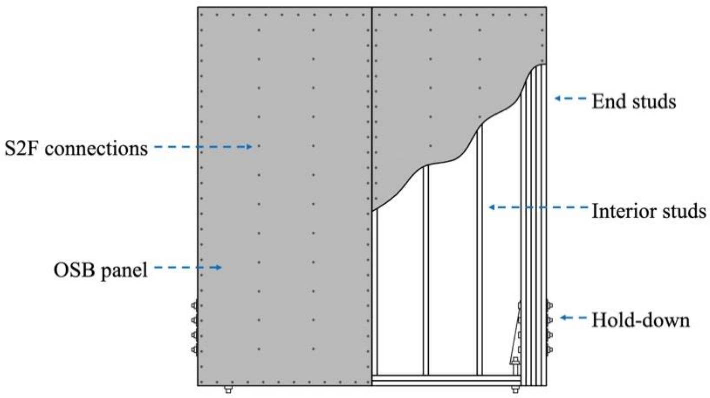


Fig. 1 – Typical configuration of wood frame walls.

In order to gain a deep understanding of the wood frame system at all levels, an exhaustive experimental program was conducted first. The behavior of materials, connections, and assemblies, were characterized to validate the analytical and numerical tools employed in the subsequent phases of the research as described in the following sections.

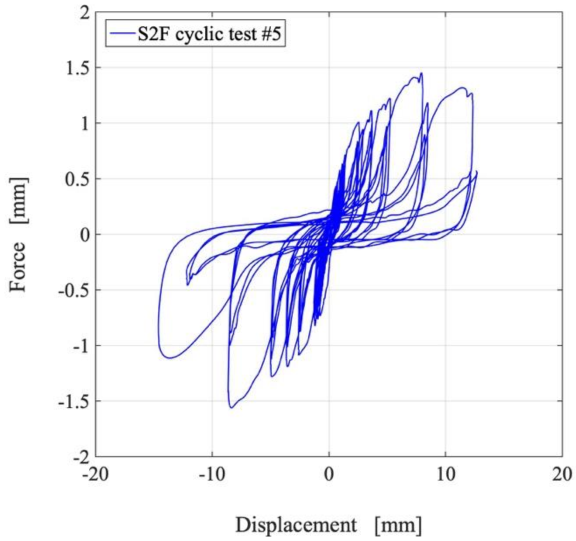
2.1 Material testing

The two main materials that are used in constructing a wood-frame wall were tested in the first phase of the project. Timber studs and OSB panels were tested aiming at determining the mechanical properties and behavior of the local wooden products. Stud specimens consisted of Chilean MGP10 radiate pine. Forty-five studs were tested under bending, tensile, and compression quasi-static load in the facilities of the INFOR Structural Wood Laboratory, Concepción, Chile, following the guidelines of the Chilean testing standard NCh3028/1 [11]. Additionally, twenty 11.1 mm thick OSB panels (radiate pine strands) were tested according to the ASTM D2719 standard [12] in the facilities of the Engineered American Wood Association in Tacoma, WA, USA, to determine their shear modulus in both the longitudinal and transverse directions. Results showed that the mechanical properties of the local products are consistent with the international standards and previous investigations [13–15], proving the suitability of their use for structural engineering purposes. Documentation of the testing program and further details of experimental results can be found in [16].

2.2 Connection testing

The shear response of the sheathing-to-framing (S2F) connections was examined in the second phase. The specimens consisted of double shear OSB-stud joints pneumatically driven with 70 mm long spiral nails (shank diameter of 3.0 mm). The test setup is shown in Fig. 2(a). Two monotonic tests were carried out in both directions (parallel and perpendicular to framing grain) until the failure of the connections, and the yielding displacement measured in the load-slip response was then employed to compute the CUREe-Caltech reversed testing protocol [17]. Subsequently, 30 cyclic tests were conducted employing three different OSB thicknesses: 9.5 mm, 11.1 mm, and 15.1 mm. Typical failure mechanisms were observed during the tests, such as yielding and fatigue failure of nails, nails being pulled out, and crushing of the OSB panel. Results showed consistency with previous investigations in terms of capacity, stiffness, ductility, and energy dissipation [14,18–20]. A detailed report of the testing program and results can be found in [21].

(a)

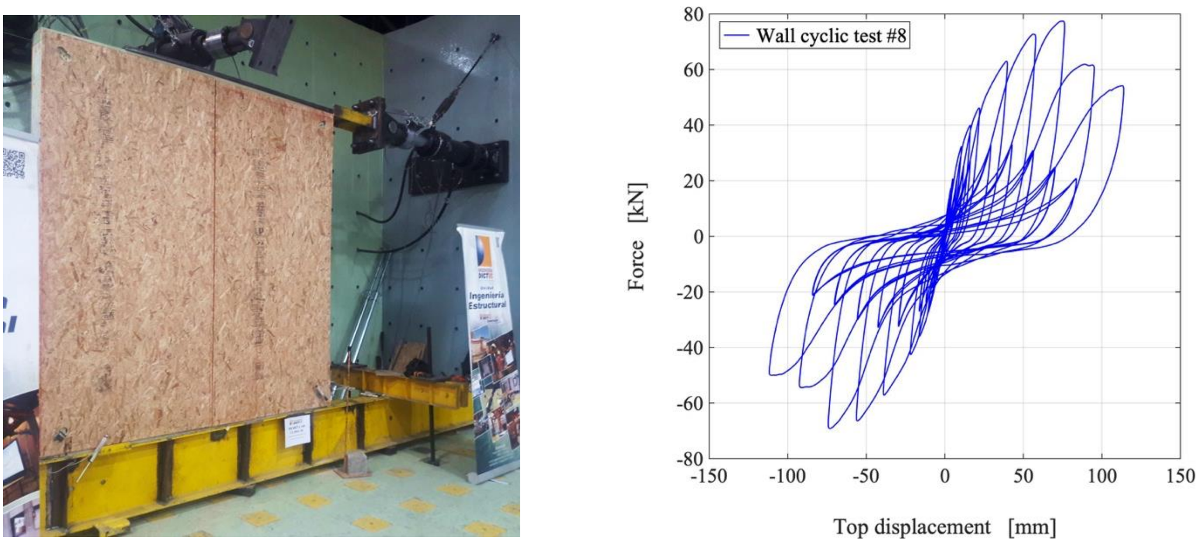


(b)

Fig. 2 – Connection testing: (a) test setup [21], (b) cyclic response of specimen #5 with 11.1 mm thick OSB.

2.3 Wood-frame walls testing

As the final step of the experimental program, nineteen full-scale wood frame walls were tested under monotonic and cyclic quasi-static lateral load. The specimens were 2470 mm high, and four different lengths were studied: 700 mm, 1200 mm, 2400 mm, and 3600 mm. The timber frame was 38×135 mm (2×6") dimensional lumber, with the studs spaced at 407 mm on center. OSB panels (11.1 mm thick) were installed on both sides of the wall using 70 mm long spiral nails with a shank diameter of 3.0 mm, and spaced at 50 or 100 mm in the wall edges and at 200 mm along the interior studs. To avoid sliding of the wall, φ1×10" shear bolts were installed through the bottom plate. SIMPSON Strong-Tie HD12 hold-down anchors were installed at the lower corners employing four φ1×10" horizontal bolts and one φ1-1/8×10" vertical bolt. Monotonic tests consisted of four 1200 mm and three 2400 mm length walls. Cyclic tests were two 700 mm, four 1200 mm, four 2400 mm, and two 3600 mm length specimens. The monotonic tests and the guidelines of the ASTM E2126 standard [22] were employed to compute the CUREE-Caltech protocol [17] for the cyclic tests. A detailed report of the experimental program can be found in Guíñez et al. [23].



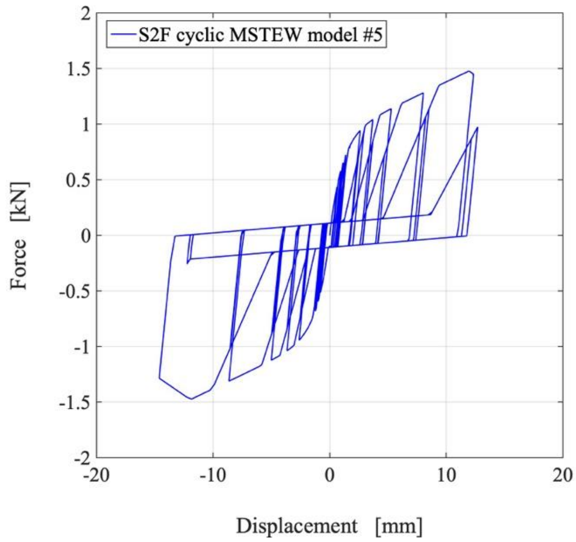
(a) (b)

Fig. 3 – Wall testing: (a) test setup for a 2400 mm length wall, (b) cyclic response of specimen #8: 2400 mm length wall with a nail spacing of 100 mm.

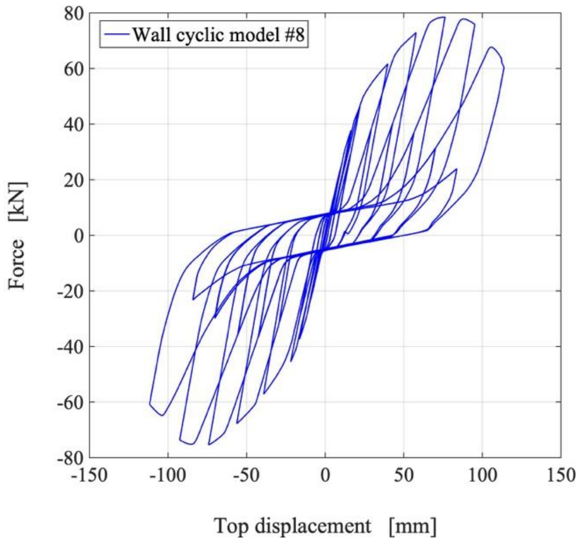
Results showed that the damage was mainly concentrated in the S2F connections at the failure point of the walls. Nails were cut, pulled out, and sheathing panels crushed. However, the timber frame remained mainly undamaged. Stiffness and capacity were found to depend on the wall length and nail spacing. Whilst the cyclic loading path resulted in a decrease of 8% to 16% of shear capacity when compared to the monotonic tests, no differences in the initial wall stiffness were found between both loading paths. On the other hand, it was found that the shear capacity computed with the SDPWS guidelines [13] underestimates the strength of the walls, while the stiffness is overestimated in most of the specimens. This highlights the need for further investigations on the design expression for wood frame walls in mid-rise multi-story buildings. A detailed discussion of the results can be found in [23].

3. Numerical modeling

Numerical models were developed for both S2F connections and wall assemblies aiming at providing a tool to better understand the lateral response of wood frame structures under large displacements and nonlinear behavior. Three different element types were employed when developing the wall model: (1) 6-DOF planar-frame (beam) elements for the timber studs, (2) 5-DOF shear-panel elements for the OSB panels, and (3) 3-DOF link elements for sheathing-to-framing connections and hold-down anchors. The timber frame, sheathing members and hold-down devices were assumed to be linear-elastic, whereas the sheathing-to-framing connections were modeled using nonlinear hysteretic springs. This approach increases the model efficiency without compromising the accuracy of the predictions, since the nonlinear response of a wood frame wall is mainly controlled by the force-displacement properties of the sheathing-to-framing connections. The nonlinear springs employed the Modified-Stewart (MSTEW) hysteretic model proposed by Folz and Filiatrault [14] as shown in Fig. 4(a), whose parameters were calibrated based on the test results described in Section 2.2. Fig. 4(b) shows the model prediction for a 2400 mm length wall (specimen #8 shown in Fig. 3). A detailed description of the model formulation and further analyses of the results can be found in [24].



(a)



(b)

Fig. 4 – Modeling results: (a) MSTEW prediction for the S2F connection test #5 with 11.1 mm thick OSB panel, (b) model prediction for wall speimen #8, a 2400 mm length wall with a nail spacing of 100 mm.

4. Ground motion selection

For the rational quantification of seismic performance factors, the guidelines provided by the FEMA P-695 methodology were followed [1]. As it will be explained in Section 5, the quantification procedure is carried out through statistically evaluating the results of a series of nonlinear dynamic analyses (virtual tests). Such analyses will need a dynamic input in the form of ground motions obtained from previous recorded earthquakes. Although FEMA P-695 methodology provides two ground motion sets (i.e. a far-field and a near-field set) for dynamic analysis, those sets are from shallow crustal earthquakes which are representative of the western areas of the United States. Therefore, they do not include strong motion records from Central and Eastern United States earthquakes, or from deep subduction earthquakes such as those expected in Japan, New Zealand, or in areas on the Pacific Coast of South America. This represents an important limitation in the scope of the research if the methodology is intended to be applied in areas prone to subduction earthquakes. Employing the two provided sets would lead to non-conservative results since subduction earthquakes are known to have longer durations than shallow crustal ones, release more energy, and induce more damage, even to buildings designed with modern codes. Therefore, to be consistent with the subduction seismic hazard that threats Chile, a new set of ground motions aimed at extending the application of the FEMA P-695 methodology to zones prone to subduction earthquakes was developed as part of this research project.

Twenty-six pairs of ground motions were selected from reports published by the seismological agencies of different countries. The criteria applied during the selection process were fully consistent with the guidelines provided by the FEMA P-695 methodology [1]. The ground motions were chosen such that 2/3 were subduction records and 1/3 crustal ones. Hence, the set includes 18 records from subduction earthquakes and 8 from shallow crustal earthquakes, of which 3 are from thrust faults and 5 from strike-slip faults. Fig. 5 shows the response spectra for the 26 pairs of ground motions, as well as the average spectrum of the set plus one and two standard deviations. A period of approximately 30 years has been covered in the set, with data recorded from 1987 until the recent earthquake in Kaikoura (New Zealand) in 2016. Earthquake magnitudes range from M = 6.5 for the Superstition Hills (USA) strike-slip earthquake, to M = 9.0 for the Japanese earthquake of Tohoku in 2011. The average magnitude of the set is M = 7.8. The mean significant duration $Ds_{5\text{-}75}$ [25] of the set is 14.56 seconds. Additionally, for proper consideration of the effects of the spectral shape when computing seismic collapse capacities, the spectral shape factors [1] for this new ground motion set were calculated employing the ground motion prediction equation proposed by Contreras and Boroschek [26] for different fundamental periods and ductility capacities. Detailed information about the selection, processing, and validation of the set can be found in [27].

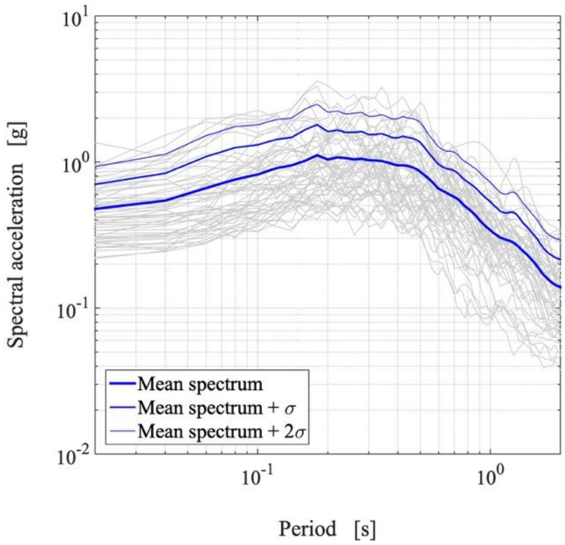
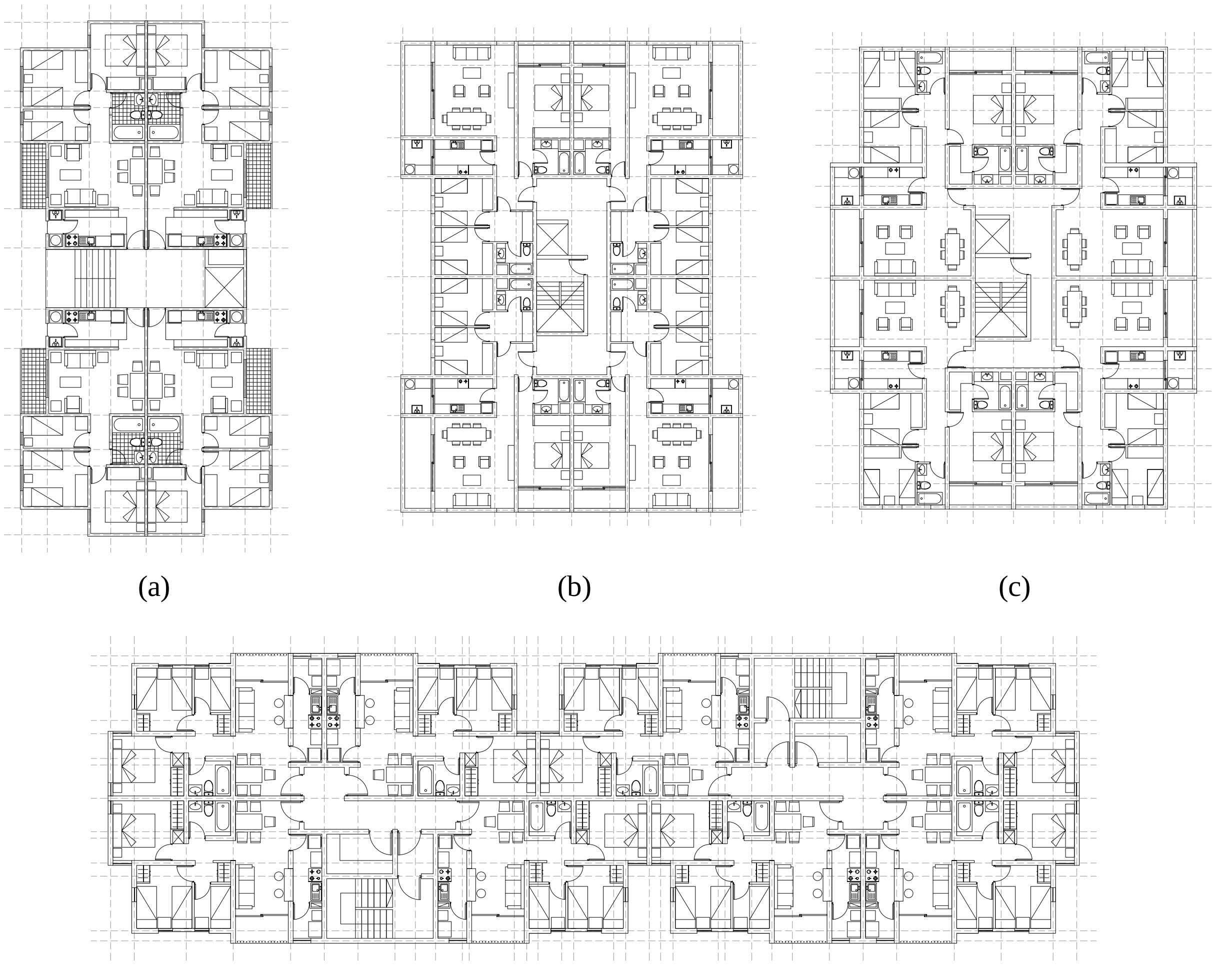


Fig. 5 – Response spectra for the 26 pairs of ground motions and average spectrum of the set plus one and two standard deviations.

5. Analysis of seismic design factors and preliminary results

According to the guidelines of the FEMA P-695 methodology [1], the quantification of a new set of seismic design factors is carried out by means of nonlinear dynamic analyses following a trial-and-error approach. First, a first trial set of seismic design factors (R factor and Δ_{max}) is defined. Second, a set of structural archetypes (i.e., buildings) is developed and designed employing the defined set of seismic design factors. Third, nonlinear incremental dynamic analyses IDA [28] are conducted on the previously defined set of archetypes. Fourth, collapse capacities are calculated and adjusted for each archetype in the set. In the last step, results are analyzed statistically based on collapse probabilities. If the results are satisfactory in terms of safety margin ratios, the new set of seismic design factors is validated. Otherwise, a new set should be defined and the procedure repeated until satisfactory results are obtained.

Two sets of seismic design factors were selected for this research: (i) R = 5.5 and Δ_{max} = 0.002h, to validate the values in the current NCh433 standard first; (ii) R = 6.5 and Δ_{max} = 0.004h, to verify a new set of less conservative factors. These two values were carefully selected after conducting several parametric analyses on critical archetypes to determine the most optimal factors. Subsequently, a comprehensive set of structural archetypes was developed aiming at embracing all the possible applications of the wood frame wall system in Chile. After conducting exhaustive research on the most common floor plans for reinforced concrete and masonry buildings across the country, four architectural designs (named “C”, “D”, “P”, and “Q”, respectively) were developed to take into account the local constructive customs [29]. In the process, different space distributions, asymmetries, maximum spans, perimeters, and discontinuities were covered. The floor plans of each archetype are shown in Fig. 6. In addition to the architectural configurations, different locations throughout Chile were considered to place the archetypes. Following the guidelines provided by the NCh433 standard [3], the buildings were placed in Seismic Zone I or III, and on soil type A, B, C, or D. Furthermore, four heights were considered for the buildings: three, four, five, and six stories. Linear methods were employed to carry out the structural designs as suggested by the NCh433 guidelines [3], employing the SDPWS standard [13] to estimate the mechanical properties of the wood frame walls and the NCh1198 standard [30] to design components and connections. After the permutations between the different variables considered, and eliminating those designs that were not feasible due to structural limitations, the resulting archetype set was made up of 201 buildings: 89 for the current seismic design factors, and 112 for the new ones.



(a) (b) (c)

(d)

Fig. 6 – Floor plan and walls distribution of: (a) building “C”, (b) building “P”, (c) building “Q”, (d) building “D”.

A 3D nonlinear model was developed for each archetype in the set. As proposed by Pei and van de Lindt [31], wood frame walls were modeled using nonlinear spring elements which connect two consecutive floors. The hysteretic behavior of each wood frame wall was modeled using the MSTEW model proposed by Folz and Filiatrault [14], which is able to properly capture phenomena associated with great damage states, such as force degradation, stiffness degradation, and pinching. The modeling approach presented in Section 3 was employed to determine the MSTEW parameters for wood frame walls of different configurations. The vertical flexibility of the buildings was modeled using a simplified bi-linear model which represents the combined stiffness of hold-downs, shear wall studs, continuous steel rods, and any special fastener devices. To compute the collapse capacity of each building, bidirectional IDA analyses were conducted employing the software SAPWood V2.0 [32] employing the ground motion set discussed in Section 4. The ground motions were scaled up until one-half of the records in the set caused the collapse of the archetype, defined as the occurrence of a 3% inter-story drift on any floor [33]. The ground motion pairs of the set were applied twice to each archetype, once with the ground motion records oriented along the principal direction, and then again with the records rotated 90 degrees.

An evaluation of the seismic performance of each archetype was conducted based on the IDA results. For a better understanding of this procedure, an example is presented below. Fig. 7 shows the IDA results for a five-story archetype, floor plan "Q", R = 5.5, Δ_{max} = 0.002h, seismic zone, and soil type B. First, the mean collapse capacity S_{CT} of the archetype, defined as the ground motion intensity which causes collapse (3% interstory drift) to one-half of the dynamic analyses was calculated from the IDA curves, S_{CT} = 0.74 g. Subsequently, the collapse margin ratio CMR was calculated as the ratio of the mean collapse capacity to the 5%-damped spectral acceleration of the maximum considered earthquake MCE ground motion defined in the design standard, i.e., CMR = S_{CT} / S_{MT} = 0.74 / 0.23 = 3.22. Then, the CMR was adjusted to consider the effects of the spectral shapes of the ground motions [27] and employing 3D models [1], ACMR = CMR×SSF×1.2 = 3.22×1.27×1.2 = 4.91. Finally, the collapse probability of the archetype was evaluated based on the ACMR and the inherent uncertainty of the process in two different phases, according to the FEMA P695 methodology [1]. For the first phase, the ACMR of the individual archetype must be greater than a given $ACMR_{min}$ associated with a collapse probability of 20%. If so, the seismic design factors used in the design process were considered adequate to provide an acceptable level of collapse safety. For this example, the $ACMR_{min}$ was equal to 1.49. Therefore, the archetype is approved, 4.91 > 1.49. This procedure was repeated for the 201 archetypes in the set, and the approved/fail status was evaluated on an individual basis.

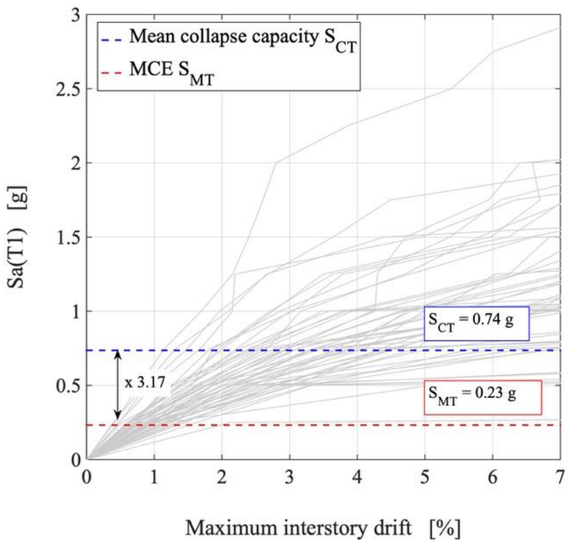


Fig. 7 – IDA results for a five-story building, floor plan "Q", R = 5.5, Δ_{max} = 0.002h, seismic zone, and soil type B

The second evaluation phase was based on group performances. The archetypes in the set were assembled into performance groups that reflect major differences in configuration, design, seismic load intensity, and structural period. The binning of archetype configurations provides the basis for statistical assessment of minimum and average properties of the seismic design factors. Thirty-three performance groups were defined out of the 201 archetypes: 16 for the current seismic design factors, and 17 for the new ones. The average ACMR of the group must be greater than a given $ACMR_{min}$ associated with a collapse probability of 10% to validate the seismic design factors under analyses. For this phase, $ACMR_{min}$ = 1.89.

Fig. 8 shows the ACMR results for each performance group, as well as the minimum $ACMR_{min}$. A preliminary analysis of the data shows that both design factors sets meet the $ACMR_{min}$ requirement to guarantee a collapse probability of less than 10%. It means that the current Chilean design factors (R = 5.5 and Δ_{max} =

0.002) lead to resilient buildings with very good seismic behavior, while less conservative factors (R = 6.5 and Δ_{max} = 0.004) also result in code-compliant structures that fulfill the primary objective of safeguarding life and prevent building collapse. For the current design factors, the average ACMR/ACMR$_{min}$ ratio is equal to 2.40, while for the new ones it is 2.04, showing a decrease of 15% in the over-capacity margin, but without compromising the safety of the structure. It is interesting to note that some performance groups in Fig. 8 show high ACMR values compared to the average. This is due to the location of the archetypes; most of the archetypes in those groups were placed in low seismicity areas (zone I) with good quality soils (type A or B). Therefore, the spectral base design shear was low and the code-defined minimum shear controlled the design.

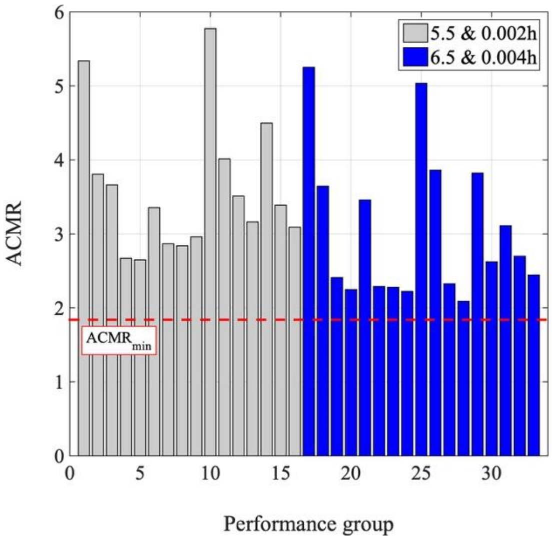


Fig. 8 – ACMR values for each performance group considering two sets of design factors.

6. Conclusions

A comprehensive research project was conducted aiming at proving the feasibility of a new set of seismic design factors for wood frame buildings in Chile. Real-scale tests were performed to gain knowledge on the mechanical behavior of materials, connections, and wall assemblies. Subsequently, an efficient numerical modeling approach was developed to study the lateral response of wood frame walls with different properties, extending the scope of the research beyond the lab results. Finally, the seismic behavior of 201 buildings was analyzed through 3D nonlinear dynamic analyses employing a set of 26 ground motions. Results showed that the current seismic design factors in the Chilean NCh433 standard (R = 5.5 and Δ_{max} = 0.002h) result in buildings with a very safe seismic performance even under demands similar to the maximum considered earthquake. However, it was also found that a modification to R = 6.5 and Δ_{max} = 0.004h could also lead to code-compliant structures with a good seismic performance that fulfill the primary objective of safeguarding life and prevent collapse. Furthermore, using a less conservative set of factors improves the cost-benefit ratio of the buildings and provides a more flexible designing framework for engineers and architects in Chile.

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