Assessment of seismic design factors and proposal of modification to Chilean seismic building design standard (NCh 433) for mid-rise wood light-frame buildings

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1 Introduction

The Chilean forestry industry has increased its productivity and today constitutes, after mining, the second highest exporting sector of the country's economy. The managed forests planted in Chile cover 2.414.208 hectares (INFOR, 2017). However, despite all this potential, the volume of timber residential construction in Chile is much lower than other forested countries in the Americas like the United States and Canada. In these countries, wood construction is used for more than 90% of the total residential construction, while for the year 2017, it represented only 18% of residential construction in Chile (Cámara Chilena de la Construcción 2014). In addition, unlike those other countries where wood light-frame system construction can be used in buildings up to six stories, in Chile it is used for only one- to two-story dwelling houses, thus the Chilean society is missing the advantages of this timber construction system in much of the potential applications.

In recent years, governmental institutions considering the potential of the Chilean forest products industry have supported the development of timber buildings. This within the context of increasing concern about global warming, and as a possible strategy for moving forward on addressing the challenge of providing sustainable alternatives for the future built environment.

However, considering the Chilean high seismic risk, it was necessary to improve comprehension of the seismic behaviour of mid-rise wood-frame systems. In order to increase the allowable building height of wood light-frame structures from two- up to six-stories in the Chilean market, it was required to establish a response modification factor (R-factor) calibrated for mid-rise wood-frame system and the performance expectations of the Chilean standard. This due to uncertainties whether current R-factor in the Chilean seismic design standard was adequate for six-story buildings. Therefore, a project was conducted to determine the most suitable R-factor for wood light-frame building construction up to six-stories in height.

This manuscript summarizes the main issues addressed for the assessment of the seismic performance factor. This investigation involved: (i) develop a set of building configurations for wood light-frame archetype buildings, (ii) structural design of the building archetypes, (iii) conduct monotonic and cyclic tests of shear wall elements, (iv) subassembly level tests: framing-to-sheathing connection tests, wood structural panel sheathing mechanical property tests, nail tests, and testing of wood frame elements to determine their mechanical properties, (v) numerical modelling using the test results as input parameters to complete shear wall and building archetypes models, (vi) nonlinear time history dynamic analysis simulation, and (vii) evaluation of seismic performance factor. The procedure for quantitatively establishing the seismic performance factors with consideration for the performance expectations provided in the Chilean seismic design standard followed the FEMA P-695 guidelines (FEMA, 2009). Thus, a response modification factor (R-factor) for use in force-based design procedures was estimated.

2 Provisions of the Chilean seismic design standard, NCh-433 Of.1996 Mod. 2012/DS61.

The NCh-433 - Building seismic design standard (INN, 2009) defines a response modification factor R-Factor of R=5.5 for timber light-frame shear walls.

One provision included in NCh 433 with relevant impact in the structural design of the wood light-frame buildings is the maximum inter-story drift. It is defined in Section 5.9.2 and states that: *"the maximum relative displacement between two consecutive stories, measured at the centre of mass in each direction of analysis, must not be greater than the story height multiplied by Δ = 0.002 (0.2%)"* (INN, 2009). This inter-story drift requirement is to be determined for the forces associated with the design spectrum reduced by R-factor. No deflection amplification factor (C_d) is defined or required to be used. This is essentially the major difference in the Chilean performance requirements when compared to the common performance requirements in other countries.

The disadvantage regarding the maximum inter-story drift provision is that was originally set to control the performance of reinforce concrete buildings, but actually this provision governs all construction materials equally. Therefore, due to the inherent lower lateral stiffness of timber structures, it is difficult to achieve the high lateral stiffnesses required to control the inter-story drifts. Thus, wood light-frame structural systems are at a disadvantage when compared to other materials. For this reason, this research project also focused on trying to find a maximum drift limit value more suitable to this timber system.

3 Structural archetypes of buildings configurations

A series of structural archetypes were developed with the aim of covering as wide of range of possible configurations for wood light-frame buildings according to FEMA P-695 guidelines. Four building architypes were designed considering the most representative characteristics observed in the Chilean building stock (Cárcamo S., 2017): two configurations representing social housing, and two configurations representing the private market buildings. First, extensive research on residential timber buildings built with the wood light-frame system internationally was completed. Later, extensive research of the most typical construction typologies in reinforced concrete and masonry residential buildings currently being built for the five- and six-story real estate market in Chile was completed. These investigations provided information required to set the different space distributions, symmetries, maximum spans, different tributary areas, simple and complex perimeters, discontinuities, among others for the four archetypes developed. Two of these four archetypes used in the investigation are shown in Figure 1.

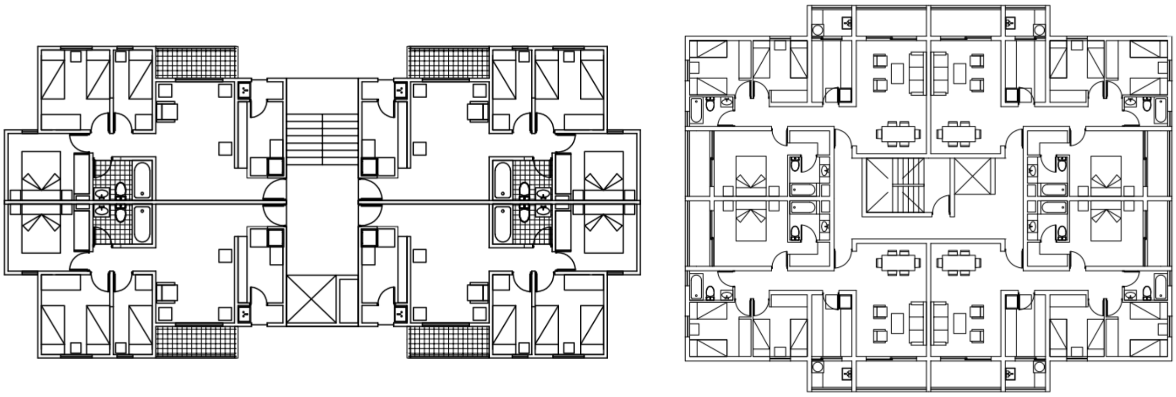

Figure 1. Some buildings configurations, "Type A" (Left) and "Type B" (Right).

These four building configurations were designed using the Equivalent Lateral Force and Modal Response Spectrum analysis procedures following the provisions of the NCh 433 design standard for various locations throughout Chile. The matrix of archetypes includes buildings located at Seismic Zones 1 (A_0 = 0.2g) and 3 (A_0 = 0.4g), and located on Soil Types A, B, C, and D. Four different heights of buildings were considered: three-, four-, five- and six-stories.

For the structural design of the building configurations archetypes, the following standards, papers and guidelines were considered: AWC-SDPWS American Wood Council, (2015), NCh 1198 standard INN, (2014), S. Rossi et al., (2015), NBCC, (2005), Nassani D.E., (2014), BSSC, (2003), CECOBOIS, (2015), Newfield, G et al., (2013), Newfield, G et al., (2015), Leung T. et al, (2010) and APEGBC, (2011).

According to the main purpose of this investigation, regarding the determination of a more suitable R-Factor for the wood light-frame system up to six-stories height in Chile, several trial R-Factors were considered for the structural designs. A set of four seismic design factors were considered: R = 5.5, R = 6.0, R = 6.5, R = 7.0.

Also, different maximum inter-story drifts were considered for the structural designs, because as mentioned on Section 2, it is a key performance parameter. In this sense, the following trial drift limits were considered: Δ = 0.002 (0.2%), Δ = 0.003 (0.3%) and Δ = 0.004 (0.4%).

Regarding anchorage devices for the wood light-frame shear walls buildings, the Hold-Down (HD) devices were considered for the structural designs of the 3-, 4- and 5-story buildings. On the other hand, Anchor Tie-down System (A.T.S.) devices were implemented for the structural designs of the 5- and 6-story wood light-frame buildings.

Additionally, the structural designs of buildings were considered varying the mechanical structural grade of the shear wall studs and sole plates. Therefore, shear walls with a timber frame made of C16 and MGP10 Radiata Pine sawn timber were considered for the building's structural archetypes.

Carrying out all the permutations and combinations between all the different variables described above, the resulting matrix consisted of more than 1.000 different structural building designs. Afterwards all this work, the structural design archetypes associated with the combination of parameters: R-Factor & Δ, of greatest interest were selected to perform the FEMA P-695 analysis methodology (FEMA, 2009). Thus, only 201 archetypes considering the actual NCh 433 standard provisions: R = 5.5 & Δ = 0.002 (0.2%), together with the interest combination given by: R = 6.5 & Δ = 0.004 (0.4%), were selected for the collapse assessment process of the FEMA P-695 methodology.

This resulted in a manageable number of idealized non-linear models to sufficiently represent the range of intended applications for a proposed system being achieved.

4 Experimental program

The experimental program developed for this research project involved testing of shear walls, their sub-assembly components, elements, materials, and relevant connections. Two different anchorage conditions were tested: Hold-Downs (HD) anchor devices and Anchor Tie-down System (A.T.S.).

The shear walls tests were divided into 3 Phases:

Phase 1 included shear walls tests subjected to monotonic and cyclic loading for walls with different aspect ratios (Height / Length), different edge nail spacings, but using the same HD devices. The configurations of the test specimens are shown in Table 1.

Phase 2 also involved shear walls tests with cyclic loading, HD anchors, different edge nail spacings and different wall lengths. However, additional gravity load and overturning moment forces were added. This was done in order to characterize the response of the shear walls of the first story of wood light-framed buildings to the action of tractions and compressions that the overturning moment subjected to them. The additional compression load from the overturning moment could cause a compression failure at the post-studs or buckling of the OSB sheathing panel. On the other hand, the additional tensile load due to the overturning moment action on the shear wall specimens, could cause a HD failure due to a state of greater stress than generated by the traction from the cyclic shear loading alone. The configuration of these test specimens are shown in Table 2.

Phase 3 included tests of shear wall specimens tests subjected to cyclic shear loading with the same aspect ratio, but different nail spacings. However, the main difference is the anchorage condition provided by the A.T.S. These shear wall tests with this anchorage were carried out in specimens of one- and two-story heights. Which were necessary to characterize the response of six-story building models using this type of anchorage, which conceptually works very differently than the traditional HD anchor. The configurations of the test specimens are shown in Table 3.

Thirty-two shear walls subjected to monotonic and cyclic loading were tested during this research program. The details of the specimen's configurations are shown in Tables 1 to 3. All the sawn timber used on the project for studs and sole-plate was 2x6" with exact dimensions of 35 x 138 mm. All the wood frame elements used in constructing the shear walls consisted of mechanically graded, Chilean MGP10 (Australian structural grade) Radiate Pine. The wood framing elements were donated by Arauco. The wood structural panel sheathing used throughout the project was 11.1 mm thick OSB rated sheathing panels, which were installed on both sides of the walls. The OSB panels were donated by Louisiana Pacific. One type of nail was used throughout the project, it was 70 mm long with a nominal diameter of 3.3 mm helical nails installed with pneumatic gun.

The HD and A.T.S. anchoring devices were all donated by Simpson Strong-Tie Company. The HD used on Phase 1 and 2 were HD12. The A.T.S. rod's diameters were used on the shear walls specimens as indicated in Table 3. Shrinkage compensation devices were implemented according to the rod's diameter.

Table 1. Test configuration specimens for shear walls – Phase 1.

Notation	Loading Protocol	Length [mm]	Edge Nailing Spacing [mm]	Anchor Device
M120-10-01	Monotonic	1200	100	Hold-Down
M120-10-02	Monotonic	1200	100	Hold-Down
M120-05-01	Monotonic	1200	50	Hold-Down
M120-05-02	Monotonic	1200	50	Hold-Down
M240-10-01	Monotonic	2400	100	Hold-Down
M240-10-02	Monotonic	2400	100	Hold-Down
M240-05-01	Monotonic	2400	50	Hold-Down
C120-10-01	Cyclic	1200	100	Hold-Down
C120-10-02	Cyclic	1200	100	Hold-Down
C120-05-01	Cyclic	1200	50	Hold-Down
C120-05-02	Cyclic	1200	50	Hold-Down
C240-10-01	Cyclic	2400	100	Hold-Down
C240-10-02	Cyclic	2400	100	Hold-Down
C240-05-01	Cyclic	2400	50	Hold-Down
C240-05-02	Cyclic	2400	50	Hold-Down
C360-10-01	Cyclic	3600	100	Hold-Down
C360-10-02	Cyclic	3600	100	Hold-Down
C70-10-01	Cyclic	700	100	Hold-Down
C70-10-02	Cyclic	700	100	Hold-Down

NOMENCLATURE:

M-120-10-0X: Type of test (M = Monotonic test; C = Cyclic test) – shear wall length – edge nail spacing – No. of specimen test"

Table 2. Test configuration specimens for shear walls with axial load and moment - Phase 2.

Notation	Loading Protocol	Length [mm]	Edge Nailing Spacing [mm]	Anchor Device
C120-10-01	Cyclic	1200	100	Hold-Down
C120-10-02	Cyclic	1200	100	Hold-Down
C120-05-01	Cyclic	1200	50	Hold-Down
C240-10-01	Cyclic	2400	100	Hold-Down
C240-10-02	Cyclic	2400	100	Hold-Down
C240-05-01	Cyclic	2400	50	Hold-Down
C360-10-01	Cyclic	3600	100	Hold-Down
C360-10-02	Cyclic	3600	100	Hold-Down

Table 3. Test configuration specimens for shear walls with A.T.S. - Phase 3.

Notation	Loading Protocol	Length [mm]	Edge Nailing Spacing [mm]	Anchor Device
C240-10-01	Cyclic	2400	100	A.T.S. SR4
C240-10-02	Cyclic	2400	100	A.T.S. SR14
C240-05-01	Cyclic	2400	50	A.T.S. SR10
C240-05-02	Cyclic	2400	50	A.T.S. SR14
Story 1 = C240-10 Story 2 = C240-05	Cyclic	2400	50 100	A.T.S. SR14 A.T.S. SR10

The monotonic tests were conducted by applying a linear increasing load until the shear wall failure was observed. The CUREE-Caltech reversible displacement protocol (Krawinkler H, et al. 2001) was used for the cyclic tests. The cyclic protocol was calibrated based on the results from the initial monotonic tests. Additional information about this experimental campaign results and all the sub-assembly level tests such as: framing-to-sheathing connection tests, panel sheathing mechanical properties tests, nails tests, and testing of wooden frame elements to determine the mechanical properties can be found in Guiñez F, (2019) and Estrella X, (2020).

This experimental campaign was designed to fulfill three fundamental objectives: (1) to prove that structural wood light-frame systems provide adequate lateral resistance for buildings up to 6-stories located in areas of high seismicity in Chile, (2) verify that the SDPWS-2015 design standard (American Wood Council, 2015), can be homologated for use in Chile utilizing Chilean materials and meeting the national constructive practices, and (3) develop reliable characterization variables to use in numerical models.

In this context, it was possible to verify that the light-frame system has an adequate response to cyclic loads. It was also possible to verify that this structural system has a high deformation capacity without losing its structural integrity, and that this system responds in a ductile failure mode. Photos of the shear walls tests described earlier are presented in Figures 2 and 3.

Figure 2. Shear walls tests speciments subjected to cyclic shear plus gravity load and overturning moment action.

Figure 3. Shear walls tests speciments with A.T.S., one story test (Left), two stories test (Right).

5 Numeric Modelling

Numerical models of the tested shear walls specimens were developed in order to simulate the behaviour of these elements when subjected to large deformations within the non-linear range. The mechanical behaviour of the shear walls is simulated using non-linear springs that were calibrated using the reversed cyclic testing of shear walls and their key connections. Several detailed models of shear walls tested were developed using the Matlab program M-CASHEW developed by Pang W & Hassanzadeh S, (2012). This made it possible to extrapolate the results obtained at the laboratory to other shear wall dimensions that were not tested, providing valuable information used to develop and analyse archetype buildings. A model for wood light-framed shear walls was developed, consisting of: (1) Euler-Bernoulli frame two-node elements with 3-DOFs per node to represent the studs and sole plate, (2) Sheathing OSB panels were modelled using rectangular shear-panel elements with 5 DOFs, and (3) two-node-link elements to represent both sheathing-to-framing and Hold-Down connections. A schematic of the developed shear wall model is shown in Figure 4. The mechanical properties of the shear wall components were taken from sub-assembly level tests results as indicated in Estrella X, (2020).

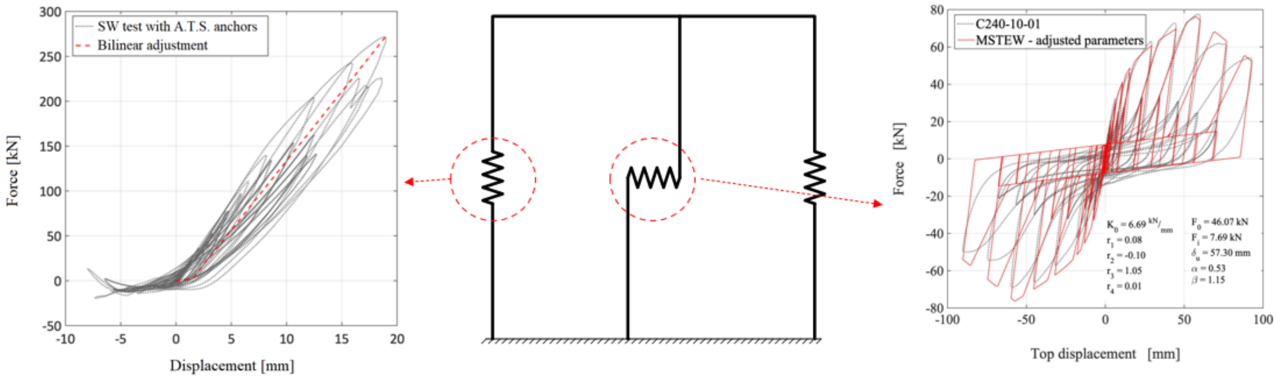

Figure 4. Representation of wood-framed shear walls non-linear model.

The results of the model predictions versus cyclic and monotonic shear walls test results with different aspect ratios are shown in Figure 5. It can be seen good agreements between the model and the experimental responses in terms of maximum force, maximum displacement, initial stiffness, ultimate displacement, ductility, and energy dissipation. The average model errors when predicting each of the aforementioned parameters are 0.8%, 2.3%, 6.9%, 6.8%, 3.8% and 9.3%, respectively. These levels of error were considered admissible for non-linear models.

6 Modelling and nonlinear analysis

The FEMA P-695 presents a rational methodology for the quantification of seismic design factors for structural systems. Non-linear analyses are used in the P-695 methodology to evaluate the buildings capacity in terms of an acceptably low probability of collapse for seismic demands of different intensities. For this, non-linear models were developed for each one of the 201 structural archetypes described earlier.

A 3D nonlinear model was developed for each building archetype. As proposed by Pei and van de Lindt (2009), wood-frame shear walls were modelled using nonlinear spring elements which connect two consecutive floors. The hysteretic behaviour of each wood-frame shear wall was modelled using the Modified-Stewart (MSTEW) model proposed by Folz and Filiatrault (2001). More detailed information about the model formulation proposed for the wood-frame shear walls and the mid-rise building archetypes can be found in Estrella X. et al., (2019a, 2020).

Once the structural nonlinear models for the structural system archetypes were complete, static pushover analyses were carried out for each of the buildings archetypes.

This was to study their maximum resistance, maximum deformation capacity, initial stiffness, and quantify their ductility. The archetypes response was analysed for loading in both directions separately by applying a monotonically increasing lateral load pattern distribution. The load pattern was associated with the first mode of vibration.

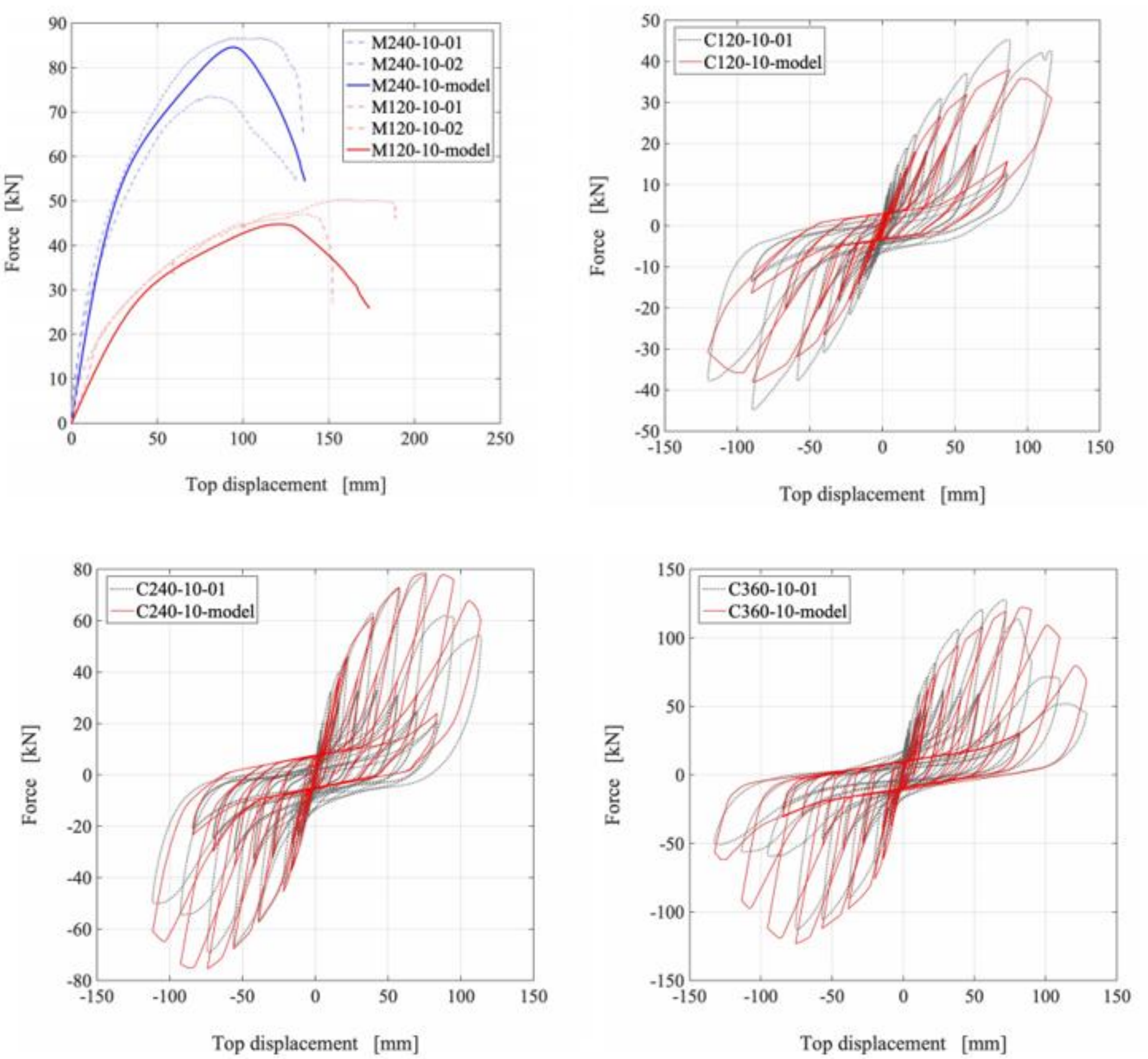

Figure 5. Comparison between shear walls tests and proposed numerical model predictions.

An example the response results for the X-X direction of archetype "Type A" located in Seismic Zone 1, Soil Type B, and structurally designed using both of the interest parameter combinations (R-Factor & Δ) are shown in Figure 6. When analysing the building model response for the X-X direction, it was noted that the strength of the building model falls by 24.8% when the archetype is structurally designed using the combination of parameters given by R = 6.5 & Δ = 0.004, while the initial stiffness decreases only 7.8%. There are no notable changes in the building ductility (displacement capacity).

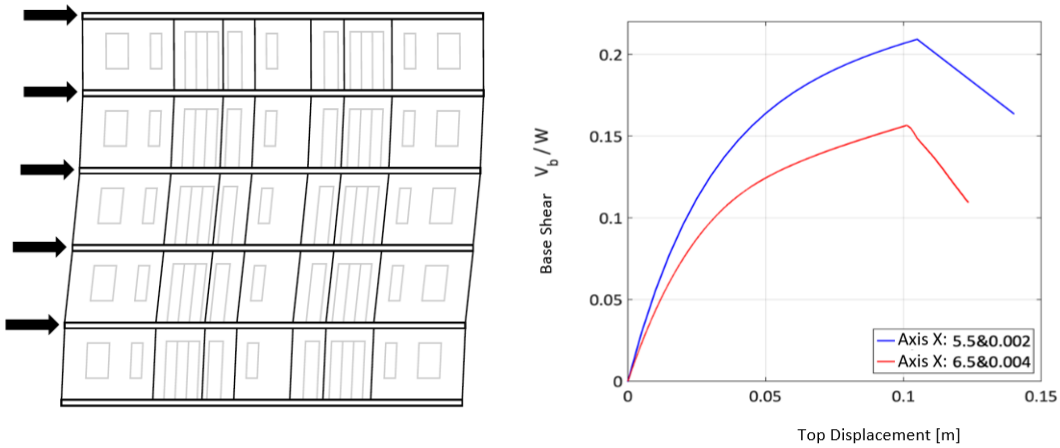

Figure 6. "Type A" building archetype model results for Static-Pushover analysis using the two interest combinations of parameters of seismic design factors and inter-story admissible drifts.

Subsequently, bidirectional Incremental Dynamic Analyses (IDA) were performed for each principal direction of the building archetype models using a set that includes 26 pairs of ground motions records (horizontal components) selected particularly for this study. Thus, for the X-X and Y-Y building main directions, a total of 52 analyses resulted for each combination of design factor and admissible inter-story drift parameters. To compute the collapse capacity of each building archetype, bidirectional IDA analyses were conducted employing the software SAPWood V2.0 developed by Pei and van de Lindt, (2009).

Detailed information about ground motion selection such as: earthquake magnitude, fault type, distance to the fault, record components, intensity measures, number of records per earthquake, accelerogram correction, soil conditions, among others, can be found in Estrella X, (2019b).

The ground motions were monotonically and systematically scaled to the earthquake intensity that caused the collapse of an archetype model. For this research, the collapse of the structure was established as an equivalent drift of 3% of the story height as per FEMA 356 (FEMA, 2000). The scaling protocol followed, for progressively increasing the ground motion record intensities until the structure reached the limit state, is described in detail in Estrella X, (2019b).

It Is known that the calculation of the collapse capacity of structural systems through IDA analyses is influenced by the spectral shape of the ground motions. The approach followed for this research project is described in Estrella X, (2019b).

The FEMA P-695 methodology defines collapse level ground motions as the intensity that would result in median collapse of the seismic-force-resisting system. The median collapse occurs when one-half of the structures exposed to this intensity of ground motion would have some form of life-threatening collapse (FEMA, 2009).

The 52 IDA response curves for the previously presented building archetype (5-story "Type A" building, located at Seismic Zone 1 and Soil Type B) are provided in Figure 7.

The IDA response curves are plotted for the two combinations of parameters of interest. The resulting median of the collapse level ground motions Sa_{COLL} was determined for the 3% inter-story drift for the 5%-damped design level spectral acceleration and the 5%-damped of the Maximum Credible Earthquake (MCE) Sa_{MCE} spectral acceleration. The MCE ground motions is defined as 1.2 times the design earthquake according to the NCh-433 standard.

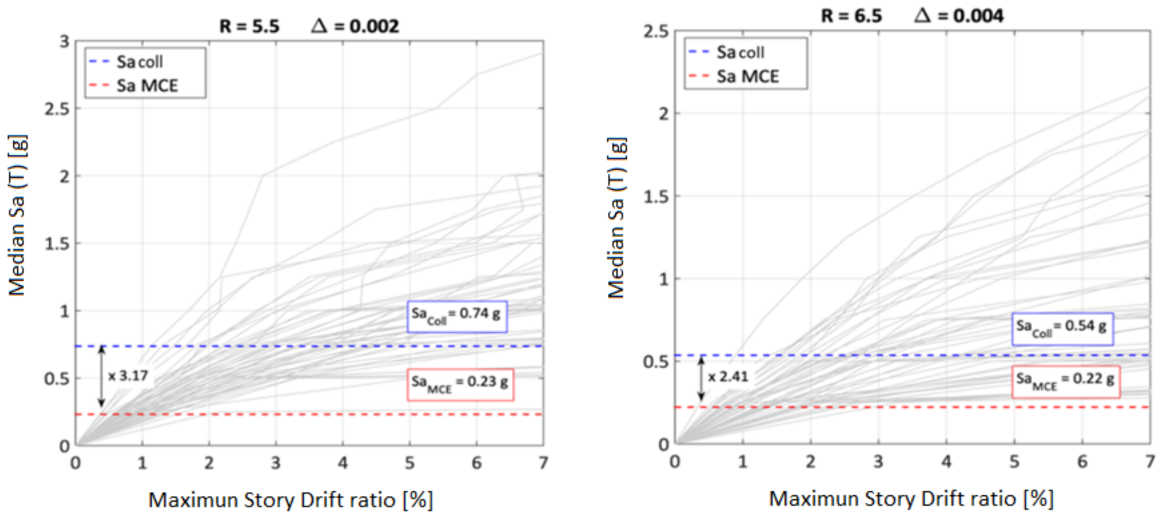

Figure 7. IDA curves responses of "Type A" building archetype model for the two interest combinations of parameters, current NCh-433 standard parameters (Left), proposed modified parameters (Right).

The Collapse Margin Ratio calculated as CMR = Sa_{COLL} / Sa_{MCE} for the cases shown in Figure 7 are 3.17 and 2.41 for the current and proposed NCh-433 parameter values, respectively. Subsequently, the Adjusted Collapse Margin Ratio (ACMR) is calculated, which is obtained by adjusting the CMR by the Spectral Shape Factor (SSF) that depends on the set of seismic records used, and by a factor of 1.2 that is applied when performing bidirectional dynamic analyses (ACMR = CMR x SSF x 1.2) (FEMA 2009). Detailed information about the determination of SSF factors can be found in Estrella X, (2019b).

The ACMRs calculated for each case are presented in Table 4. There was a 25% decrease in the ACMR when the proposed NCh-433 modified parameters (R = 6.5 & Δ = 0.004) were compared to the current parameters.

Section 7.1.2 of the FEMA P-695 methodology defines the collapse performance objectives as: (1) a conditional collapse probability of 20% for all individual wood light-frame archetypes, and (2) a conditional collapse probability of 10% for the average of each of the performance groups of wood light-frame archetypes. The Collapse Margin Ratios computed (CMR), the period-based ductility (μ_T), the Spectral Shape Factors (SSF), and the Adjusted Collapse Margin Ratio (ACMR) are presented in Table 4. Two individual archetypes (incorporating low aspect ratio shear walls) shown in Table 4 pass the acceptable criteria of Adjusted Collapse Margin Ratio performance objective, given by and $ACMR_{20\%}$ of 1.49. The $ACMR_{10\%}$ for the performance objective of the average of each of the performance groups is 1.84.

Table 4. Adjusted Collapse Margin Ratios and Acceptable Collapse Margin Ratios for individual wood-frame archetypes performance objective.

Archetypes	Stories	Comb. of Parameters	CMR	μ_T	SSF	ACMR	$ACMR_{20\%}$	Acceptance Check
C	5	2400	3.17	4.15	1.17	4.86	1.49	Pass
C	5	2400	2.41	4.02	1.51	3.64	1.49	Pass

The aforementioned procedure is equivalent for all the other 201 combinations of structural archetypes, soil types and seismic zones. The complete results considering the 201 structural archetypes, the performance groups, the buildings located in Seismic Zone 1, 3, and Soil Types A, B, C and D are summarized in the Figure 8. Further details regard the seismic performance evaluation, the total system collapse uncertainty and the robustness of the analysis can be found in Estrella X, (2019b and 2020).

It can be seen in Figure 8 that the structural system of wood light-frame shear walls fulfills the acceptance criteria of FEMA P-695.

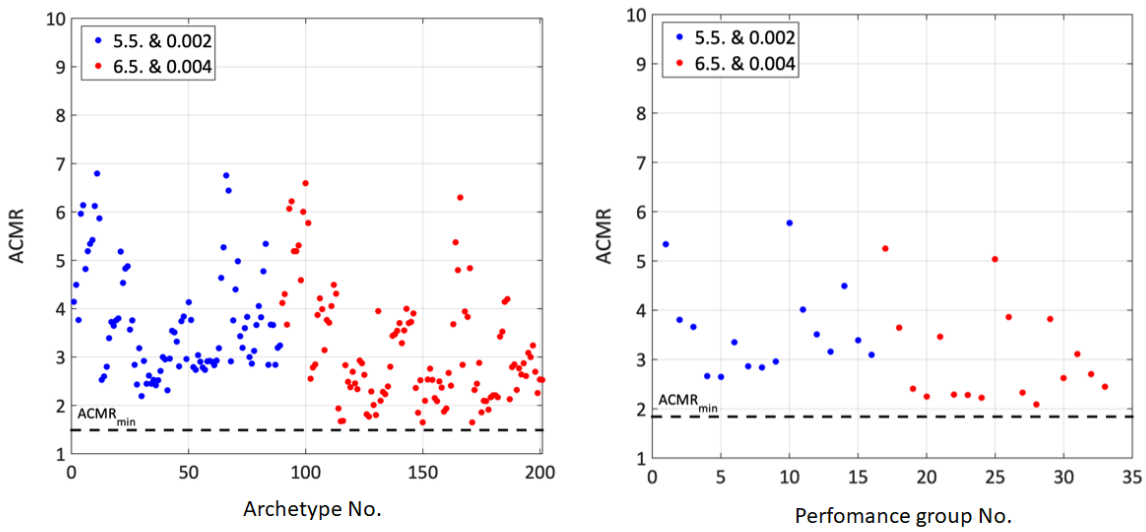

Figure 8. ACMR results for the buildings archetype and perfomance group analysed.

7 Conclusion

The following conclusions can be drawn, based on the analyses presented:

1. The current seismic provisions included in the NCh 433 standard assumes that an R-Factor of R=5.5 provides an acceptable level of collapse safety. However, the results of this research reveal that R=6.5 also meet the collapse performance

objectives of the FEMA P-695 methodology. Therefore, this new R-Factor could be approved as the design seismic factor for the studied structural system. Therefore, this research project recommends a R=6.5 value for R-Factor in Chile.

2. It was found that the maximum inter-story drift included in NCh 433 standard had a significant impact in the structural design of the wood light-framed buildings. Two interest combinations of parameters for R-Factor & Δ were studied in detail. The results show that the combination of current NCh433 standard parameters R = 5.5 & Δ = 0.002 leads to safe, but conservative and stiff buildings.
3. It was also verified that the structural archetype designs for a maximum inter-story drift limit of 0.004 (0.4%) also met the collapse evaluation methodology limits. As a result, it was possible state that building designs are safe for up to 6-stories in height. Therefore, the maximum inter-story drift limit of 0.004 could be implemented for structural designs of the wood light-frame systems studied, as long as non-structural elements in buildings construction are protected and properly designed for the seismic forces at their interface with the main structure.

8 References

INFOR - Instituto Forestal, (2017): El sector forestal chileno 2017 - Chilean forestry sector 2017. https://simef.minagri.gob.cl/bibliotecadigital/handle/123456789/15910.

Cámara Chilena de la Construcción, (2014): “Balance de la Vivienda en Chile”. Disponible en: http://www.cchc.cl/wp-content/uploads/2014/07/Balance-de-la-Vivienda-2014.pdf

FEMA, (2009): Quantification of Building Seismic Performance Factors. Federal Emergency Management Agency, Washington, D.C.

INN - Instituto Nacional de Normalización, (2009): NCh 433 Of96 Mod2009, Diseño Sísmico de Edificios.

Krawinkler H, Parisi F, Ibarra L, Ayoub A, Medina R, (2001): Development of a Testing Protocol for Woodframe Structures. CUREE Report W-02, Task 1.3.2, Consortium of Universities for Research in Earthquake Engineering. Richmond, California.

Guiñez, F., Santa-Maria, H., Almazán, J., (2019): Monotonic and Cyclic Behaviour of Wood Frame Shear Walls for Mid-Height Timber Buildings. Engineering Structures. 189. 100-110. 10.1016/j.engstruct.2019.03.043.

American Wood Council, (2015): Special Design Provisions for Wind and Seismic, ANSI/AWC SDPWS-2015, Leesburg, VA.

Pang W, Hassanzadeh S. (2012): Corotational model for cyclic analysis of light-frame wood shear walls and diaphragms. J Struct Eng;139:1303–17. doi:10.1061/(asce)st.1943-541x.0000595.

Cárcamo S., (2017): Informe Nº 7 – Configuraciones arquitectónicas propuestas, Proyecto Corfo 16BPE-62260. Centro de Innovación en Madera CIM-UC.

Instituto Nacional de Normalización. NCh 1198: Madera - Construcciones en Madera – Cálculo. 2014, Santiago.

Rossi S, et al., (2015): Seismic Elastic Analysis of Light Timber-Frame Multi-Storey Buildings: Proposal of an Iterative Approach, Constr. Build. Mater. http://dx.doi.org/10.1016/j.conbuildmat.2015.09.037

NBCC, (2005): National Building Code of Canada. Part 4: Structural Design. Canadian Commission on Building and Fire Codes, National Research Council of Canada (NRCC), Ottawa, Canada.

Nassani D, (2014): A Simple Model for Calculating the Fundamental Period of Vibration in Steel Structures. Hasan Kalyoncu University. Turkey.

BSSC, (2003): FEMA 450: Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. Building Seismic Safety Council, National institute of building sciences, Washington DC.

Centre d' expertise sur la consruction commerciale en bois (CECOBOIS), (2015): Guide technique sur la conception de bâtiments de 5 ou 6 étages à ossature légère en bois - Volume 2: Exemple de calcul d'un bâtiment de six étages à ossature légère en bois. Dépôt légal Bibliothèque nationale du Québec.

Newfield, G., Ni, C., Wang, J., (2013): A Mechanics-Based Approach for Determining Deflections of Stacked Multi-Story Wood-Based Shear Walls. FPInnovations, Vancouver, B.C. and Canadian Wood Council, Ottawa, Ont.

Newfield, G., Ni, C., Wang, J., (2015): Design Example: Design of Stacked Multi-Storey Wood-Based Shear Walls Using a Mechanics-Based Approach. FPInnovations, Vancouver, B.C. and Canadian Wood Council, Ottawa, Ont.

Leung T. et al, (2010): "Predicting Lateral Deflection and Fundamental Natural Period Of Multi-Storey Wood Frame Buildings". Paper for the world conference on timber engineering WCTE-10.

APEGBC, (2011): Structural, Fire Protection and Building Envelope Professional Engineering Services for 5 and 6 Storey Wood Frame Residential Building Projects (Midrise buildings). APEGBC Technical and Practice Bulletin. Professional Engineers and Geoscientists of BC. Canadá.

Pei S, van de Lindt J W., (2009): Coupled Shear-Bending Formulation for Seismic Analysis of Stacked Wood Shear Wall Systems. Earthquake Engineering And Structural Dynamics. 38: 1631-1647.

Folz, B., Filiatrault, A. (2001): Cyclic Analysis of Wood Shear Walls. Journal of Structural Engineering, 127(4): 433–441.

Estrella, X., Almazán, J., Guindos, P., Santa María, S., and Malek, S. (2019a): Seismic Design Factors for Wood Frame Buildings. IV Latin American Conference on Timber Structures, Montevideo, Uruguay.

Estrella, X., Guindos, P., Almazán, JL., Malek, S., (2020): Efficient Nonlinear Modeling of Strong Wood Frame Shear Walls for Mid-Rise Buildings. Engineering Structures 215, 110670.

Estrella, Xavier & Guindos, Pablo & Almazán, Jose. (2019b). Ground Motions for FEMA P-695 Application in Subduction Zones. Latin American Journal of Solids and Structures. 16. 10.1590/1679-78255848.

FEMA, (2000): FEMA 356 - Prestandard and Commentary for the Seismic Rehabilitation of 869 Buildings. Federal Emergency Management Agency, Washington, D.C.

Pei, S, van de Lindt J W. User's Manual for SAPWood for Windows. Version 2.0. Colorado State University. 2010.

Santa María H, Caicedo N, Montaño J, Almazán J L, (2016): Towards Timber Mid-Rise Buildings in Chile: Structural Design Challenge and Regulations Gaps. In: *World Conference on Timber Engineering*. 2963-2970.