SEISMIC ANALYSIS AND DESIGN OF AN LTF BUILDING COMPLYING WITH THE NEW ZEALAND AND CHILEAN SEISMIC STANDARDS USING NOVEL EXPRESSIONS FOR THE EQUIVALENT LINEAR-ELASTIC PROPERTIES OF THE PANELS

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ABSTRACT

This work presents the seismic design and analysis of a four-storey residential building constructed with OSB-sheathed light timber framed (LTF) walls. This building was used to develop a three-dimensional finite element model used for modal spectral analysis according to the NCh433 Chilean and NZS1170.5 New Zealand standards. The LTF walls were modelled with shell elements implemented with equivalent properties which consider all the sources of deformation of the panels under seismic actions via an equivalent thickness and equivalent shear modulus, combining and optimizing previous expressions available in the literature. After an iterative process which involved changes in the location, length, and structural properties of the LTF walls, a structure which complied with the requirements of NCh433 and NZS1170.5, and the timber Chilean code NCh1198 and Australian standard AS1720 (to be adopted in New Zealand), was defined. It was found that the inter-storey drift limitations of the Chilean standard were the most restrictive code-requirements. It was concluded that the proposed equivalent elastic properties provide a simple and easy to implement tool for seismic analysis of LTF buildings.

1 INTRODUCTION

Chile and New Zealand, as many other countries, have a large stock of existing single-storey buildings constructed with light timber framing (LTF), many of these serving as residential houses. Nevertheless, relatively few multi-storey buildings structured with LTF walls sheathed with plywood have been constructed to date in both countries, although in New Zealand the number is growing.

In New Zealand, older examples include the buildings reported by Banks [1] (Auckland) and Milburn and Banks [2] (Wellington), whose structure included four and six storeys constructed with LTF panels. More recently Te Ara o Puanga (Mary Potter Hospice Apartments), a five storey LTF and plywood building on steel and concrete podiums in Wellington, has been completed. In Chile, on the other hand, only the 4-storey “Peñuelas Tower” (Viña del Mar) has been constructed to date in the most restrictive hazard zone of the country and was erected to prove the feasibly of LTF multi-storey construction rather than for formal residential or commercial use.

As Chile and New Zealand are located in high-seismicity regions, the design of LTF multi-storey buildings require analysis against earthquake loading, following the seismic standards NCh433 [3] and NZS1170.5 [4], respectively. In practice, to implement the linear-elastic seismic analysis methods prescribed by these standards, such as the equivalent static or modal spectral response methods, it is desirable to have a finite element model constructed in typical commercial software. To define the properties of the elements representing the LTF walls in such model, in turn, equivalent linear-elastic properties are required.

Although limited information is available to date on the subject, the procedures proposed by Newfield et al. (2013) [5] and in Carradine et al. (2019) [6] provide guidance on how to obtain such properties, based on similar mechanistic approaches. Both methodologies obtain the elastic properties of a linear elastic solid wall with equivalent thickness, t_{eq}, length, L_{eq}, and mechanical properties G_{eq} and E_{eq} (shear and Young's modulus, respectively), from the characteristics of the area of the end-chords, the thickness of the sheathing panels, the configuration and stiffness of the sheathing connectors and hold-downs, and the perpendicular-to-grain deformation of the bottom beam supporting the end-chords. Even though both methods consider the same effects for relating such variables, different assumptions are made, and modelling approaches used in such methodologies lead to different values of the equivalent properties, and some aspects in [5] are not covered in [6] and vice versa.

Integrating aspects of both methodologies, this paper presents a simpler and more straightforward formulation of the equivalent properties of the LTF walls, mostly based on [5], but including additional aspects addressed in [6], suitable for shell elements. The formulation is used for determining the properties of the LTF walls of a 4-storey case study residential building included as a design example in [6], analysed with the response spectrum methods required by NCh433 [3] and NZS1170.5 [4], and designed to comply with the wood standards NCh1198 [7] (Chile) and AS1720.1 [8] (to be adopted in New Zealand with modifications) [9, 10]. This paper presents the most important challenges and highlights of such designs, after providing a detailed description of the construction of the compact formulations proposed for the equivalent properties of the panels.

2 EQUIVALENT ELASTIC PROPERTIES OF LTF PANELS

The equivalent properties determined in this paper are based on what is proposed in Newfield et al. (2013) [5]. It is firstly assumed that: (1) $l_{eq} = l_T$; (2) $E_{eq} = E_0$, where l_T is the total length of the LTF wall and E_0 is the parallel-to-grain elastic modulus of wood, and l_{eq} and E_{eq} are the equivalent length and elastic modulus, respectively. What was proposed in [5] was improved in two aspects: (a) the flexibility of the anchorage was explicitly included in the calculation of the equivalent shear modulus, G_{eq}, integrating what was proposed by Carradine et al. (2019) [6]; and (b) the procedure to calculate the equivalent thickness of the walls, t_{eq}, used to model the behaviour of the wall in bending, was summarized in a single equation, therefore easier to implement in practice.

2.1 Proposed equivalent thickness

To obtain the equivalent thickness, the moment of inertia (MOI) of the equivalent section of the

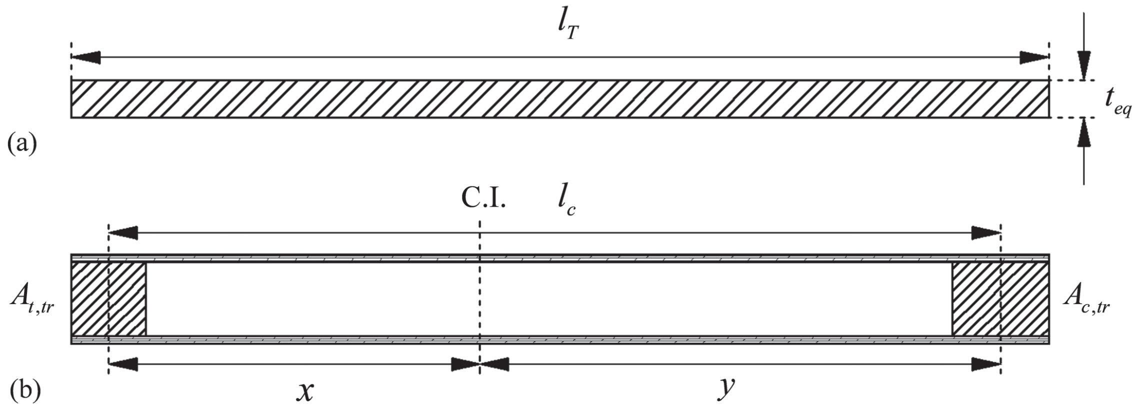

Figure 1: *LTF wall: (a) equivalent elastic panel cross-section; (b) transformed areas and centre of inertia*

$$I_{eq} = \frac{t_{eq} l_T^3}{12} \quad (1)$$

LTF panel is related with the transformed areas of the end chords resisting compression and tension, including the following effects: (a) the axial flexibility of the chord itself; (b) the flexibility of the hold-down resisting tension; and (c) the perpendicular-to-grain flexibility of the horizontal bottom plate [5]. Figure 1(a) presents the section of a LTF wall, whose equivalent MOI (I_{eq}) is given in Equation 1, where l_T and t_{eq} are the total length and the thickness of the equivalent linear-elastic panel, respectively.

The actual cross-sectional MOI of the walls (I_a), referred to the centre of inertia (C.I.) presented in Figure 1(b), is calculated as follows. The distances x and y that define the location of C.I. (Figure 1(b)), measured from the centre of the studs in tension and compression, respectively, can be calculated with Equations 2 and 3. The inertia I_a is calculated with Equation 4. Replacing Equations 2 and 3 into Equation 4, leads to Equation 5, which can also be written as Equation 6.

$$x = l_c \frac{A_{c,tr}}{A_{c,tr} + A_{t,tr}} \tag{2}$$

$$y = l_c \frac{A_{t,tr}}{A_{c,tr} + A_{t,tr}} \tag{3}$$

$$I_a = A_{t,tr} x^2 + A_{c,tr} y^2 \tag{4}$$

$$I_a = A_{t,tr} \left(\frac{l_c A_{c,tr}}{A_{c,tr} + A_{t,tr}} \right)^2 + A_{c,tr} \left(\frac{l_c A_{t,tr}}{A_{c,tr} + A_{t,tr}} \right)^2 \tag{5}$$

$$I_a = l_c^2 \frac{A_{c,tr} A_{t,tr}}{A_{c,tr} + A_{t,tr}} \tag{6}$$

In Equations 2 to 6, $A_{c,tr}$ and $A_{t,tr}$ are the 'transformed' areas of the end chords resisting in compression and tension, respectively, as shown in Figure 1(b). The transformed areas are used for including other sources of flexibility, as discussed next, but could also be thought of as those corresponding to LTF walls having different amounts of wood for each end chord (i.e., an asymmetric LTF wall). Imposing $I_{eq} = I_a$ and combining Equations 1 and 6, t_{eq} can be expressed as in Equation 7, a novel expression.

$$t_{eq} = \frac{12 l_c^2}{l_T^3} \left(\frac{A_{c,tr} A_{t,tr}}{A_{c,tr} + A_{t,tr}} \right) \tag{7}$$

The transformed area in compression accounts for: (a) the parallel-to-grain flexibility of the chord in compression; and (b) the perpendicular-to-grain flexibility of the bottom plate, as proposed by [5]. It is derived from the calculation of the total axial flexibility *(L/EA)* resulting from the addition of the two flexibilities in series, as schematically shown in Figure 2. The flexibilities of the chord and of the plate correspond to the first and second terms of the left hand-side of Equation 8. The resulting $A_{c,tr}$ is given in Equation 9, where H_S, H_c, and h_c are the dimensions shown in Figure 2(a), A_c is the cross-sectional area of the stud in compression, and E_0 and E_{90} are the modulus of elasticity of the timber parallel and perpendicular to the grain direction, respectively.

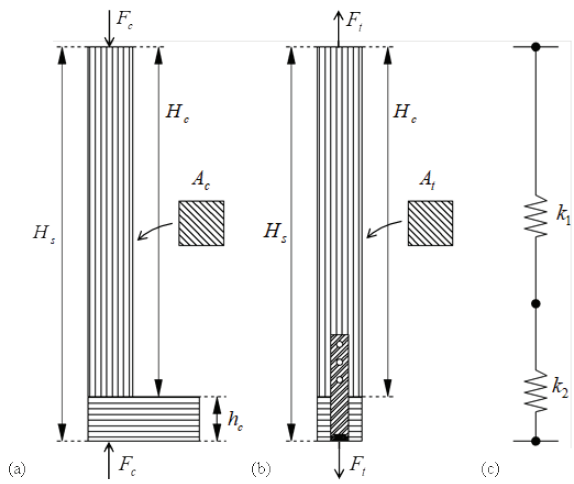

Figure 2: *Total flexibility of the chords: (a) under compression; (b) under tension; (c) springs in series idealization*

$$\frac{H_c}{E_0 A_c} + \frac{h_c}{E_{90} A_c} = \frac{H_s}{E_0 A_{c,tr}} \tag{8}$$

$$A_{c,tr} = \frac{H_s E_{90} A_c}{H_c E_{90} + h_c E_0} \tag{9}$$

The transformed area in tension, in turn, accounts for: (a) the flexibility of the chord in tension; and (b) the flexibility of the hold-down, as proposed by [5], and corresponds to the first and second terms of the left-hand side of Equation 10. The resulting $A_{t',tr}$ is given in Equation 11, where H_s and H_c are the dimensions shown in Figure 2(b), A_t is the area of the stud in tension (note that $A_t = A_c$), and K_{hd} is the axial stiffness of the hold-down.

$$\frac{H_c}{E_0 A_t} + \frac{1}{K_{hd}} = \frac{H_c}{E_0 A_{t,tr}} \tag{10}$$

$$A_{t,tr} = \frac{H_c A_t K_{hd}}{H_c K_{hd} + E_0 A_t} \tag{11}$$

To construct an expression for computing t_{eq}, firstly Equation 7 is rewritten as Equation 12

$$t_{eq} = \frac{12 l_c^2}{l_T^3}\left(\frac{1}{A_{c,tr}} + \frac{1}{A_{t,tr}}\right)^{-1} \tag{12}$$

Replacing Equations 9 and 11 into Equation 12 leads to Equation 13. After some algebraic manipulation, Equation 13 can be expressed as Equation 14.

$$t_{eq} = \frac{12 l_c^2}{l_T^3}\left(\frac{H_c E_{90} + h_c E_0}{H_s E_{90} A_c} + \frac{H_c K_{hd} + E_0 A_t}{H_c A_t K_{hd}}\right)^{-1} \tag{13}$$

$$t_{eq} = \frac{12 l_c^2}{l_T^3}\left(\frac{H_c A_t K_{hd}\left(H_c E_{90} + h_c E_0\right) + H_s E_{90} A_c \left(H_c K_{hd} + E_0 A_t\right)}{H_s E_{90} A_c H_c A_t K_{hd}}\right)^{-1} \tag{14}$$

Further, rearranging terms, Equation 14 can be written as Equation 15, a novel expression which allows for computing t_{eq} with just one single step, given the geometry, mechanical properties of the timber, amount of wood in the end chords, and the stiffness of the hold-down.

$$t_{eq} = \frac{12 l_c^2 A_c A_t H_s H_c E_{90} K_{hd}}{l_T^3\left[H_c A_t K_{hd}\left(H_c E_{90} + h_c E_0\right) + H_s E_{90} A_c\left(H_c K_{hd} + E_0 A_t\right)\right]} \tag{15}$$

Note that Equation 15 is a general expression which can have $A_c \neq A_t$. For the particular and typical case where $A_c = A_t = A_{ch}$ (cross-sectional area of the end chords), Equation 15 takes the form of Equation 16.

$$t_{eq} = \frac{12 l_c^2 A_{ch} H_s H_c E_{90} K_{hd}}{l_T^3\left[H_c K_{hd}\left(H_c E_{90} + h_c E_0\right) + H_s E_{90}\left(H_c K_{hd} + E_0 A_{ch}\right)\right]} \tag{16}$$

2.2 Shear modulus

The equivalent shear modulus, G_{eq}, includes the shear stiffness of the sheathing panels and the flexibility of the connections between these and the internal wooden frame [6]. Both [5] and [6] propose formulas that include both effects using the same logic. However, Carradine et al. [6] explicitly includes the flexibility of the panel to framing connections using the parameter K_{ser}.

For determining G_{eq} (Equation 17), the expression proposed in [6] was preferred over that presented in [5], as the latter requires a pre-design of the building under study to estimate an initial force in the connector. Nevertheless, to be consistent with the thickness of the equivalent panel t_{eq}, the original expression in [6] was multiplied by t_b/t_{eq}, where t_b is the total thickness of the sheathing boards.

$$G_{eq} = \frac{1.2}{t_{eq} l_T^3}\left[\frac{1}{G \cdot t_d} + \frac{2s}{K_{ser} \cdot l \cdot h}\left[\left(1+n\right)l + \left(1+m\right)h\right]\right]^{-1} \tag{17}$$

In Equation 17, G, t_d, h, and l are the shear modulus, the total thickness, the height, and the length of the sheathing panels, respectively; s is the spacing of the connectors, and n and m the number of horizontal and vertical joints in the panel. Note that $h = H_s = H_c + h_c$ in the typical case (see Figure 2). The value of K_{ser}, in turn, can be obtained from Eurocode 5 [11], for different types of framing timber, fasteners and structural panels.

3 CASE STUDY DESCRIPTION

3.1 Architectural and structural layouts

The case study structure is a 4-storey apartment with a total height of 13.5 m, and footprint area of 14.1 m by 13.0 m. It is based on the design example of [6] but modified to decrease torsional effects and to comply with not only the New Zealand, but also the Chilean seismic standards. The height of the first three floors is 3.0 m, while that of the fourth is 4.5 m, resulting in a total height of 13.5 m.

Figure 3 presents the original architectural elevations and the typical plan layout of the building, described in [6]. Figure 4 shows the structural plan of the building with its dimensions, and the length of the LTF walls considered in each direction. The lengths of

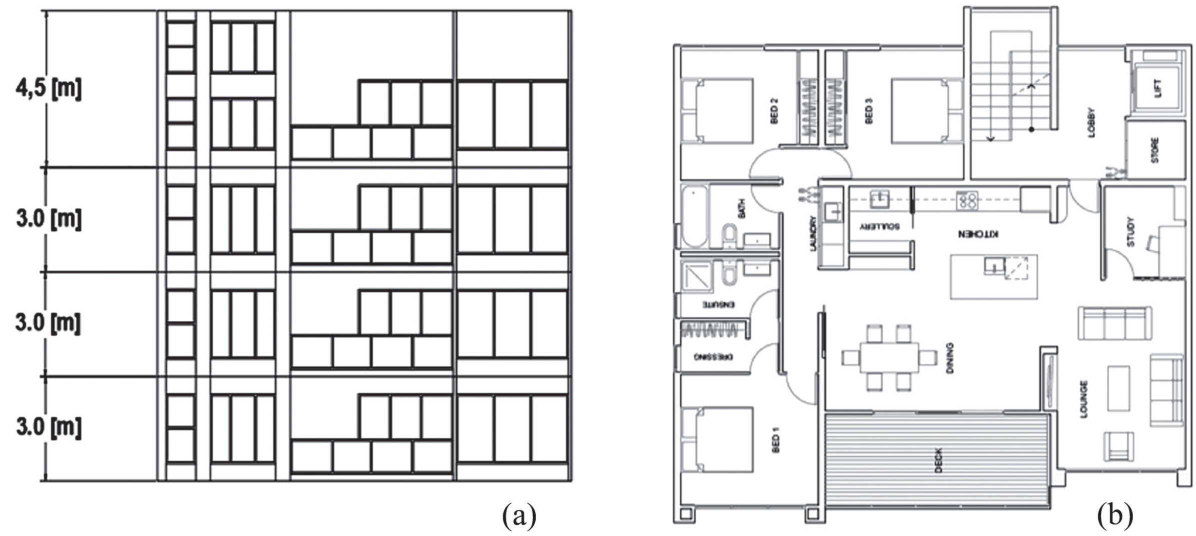

Figure 3: *Architectural drawings of the case study structure, taken from [6]; (a) elevation and (b) typical plan*

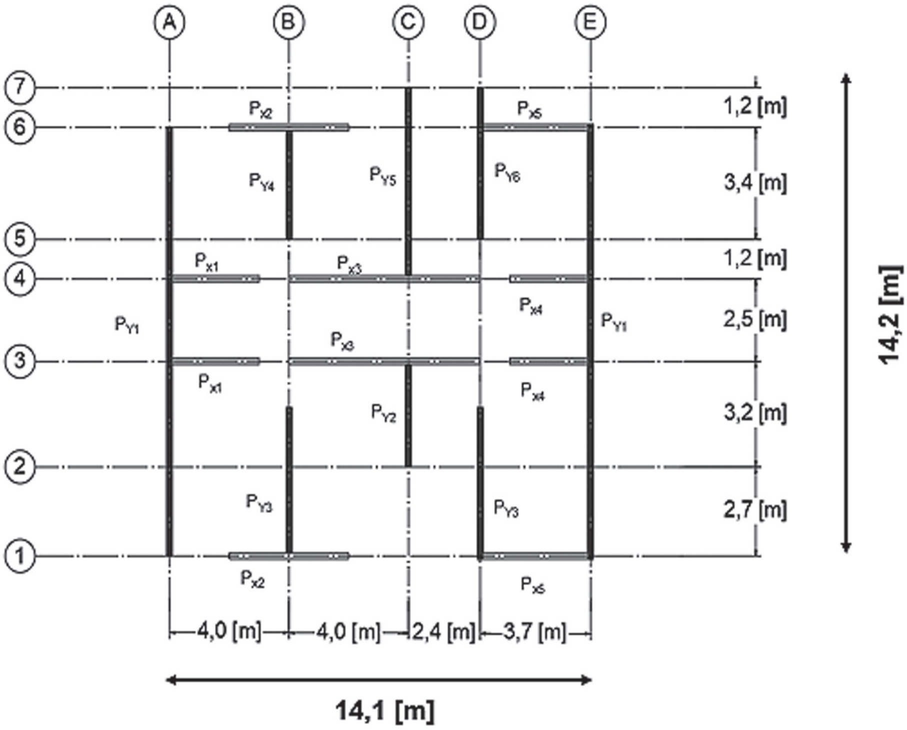

Figure 4: *Final structural layout of the LTF walls*

all the LTF walls meet the requirements for stability and deformation given in [3]. It is worth noting that the proposed structural layout would result in modifications to the architectural drawings, following the iterative process typically done in practice.

Figure 5 shows the characteristics of the components of the LTF walls and their connections. For all walls, the spacing of the inner studs and of the horizontal blocking were 300 mm and 600 mm, respectively. The number of studs lumped at the ends of the wall, designed to take the overturning moment, vary depending on the wall type. The configurations of the walls for both directions are presented in Table 1 and Table 2. The hold-downs considered in the study were those tested by Tamagnone et al. [12], which have a stiffness K_{hd} = 4280 kN/m. Even though in practice it is normal to use only one hold-down, in this study a larger number was used for the sake of completing the design. A different hold-down or hold-down system could be used in the event that only one anchorage can be used per wall end.

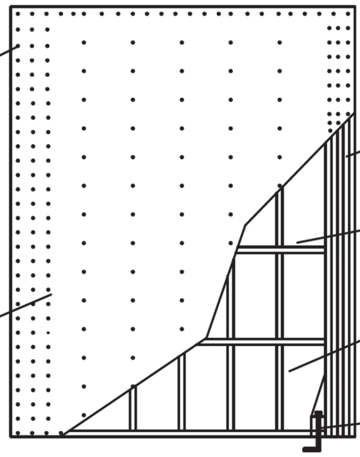

Figure 5: *Characteristics of the structural components and connections of the LTF walls considered in the design. Note: the drawing does not show ply splices and nails are shown as indicative*

Table 1: X - direction wall configurations

Wall	Length (m)	N° of studs in end chords	Number of Hold-downs	Number of sheathing panels
Px1	3.0	10	4	2
Px2	4.0	15	4	2
Px3	6.4	20	6	2
Px4	2.7	10	2	2
Px5	3.7	15	4	2

Table 2: Y - direction wall configurations

Wall	Length (m)	N° of studs in end chords	Number of Hold-downs	Number of sheathing panels
Py1	13.0	20	6	2
Py2	3.2	12	2	2
Py3	4.5	15	4	2
Py4	3.4	12	4	2
Py5	5.8	15	4	2
Py6	4.6	13	4	2

4 MODAL SPECTRAL ANALYSIS

After an iterative process which involved changes in the location, length, and cross-sectional area of the end chords, fastener spacing and sheathing thickness of the LTF walls, a structure which complied with the seismic requirements of the Chilean and New Zealand standards was defined. It is worth noting that the changes in location of the walls from the architectural drawings resulted in modifications to the architectural layout. This would often not be an option in a project where the wall locations were fixed, but for this exercise it was considered reasonable in order to avoid having to incorporate other structural systems. In general, with fixed architectural layouts, it is feasible to alter stud numbers, but not wall lengths and locations. It is also important to understand how the ductility, which comes from the nails in the case of plywood shear walls, may also be considered during the iteration process and can have significant impacts on the final design of the building.

The structure was modelled using the program ETABS v16.2 [13]. The walls were defined as shell elements with thickness t_{eq}, each of them implemented with a linear-elastic homogeneous material with properties E_0 and G_{eq}. The length of each wall was the total length of the panel, as defined in Figure 1. The modal spectral analyses were carried out following the requirements of the Chilean standard NCh433Of96 modified in 2012 (NCh433-2012) [3] and the New Zealand Standard NZS1170.5: 2004 [4]. It was assumed that the structure was built on top of soil type C (per [3] and [4]) and located in a hazard zone with effective ground acceleration of 0.4 g. A factor R_0 = 7 was used for generating the reduction factor R^* of NCh433, whereas a ductility factor μ = 2.0 was considered for calculating the spectral reduction factor k_μ of NZS1170.5:2004 [9]. The 'performance factor' stipulated by NZS1170.5:2004, was taken as S_p = 0.70, as applicable in this case. It is important to note that for LTF buildings, commonly assumed numbers for the ductility factor μ are in the range of 3.0 to 3.5 and μ = 3 is recommended in Carradine et al. (2019) [6] as a starting point. For this work, a more conservative value of μ = 2.0 was considered, which still led to smaller sizes of the LTF walls compared to the Chilean standard NCh433, mostly due to the strict inter-storey drift limitation for a serviceability limit state (SLS) of this code [9].

Through the design process, it was found that the most restrictive code-requirement to comply with was the inter-storey drift ratio limitation of NCh433 (0.2%), implicitly associated with SLS. This restriction led to stiffer walls compared to those required to satisfy the New Zealand drift limitations. The required stiffness of the walls was achieved by providing thicker sheathing panels and additional end-studs to the LTF walls [9]. Figure 6 shows the design spectra used in the analyses, according to the Chilean and New Zealand regulations. For New Zealand regulations, the design spectra corresponding to the ultimate limit state (ULS) and the serviceability limit state (SLS) were defined. For the Chilean case, in turn, the spectra is defined at SLS only, as depicted in the relative magnitude of these limit state spectra per both codes considered. Table 3 summarizes the computed fundamental periods of the structure per both codes, caused by a difference in the percentage of the live load required in the calculation of the seismic mass (25% and 30% for the Chilean and New Zealand cases, respectively).

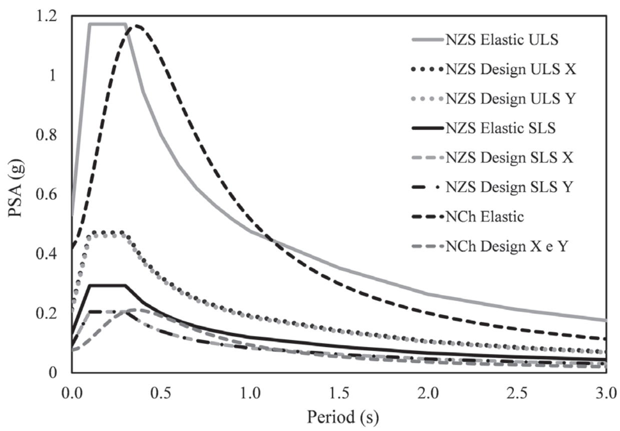

Figure 6: *Design spectra per NCh433 Chilean [3] and New Zealand [4] standards*

Table 3: Fundamental periods and equivalent mass in the direction of analysis

Code	Analysis Direction	T^* (s)	*%Me*
NCh433	Y	0.512	65.03
	X	0.504	67.02
NZS1170.5	Y	0.549	78.70
	X	0.518	72.17

Table 4: Base shear modal spectral analysis results

Standard	Analysis Direction	W_s (kN)	$Q_{o,el}$ (kN)	$Q_{o,el}/W_s$ (%)	Reduction Factor	Q_o (kN)	Q_o/W_s (%)
NCh433	Y	1375.6	981.5	71.4	5.53	177.6	13.0
	X		989.0	71.9	5.64	175.5	12.8
NZS1170.5	Y	1366.2	700.8	51.3	2.54	275.8	20.2
	X		735.3	53.8	2.48	296.4	21.7

The results of the base shear obtained with the spectral modal analysis are presented in Table 4, where $Q_{0,el}$ is the base shear obtained with the elastic non-reduced spectrum, Q_0 is the base shear obtained with the reduced spectrum, and W_s is the seismic weight of the structure. Table 4 also compares the reduction factors of the elastic spectra, and the resulting design base shear. Figure 7 presents the maximum inter-storey drift ratios (*dr*) obtained with the modal spectral analyses, and a complete quadratic modal combination (CQC).

In the case of the analyses with the standard NCh433, all the *dr* values were less than 0.2%, the limit required by the code (associated to SLS). In the case of the analyses per the standard NZS1170.5, in turn, the *dr* values complied with the limit of 2.5% stated for ULS. For SLS, on the other hand, NZS1170.5 does not specify a drift limitation. Nevertheless, the drift limit of plasterboard walls in-plane (0.33%) prescribed by the standard AS/NZS1170.0 Appendix C [14] is commonly referred to in practice. For this work, 0.3% was considered, based on the results of shake-table tests of an LTF building reported in [15], which is slightly more restrictive than 0.33%.

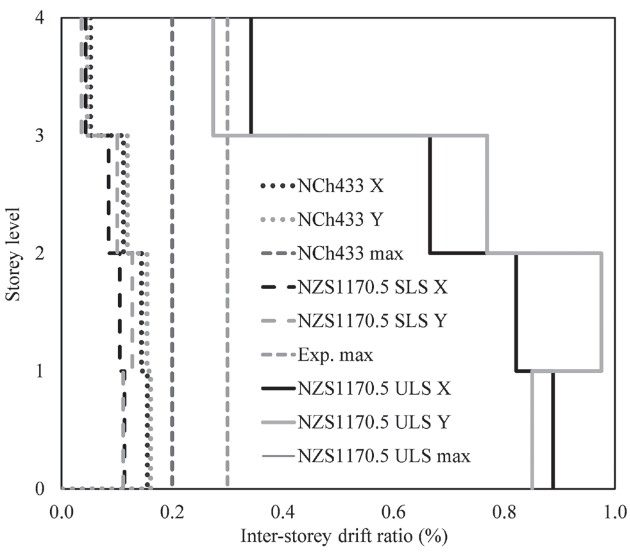

Figure 7: *Maximum inter-storey drifts obtained from analyses and comparisons with limits of standards NCh433 Chilean [3] and New Zealand 1170.5 [4].*

5 CONCLUSIONS

This work presents the seismic design and analysis process involved in the calculation of a 4-storey building constructed with plywood sheathed LTF walls, according to the Chilean and New Zealand standards. It presents an integrated and synthesized version of two available procedures for determining equivalent linear elastic properties of LTF wall panels, suitable for modelling in typical finite element analysis software. The utilization of such procedures allowed for the analysis of the building under response spectra methods. As a result of the investigation, it was found that even though the limitation of displacement between floors required by the Chilean code for SLS was quite restrictive, it was possible to find a wall configuration which satisfied such requirements. Nevertheless, the final design of the walls involved greater amounts of chord area as well as thicker sheathing.

6 REFERENCES

1. Banks W. 1996. Gulf View Towers: A case study for multi-storey timber construction. In: Int. Wood Eng. Conf., New Orleans, LA.

2. Milburn J., Banks W. 2004. Six-level timber apartment building in a high seismic zone, New Zealand Timber Design Journal, Vol12(3) 9-13

3. Instituto Nacional de Normalización (INN). 2012. Seismic Design of Buildings, Norma Chilena Oficial NCh433Of.96mod2012, Santiago, Chile (In Spanish)

4. Standards New Zealand (SNZ). 2004. NZS1170.5:2004: Structural design actions: Part 5, Earthquake actions, SNZ. Wellington, New Zealand

5. Newfield G., Ni C., Wang J. 2013. A mechanics-based approach for determining deflections of stacked multi-storey wood-based shear walls. FPInnovations, Vancouver, B.C.

6. Carradine D. et al. 2019. Multi-storey light timber-framed buildings in New Zealand – Engineering design, BRANZ, Judgeford, New Zealand

7. INN: NCh1198.Of2006. 2006. Madera - Construcciones en madera - Cálculo, Santiago, Chile (In Spanish)

8. Standards Australia. 2010. AS 1720.1-2010 Timber structures. SAI, Strathfield, NSW, Australia

9. Cortes, M. A. 2020. Análisis y Diseño Sísmico de un Edificio Estructurado con Paneles Livianos de Madera (LTF), Trabajo de Título (Ingeniería Civil), Universidad de Valparaíso, Chile (In Spanish)

10. Cortes, M. A., Quintana Gallo, P., Carradine, D. 2021. Analysis and design of a 4-storey LTF building complying with the Chilean and New Zealand seismic standards, WCTE 2021, Santiago, Chile

11. European Standard (EN). 2004. Eurocode 5: Design of timber structures- Part 1-1: General- Common rules and rules for buildings, Brussels, Belgium

12. Tamagnone G., Rinaldin G., Fragiacomo M. 2018. A novel method for non- linear design of CLT wall systems, Journal of Structural Engineering, Vol 167: p. 760-771

13. Computers and Structures Inc. 2018. Etabs v18.1.1, Three-Dimensional Analysis of Building Systems, Berkeley, CA

14. Standards Australia and Standards New Zealand. 2002. AS/NZS1170.0, Structural design actions: Part 0, General Principles, Wellington, New Zealand

15. van de Lindt J., Gupta R., Pei S., Tachibana K., Araki Y., Rammer D. and Isoda H. 2012. Damage Assessment of a Full Scale Six- Story Wood-Frame Building Following Triaxial Shake Table Tests, Journal of Performance of Constructed Facilities, Vol 26; p. 17-25

Technical Note

ROTATIONAL STIFFNESS OF ROCKING CLT WALLS

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1 HORIZONTAL DEFLECTIONS OF CLT WALLS

CLT bracing walls resist horizontal loads through cantilever action, either fixed to the foundations or to bracing walls below. The horizontal loads tend to deform the wall and drift limits will need to be checked for both ULS and SLS load combinations. The stiffness of the wall affects the seismic demand acting on the structure, as it directly impacts the first mode period.

There are several deformation contributions to be considered in a typical CLT shear wall. These are presented by Lukács (2019) and can be summarized as follows:

- Bending deformation (governed by the bending stiffness EI)

- Shear deformation (governed by the shear stiffness GA)

- Translation deformation (governed by the stiffness of the shear connections)

- Rotation deformation (governed by the anchorage flexibility, i.e. hold down stiffness and compression stiffness).

These contributions are intuitive for single storey walls and can be easily calculated. For multi-storey buildings, the rotation of the top of the wall from the bending deformation and anchorage flexibility also needs to be added to the deformation of the walls above as shown in Figure 1. This additional rotation is not to be neglected, as it adds significant deformation to a multi-storey shear wall. Refer to BRANZ (2019) for a more in-depth explanation on the cumulative rotation contributions. Although the BRANZ guide is intended for light timber framing walls, it also provides general guidance on how to calculate the various deformation contributions, which is also relevant for CLT walls.

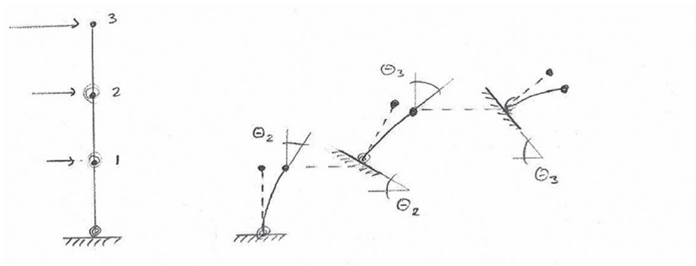


Figure 1: *Cumulative rotation of walls*

This note only focuses on the rotational deformation of CLT walls, with the remaining contributions to be determined by applying first principles (refer also to Lukács (2019)).

Due to limited CLT panel sizes available because of manufacturing and transportation restraints, CLT walls might require vertical splices. The kinematic model of spliced walls is more complex when compared to a single wall as it is a function of the stiffness of the different connections involved (i.e. hold down stiffness and stiffness of vertical splice). More information on the deformation of vertically segmented walls can be found in Masroor et al. (2022).

2. HORIZONTAL DEFLECTION DUE TO ANCHORAGE FLEXIBILITY

The governing equation linking moment and deformation for a single storey rigid wall with a flexible anchorage at the base can be written as follows

$$M = K_\theta \, \theta \tag{1}$$

where

M = base moment of wall

K_θ = rotational stiffness (force x length / angle) of the anchorage at the base

θ = wall rotation at the base

The base moment is typically generated by the storey shear H^* and for a single storey wall the relation between force and moment is

$$M^* = H^* h \qquad (2)$$

where

H^* = horizontal force (storey shear)

h = height of wall.

The horizontal deformation of the wall due to the rotational flexibility of the anchorage at the base can hence be written as

$$\Delta = \theta\, h = \frac{M}{K_\theta} = \frac{H^* h}{K_\theta} \qquad (3)$$

Figure 2 shows the schematic model of a cantilevered shear wall with a rotational spring of stiffness K_θ at the base. This model can be used for modelling purposes when the deformation of a single or multi-storey wall building is to be dertmined in a simplified computer model. Note that this model does not show the translational spring required for any flexibility from the shear connection.

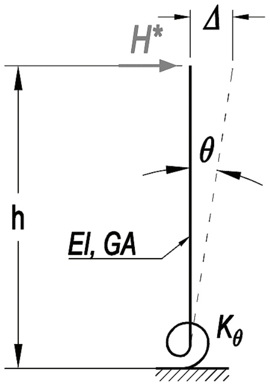


Figure 2: *Schematic model of a cantilevered shear wall*

The next sections provide further guidance on how to determine the rotational stiffness K_θ.

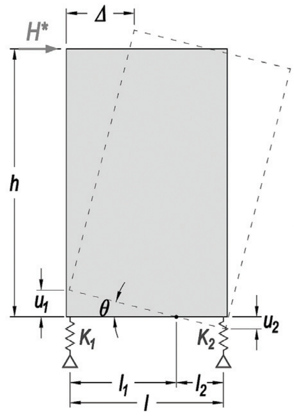
3. DETERMINING THE ROTATIONAL STIFFNESS K_θ

The rotational stiffness K_θ is a function of the wall lever arm, the tension stiffness of the hold down connection at the toe and the compression stiffness at the heel of the wall. Depending on the geometry and the level of accuracy required, the following three cases as summarized in Table 1 can be considered:

1. Generic case with different tension and compression stiffness
2. Equal tension and compression stiffness
3. Tension stiffness and rigid compression support

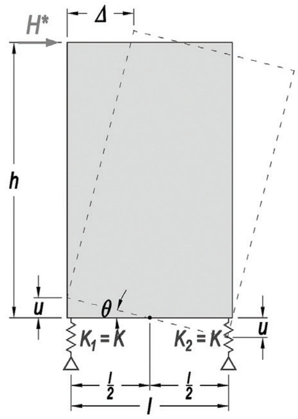
Table 1 Rotational stiffness K_θ for cantilevered walls

1. Generic wall system with different tension and compression stiffness



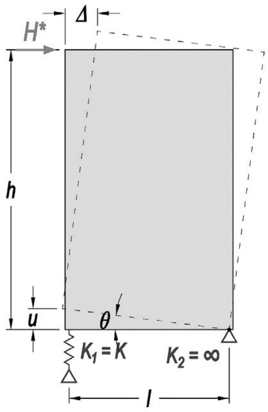
$$K_\theta = \frac{l^2 \cdot (K_1 \cdot K_2)}{K_1 + K_2} \qquad (4)$$

2. Wall system with equal tension and compression stiffness



$$K_\theta = \frac{l^2\, K}{2} \qquad (5)$$

3. Wall system with tension stiffness and rigid compression support



$$K_\theta = l^2\, K \qquad (6)$$

where

K_θ = rotational stiffness due to anchorage flexibility

K_1 = tension stiffness of hold-down/anchorage system

K_2 = compression stiffness of wall or substrate

l = lever arm (distance between hold-down and centre of compression area), refer to section 4.1

Case 1 is typically used for walls sitting on timber floors. For SLS load cases, a triangular distribution of compression loads is typically considered. For ULS load cases, a stress block, assuming the timber is fully plasticized under the bearing area, is typically used. For CLT walls sitting on a concrete foundation, case 3 is typically adequate.

A note of caution when carrying out seismic design. By assuming a shorter lever arm or a more flexible compression or tension stiffness, the wall will become more flexible. This might lead to higher drifts, but the increased building period leads to a reduction of the spectral acceleration. It is therefore recommended to carry out a sensitivity analysis on the assumed values.

4. FURTHER CONSIDERATIONS REGARDING THE CALCULATION OF THE ROTATIONAL STIFFNESS

4.1 Determination of the lever arm

Determining the lever arm l for rocking timber shear walls is not trivial. Lukács (2019) summarizes different approaches proposed by different researchers. In general terms, the lever arm is the distance between the hold down at the toe and the resultant of the compression area at the heel of the wall. Depending on the accuracy required, the lever arm can be simplified as a percentage of wall length (i.e. between 75% and 90% of the wall length) or can be calculated based on equilibrium considerations. When applying the latter, engineering judgement is required on determining if the compression area is fully plasticized (rectangular stress block, typically used for ULS load cases) or if it is still working in the elastic range (triangular stress distribution, typically used for SLS load cases).

4.2 Stiffness in series

If more than one stiffness contribution acts in series, this needs to be accounted for accordingly (i.e. hold-down above floor, hold-down under floor, tension bolt etc.). The following equation can be used when determining the total stiffness for several springs in series:

$$K_{tot} = \frac{1}{\sum \frac{1}{K_i}} \tag{7}$$

where

K_{tot} = sum of all stiffness for springs in series

K_i = individual spring stiffness

4.3 Tension stiffness

The stiffness of hold downs can either be obtained from manufacturer literature or can be determined from first principles and by using codified values for nail, screw or bolt slip. Depending on the hold down geometry, it is important to also account for any elongation of the steel plate or strap as well as the elongation of the connecting bolt or anchor. For hold downs with an angled shape, also the bending of the plate needs to be considered. Refer to Figure 3 for a schematic representation of these contributions.

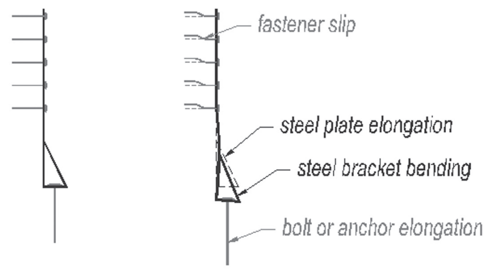


Figure 3: *Hold down in its undeformed and deformed state with deformation contributions*

It is important to note that codified values for fastener slip or stiffness are only approximate and have a high level of variability (Jockwer et al., 2021). Designer regularly use the slip modulus K_{ser} as defined in Eurocode 5 CEN (2008), providing line stiffness values for various timber fasteners. Comparisons with tested values suggest that the increasing factor of 2 for steel plates as suggested by the standard should not be used. The New Zealand timber standard NZS AS 1720.1 (SNZ, 2022) provides a new set of equations to determine the deformation of timber fasteners.

4.4 Compression stiffness

Depending on the wall substrate with either a very stiff material like concrete or steel, or a more flexible timber floor, the compression stiffness is either governed by the compression stiffness of the longitudinal laminates in the wall or the compression perpendicular to grain stiffness of the floor.

The stiffness of timber parallel to grain is relatively high, and it is typically sufficient to use Equation (6) to determine the rotational stiffness. When the wall sits on a timber floor (i.e. CLT floor panels), then the compression stiffness perpendicular to the grain together with the floor thickness need to be considered.

The compression stiffness of a wall supported by a timber floor can be calculated with the following simplified equation.

$$K_c = \frac{E_{90}\, A_p}{h_c} \tag{8}$$

where:

K_c = compression stiffness of the floor (compression perpendicular to grain)

A_p = bearing area (depth of assumed compression area)

h_c = depth of floor panel

E_{90} = modulus of elasticity perpendicular to the grain of the floor

A more precise compression stiffness formulation leading to lower deformations can be determined by considering the stress distribution in the timber substrate. A possible approach has been developed by Blass and Görlacher (2004) and is summarized in BRANZ (2019) for one-dimensional stress spreading. CLT panels have the ability to spread the stresses in two directions, further reducing the compression deformation. A future technical note will provide more information on the stress spreading in CLT panels.

4.5 Modulus of elasticity perpendicular to the grain

The modulus of elasticity perpendicular to the grain of timber is not well documented and is not specified in the New Zealand timber design standard. The ratio between the moduli of elasticity parallel and perpendicular of timber is reported to be 15 and 30 for sawn timber and glulam, respectively (CEN, 2004). The Canadian CLT Handbook and the ProHolz Guideline for the design of CLT (ProHolz, 2014) refer to a ratio of 30 for CLT panels, but more testing on this value is required. Engineering judgement should be applied when using these reference values. More information on the bearing strength values and modulus of elasticity perpendicular to the grain of SG timber will be published in a future technical note, as the upcoming New Zealand timber standard NZS AS 1720.1 will lower the bearing strength for timber.

If the load path with compression stresses perpendicular to grain is avoided by local reinforcement or by removing any perpendicular to grain contact, the compression stiffness can typically be assumed to be infinitely rigid.

5. REFERENCES AND FURTHER READING

Blass, H. and Görlacher, R. (2004). Compression perpendicular to the grain. Proceedings of the 8th World Conference on Timber Engineering, Vol. II, Lahti, Finland.

Carradine, D. (2019). Multi-storey light timber-framed buildings in New Zealand: Engineering design. BRANZ, Judgeford, New Zealand.

Casagrande, D., Doudak, G. and Polastri, A. (2019). A proposal for the capacity-design at wall- and building-level in light-frame and cross-laminated timber buildings. Bulletin of Earthquake Engineering 17

CEN (2008). EN1995-1-1. Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings. European Committee for Standardisation, Brussels, Belgium.

CEN (2004). EN1995-2. Eurocode 5: Design of Timber Structures, Part 2 Bridges. European Committee for Standardisation, Brussels, Belgium.

Jockwer, R., Caprio, D., and Jorissen, A. (2021). Evaluation of parameters influencing the load-deformation behaviour of connections with laterally loaded dowel-type fasteners. Wood Material Science & Engineering, Vol. 17, No. 1, 6-19.

Lukacs, I., Björnfot, A. and Tomasi, R. (2019). Strength and stiffness of cross-laminated timber (CLT) shear walls: State-of-the-art of analytical approaches. Engineering Structures, Volume 178.

Masroor, M., Doudak, D., and Casagrande, D. (2022). Design of Multipanel CLT Shear Walls with Bidirectional Mechanical Anchors Following Capacity-Based Design Principle. Journal of Performance of Constructed Facilities, Volume 36 Issue 1 proHolz (2014). Cross-Laminated Timber Structural Design. Basic design and engineering principles according to Eurocode. proHolz Austria, Vienna, Austria.SNZ (2022).

NZS AS 1720.1 Timber structures. Part 1: Design methods. In publication, refer to DZ NZS AS 1720.1 Public consultation draft, Nov 2018