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INCREMENTAL DYNAMIC ANALYSIS OF A FIVE STOREYS CLT BUILDING DESIGNED THROUGH FORCE-BASED METHODS

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ABSTRACT: Since the large seismic demands in Chile, there is a high level of uncertainty regarding the seismic safety behavior of timber structures. Besides, in the last decade, there has been a worldwide rise in the construction of cross-laminated timber (CLT) buildings, explained by the ease of industrialization of CLT panels and for being a sustainable material. However, its massification in high earthquake-prone zones is still under development. This research analyzes the seismic fragility of a 5-story CLT building. The aim of this research is to determine the seismic safety level of a CLT midrise building designed by Chilean force-based code regulations by means of Static Nonlinear Analysis and Incremental Dynamic Analysis. For IDA curves, a set of 540 nonlinear time-history analyses were performed using parallel computing tools. The main results show that for the collapse of the building to occur, large pseudo-acceleration values need to be reached. Besides, the collapse margin ratio (CMR) is calculated for the two principal directions, achieving values larger than 3 for 50% collapse probability. These results suggest a large margin of seismic safety of the CLT building designed in this research.

KEYWORDS: Seismic fragility of CLT mid-rise buildings, Timber construction, Incremental Dynamic Analysis.

1 INTRODUCTION

In the last decade, there has been a growing interest in promoting sustainable and industrialized construction materials, in which timber structures have advantages against other materials. In terms of construction systems, CLT is one of the most attractive systems that satisfy all these purposes. It is also a resistant system with good structural performance that has been extensively tested at element [1] and full scale [2] levels.

In general, there is sufficient evidence that shows that CLT can withstand high gravitational load and have an optimal behavior against seismic loads [3]. This last aspect is crucial in a highly seismic country such as Chile. However, in Chile there are no explicit seismic design regulations for CLT. This means, for example, that CLT buildings must meet the same deformation requirements of a reinforced concrete structure. This fact can generate over stiffened and oversized structures that do not take advantage of the deformation capacity that CLT can provide. Additionally, the application of traditional force-based design methods based on the seismic behavior of other structural systems creates uncertainty about the earthquake response and safety of CLT structures. This research aims to quantify through incremental dynamic analyses (IDA) the seismic safety of a panelized CLT mid-rise building designed under the force-based methods approach.

2 PYMELAB BUILDING

The Pymelab building was developed through a collaborative effort between public and private organizations [4]. It was funded by the Production Development Corporation of the Chilean Ministry of Economy (CORFO) and executed by the University of Bío-Bío (Concepción, Chile). It is a project that gathers a large number of SMEs in the timber sector to innovate and create associative support networks. As a result of this collaborative project, a non-habitable 5-story cross-laminated timber building was built (Pymelab Building), providing the timber companies with a living laboratory to test and validate their products and manufacturing capacities.

2.1 BUILDING DESCRIPTION

The 5-story CLT building has a plan dimension of 6.6 m length by 4.2 m width, with 2.5 m of inter-story height. Three representative views of the studied building are shown in Figure 1. On the first three floors, the platform system is employed considering 150 mm thick walls (5 layers CLT panels); while on the upper 2 storeys, the balloon frame system with 100 mm thick walls (3 layers

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panels) is used. Besides, following the local industry manufacturing capacity, walls and slabs consider a panelized configuration with panels of a maximum width of 1.2 m (walls) and 1.8 m (slabs). Spline joints are used for the parallel in-plane panel-to-panel connection, and self-tapping screws are employed for the perpendicular wall-to-wall and slab-to-wall connections. The geometry and structural information of the Pymelab structures are presented in Table 1. The structural system is based on the concept of a simple regular prism with stiff and strong corners and concentrated openings in the middle zone of each elevation, providing vertical continuous load paths.

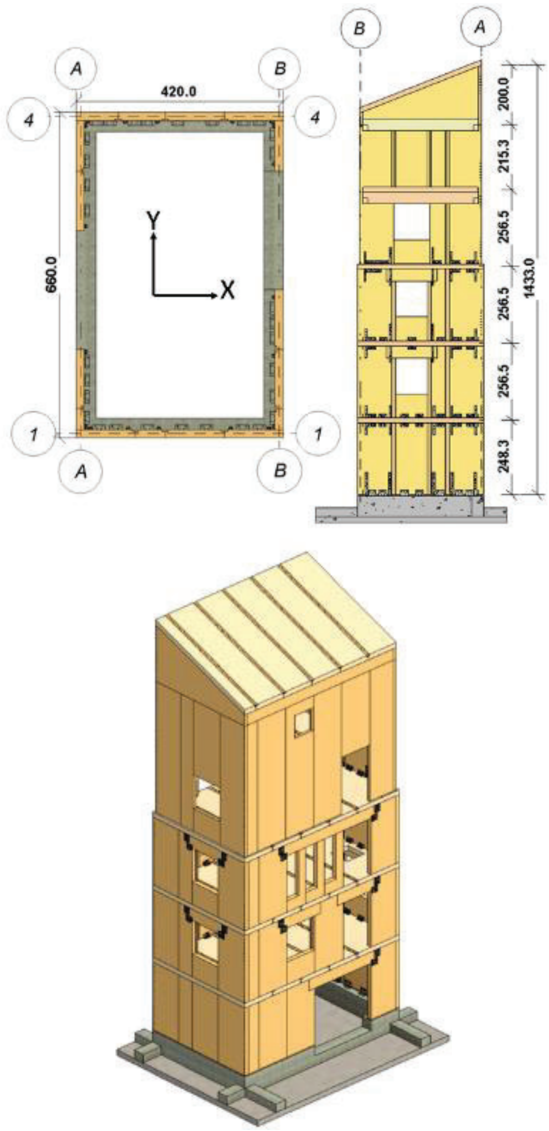

Figure 1: *Floor plan, elevation (top), and 3D view of the general structural configuration (bottom)*

Table 1: *Summary of main properties of the PymeLab building [5]*

Property	Value
N° Stories (Height)	5 (12.5 m)
Story area	27.7 m^2
Wall thickness	165 mm – 100 mm
Slabs thickness	165 mm
N° Metal connectors	2.0 /m^2
Total Wall Ratio (1st story)	10%
CLT Volume	0.34 m^3/m^2

2.2 SEISMIC DESIGN PROCEDURE

For the structural design, the allowable stress design approach was used, following the Chilean seismic code [6], the Chilean structural wood design standard [7], and international recommendations [8]. The mechanical properties of radiata pine CLT panels used in the walls and slabs are listed in Table 2.

Table 2: *Mechanical properties of CLT walls and slabs*

Mechanical properties	Value
Elastic Modulus Ex (3 layers)	2750 MPa
Elastic Modulus Ey (3 layers)	5250 MPa
Elastic Modulus Ex (5 layers)	1755 MPa
Elastic Modulus Ey (5 layers)	6250 MPa
Shear Modulus	324 MPa
Poisson Coefficient	0.3
Density	450 kg/m^3

SAP2000 [9] finite element software was utilized to develop the structural analysis for the design. All connections were considered flexible and modeled using link elements. Non-dissipative elements were modeled as elastic springs, while dissipative elements were modeled as multi-linear springs. Additionally, GAP contact elements were included to simulate the contact between wall-to-wall, wall-to-slab, slab-to-wall, and wall-to-foundation interfaces. Figure 2 displays an elevation of a case study, the graphical representation of the model created in SAP2000, and an example of the deformation of one elevation subjected to lateral seismic loads.

Regarding the seismic design, according to the Chilean code (NCh 433 [6]), the lateral load demand is defined using the maximum seismic coefficient C=0.46, which is established for non-traditional structures and calculated using a load reduction factor R=2. Additionally, the lateral load design must comply with a maximum center of mass inter-story drift of 0.2% under reduced seismic loads. This deformation limit applies to all structural systems regardless of the material used. The seismic demand is applied as lateral forces in the mass center of each floor slab. Additionally, to take into account the accidental torsion at each story level, the mass center where displaced 10% of the plan whith. Lateral forces and weight per story are summarized in Table 3. The seismic weight is determined as the Dead Load plus 25% of the

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Live Load, resulting in 2.5 kN/m^2/story and a base shear of 142 kN.

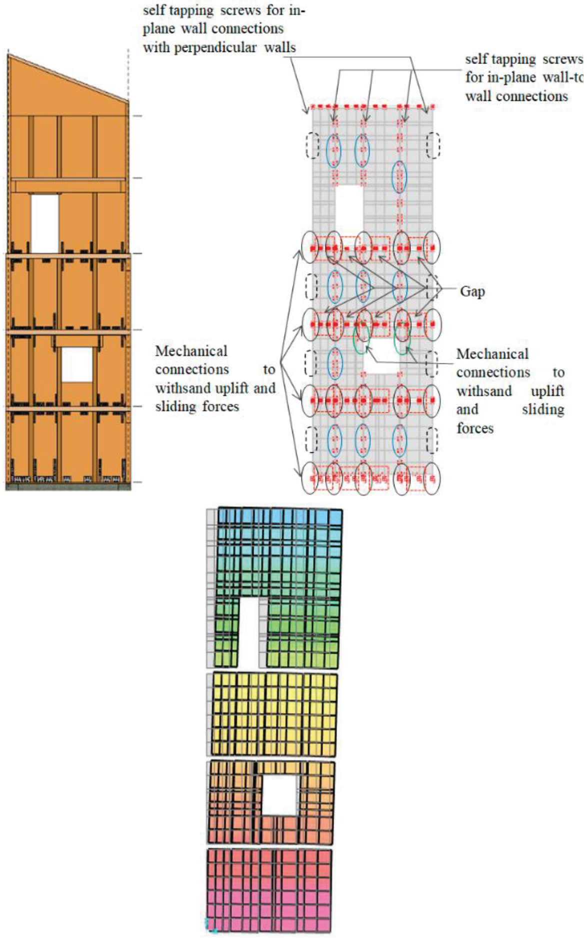

Figure 2: *1-Axis elevation, numerical design model (top), and deformed shape under seismic loads (bottom)*

Concerning the dynamic response, the design periods calculated using the Ritz vector method for the X and Y directions are 0.318 s and 0.274 s, respectively.

Table 3: *Summary of the seismic weight and lateral load demand used in the design stage*

Story	Lateral Force (kN)	Seismic weight (kN)
1	17.9	70.1
2	20.7	68.2
3	22.0	70.2
4	26.3	61.0
5	55.3	44.2

The foundation is a 20 cm thick reinforced concrete mat of 8 m x 6.4 m, stiffened with 0.7 m x 0.4 m beams. The foundation system design was challenging due to the site soil conditions, as well as the slenderness of the building. The soil beneath the mat is a medium-compacted and low-bearing capacity sandy silt. One of the main problems of the foundation is that the structure, being so light and slender, has overturning stability issues. To address this situation, additional weight was added over the foundation to improve the overturning safety, achieving stability safety factors of 1.3 and 1.7 for Y and X directions, respectively.

In terms of connections design, Izzi et al. (2018) [10] recommendations and authors' experience were followed to define the dissipative and non-dissipative connections. The joints that resist the rocking and the wall-in-plane splines are designed to be dissipative to improve the deformation capacity of the system. On the other hand, shear brackets, perpendicular joints, slab-to-wall, and slab-to-slab spline connections were considered non-dissipative.

2.3 STRUCTURAL DETAILING

Due to the panelized configuration of the building, a large number of metal plates and screw connectors are required. Examples of the connectors and metal plates used in this case study include:

Self-tapping screws: used for joints between panels at perpendicular joints. Among the screws used are ESCRC 8x240 screws for perpendicular joints between wall panels, and ESCRC 10x240 screws for joints between walls and slabs.

Angle Brackets: angular-type metal connectors are typically used to resist shear and uplift forces resulting from seismic demands. The angular type brackets used are ABR255 and ABR255SO models fastened with CNA 4x60 annular shank nails.

Hold-Down: HTT31, HTT22e, and HTT5 devices fastened with CNA 4x60 annular shank nails are employed to resist uplift loads and to create vertical load transfer paths.

Spline joints: employed for the in-plane panel-to-panel connection. In this case, radiata pine boards fastened with ESCRC 8x80, ESCRC 8x120, and DSVR312R5LB screws were considered.

Table 4 shows the per-story amount of shear brackets and hold-down connectors used in the Pymelab building.

Table 4: *List of shear and hold-down connectors per story (C: ceiling, F: floor)*

Story		Shear Brackets		Hold-down		
		ABR255SO	ABR255	HTT31	HTT22e	HTT5
5	C	0	0	0	0	0
	F	0	0	0	0	0
4	C	0	0	0	0	0
	F	0	28	0	6	12
3	C	0	26	0	6	12
	F	0	30	0	7	10
2	C	0	18	0	7	10
	F	0	32	0	3	14
1	C	0	32	0	3	14
	F	31	0	19	0	0

Due to the strict requirements of the Chilean seismic design code (NCh433 [6]), a crowded connector layout is obtained. Figure 3 provides an elevation view and a plan of the metal connector distribution.

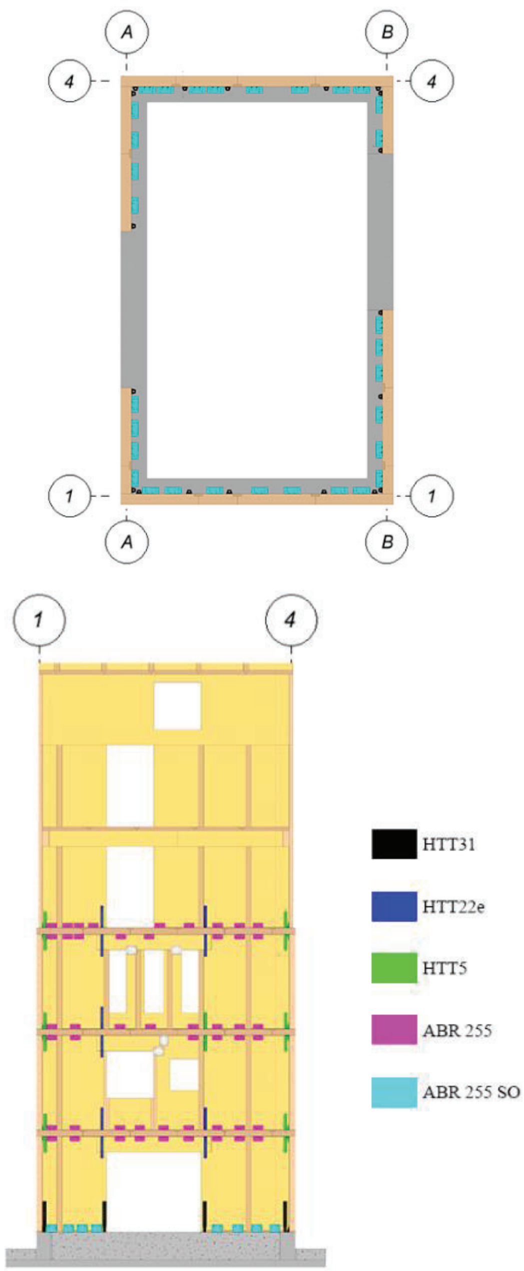

Figure 3: Mechanical connectors distribution at foundation level (top) and A-axis elevation (bottom)

3 INCREMENTAL DYNAMIC ANALYSIS APPROACH

Aiming the assessment of the safety level of the seismic design of the Pymelab Building, an incremental dynamic analysis (IDA) has been carried out. To perform this analysis, a robust and detailed model is required in which every deformation mechanism can be properly modeled. Given the model complexities and the large amount of nonlinear time history analyses involved in the IDA, OpenSees [11] models were developed using parallel computing techniques.

3.1 NUMERICAL MODEL DEVELOPMENT

The models consider linear and nonlinear components. Elastic orthotropic shell elements were employed for the CLT boards, and hysteretic nonlinear springs were assigned to the connection elements to simulate their nonlinear mechanical behavior. Moreover, a friction interface was used to represent the contact between walls, slabs, and the foundation. The proposed modeling approach is illustrated in Figure 4, while the mechanical properties of mechanical devices are presented in Table 5. Further details of the modeling process are available in Benedetti et al. (2023) [5].

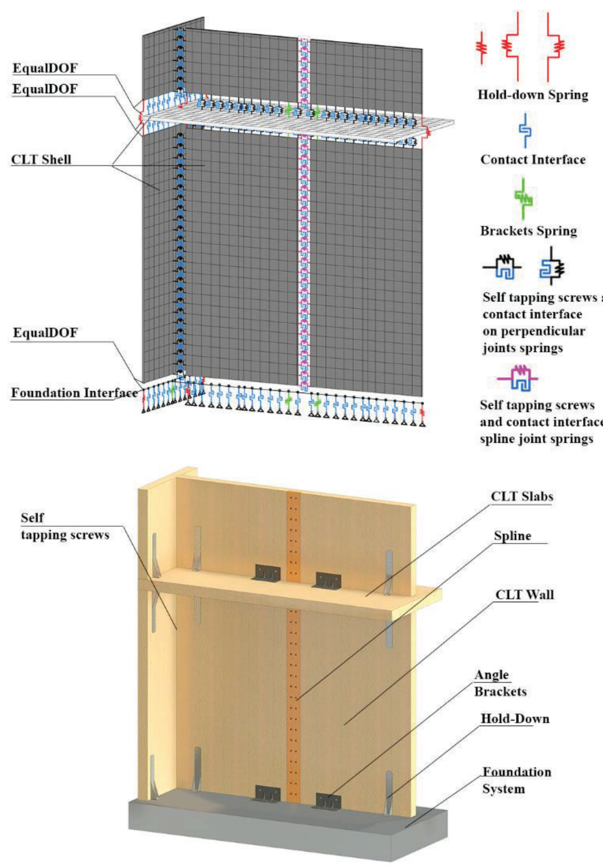

Figure 4: *Cross Laminated Timber (CLT) shear wall components (below) and model implementation (top) [5]*

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Table 5: *General mechanical properties of connections in the models*

Connection	Yield displacement Δ_y (mm)	Load capacity F_{max} (kN)	Elastic Stiffness K_e (kN/mm)
Hold Downs (tension)	8.9	72.3	5.88
Shear Brackets (shear)	6.1	110.1	15.15
Shear Brackets (tension)	7.2	88.19	9.83
Metal Plate(shear and tension)	2.4	174.8	42.73

In terms of the parallel modeling approach, several strategies have been developed to segment nonlinear computational domains [12]. However, this study focuses on a static decomposition approach due to its simplicity and straightforwardness. Moreover, the decomposition is conducted with the intent of preventing load imbalances between the processing cores.

Additionally, an iterative approach was employed to address the numerical integration and convergence issues caused by the high nonlinearity of the complex numerical model. An algorithm was used to dynamically and adaptively modify the nonlinear solver, time step (or displacement step), and convergence tolerance, if necessary in order to ease the solving of the analyses [5].

3.2 SEISMIC DEMANDS FOR THE IDA

A combination of Chilean earthquakes and other well-known seismic recordings from around the world were used in the IDA. The seismic response of the structural model was evaluated using 18 acceleration time series, including the horizontal components of the Mw 8.0 Algarrobo 1985, Mw 8.1 México 1985, Mw 6.9 Loma Prieta 1989, and Mw 8.8 Maule 2010 earthquakes. Figure 5 shows the response spectrum of the seismic demands considered, and Table 6 shows their main parameters. To perform the IDA, each seismic record is normalized and scaled according to FEMA recommendations [13].

Table 6: *Main pareameters of the seismic demands employed in the IDA*

Year	M_w	Station	Component	PGA (g)	PGV (m/s)
2010	8.8	CCP	L	0.40	0.67
			T	0.28	0.51
		CSP	EO	0.60	0.43
			NS	0.65	0.37
		Angol	EO	0.69	0.37
			NS	0.93	0.33
		Peñalolén	EO	0.29	0.22
			NS	0.29	0.29
		Valdivia	EO	0.13	0.18
			NS	0.09	0.13
1985	8.0	Llolleo	10	0.71	0.40
			100	0.44	0.23
1985	8.1	SCT	EO	0.16	0.58
			NS	0.09	0.35
1989	6.9	Treasure Island	0	0.09	0.15
			90	0.15	0.33
		Yerba buena	0	0.02	0.04
			90	0.06	0.14

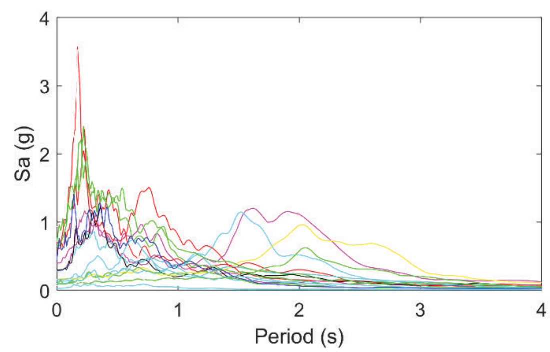

Figure 5: *Acceleration spectra of the seismic demands used in the Incremental Dynamic Analyses*

4 RESULTS AND DISCUSSION

4.1 MODEL VALIDATION

A comparison of the first mode period calculated with OpenSees detailed model with respect to the design model and ambient vibration measurements was performed to validate the model used for the IDA.

To determine the period of the first mode in the IDA numerical model, free vibration tests were conducted under initial conditions that excite the first mode, but are small enough to prevent the response from exceeding the system's yield. The results of the free vibration analysis are presented in Figure 6, showing periods of 0.32 s and 0.28 s for the X and Y directions, respectively.

Moreover, given that the Pymelab building is already built, vibration measurements and modal analysis were carried out with ARTEeMIS software [14]. With this software tool, fundamental vibration modes and

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frequencies were identified. Table 7 shows the periods obtained from the measurements, as well as the calculated with both implemented models.

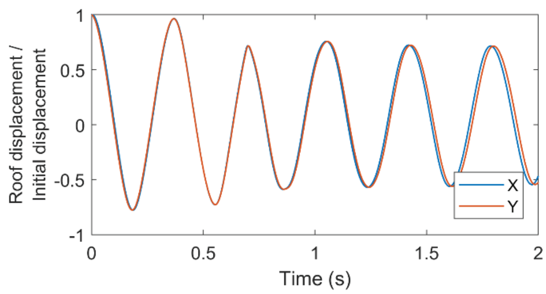

Figure 6: *Free vibration analysis of the IDA model for X and Y directions*

Table 7 periods suggest that the model used for the IDA is able to properly replicate the dominant dynamic response, with differences not larger than 7% with respect to the actual periods. Besides, the design model's and IDA model's fundamental mode periods match closely.

Table 7: *Period comparison among different models and experimental data.*

Model	Tx (s)	Ty (s)
Design model (SAP2000)	0.318	0.274
IDA Model (OpenSeeS)	0.32	0.28
Experimental Data	0.31	0.26

4.2 STATIC NONLINEAR ANALYSIS

Using the IDA OpenSees model, a nonlinear static analysis is performed to analyze the deformation capacity and the overall response of the structure in both directions. The result of the pushover analysis is shown in Figure 7.

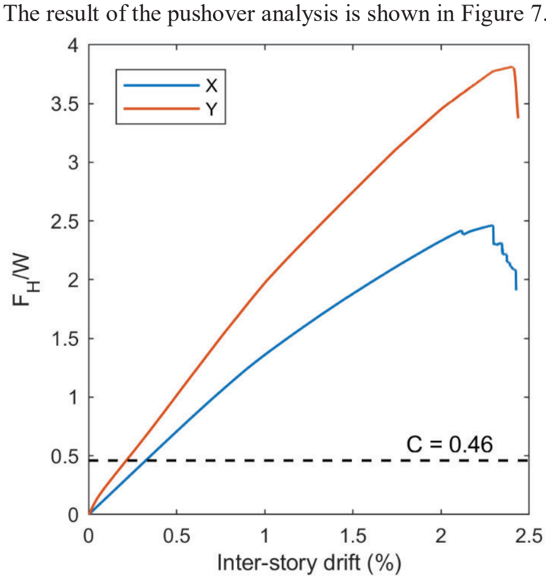

Figure 7: *Static nonlinear curves for X and Y directions*

The capacity curves obtained suggest that the global behavior of the structure for both principal directions is nearly linear during the whole drift range until de maximum capacity is achieved. However, around an inter-story drift of 1%, a break in the curve is observed, which is related to the system's yielding. The elastic stiffness calculated with the first slope of the curves are 155.9 kN/m and 252.1 kN/m for the X and Y directions, respectively.

Notwithstanding that the X-direction is more flexible and weaker than the Y-direction, both reached the maximum capacity and ultimate state at similar inter-story drifts (between 2.3% to 2.4%). Besides, the response of the structure seems to be brittle because of the abrupt drop in the lateral load capacity at the ultimate state.

Additionally, comparing the maximum lateral load capacity with the design seismic coefficient (C=0.46) and base shear, it is observed that the structure's overstrength factors (Ω) are 5.4 in the X-direction and 8.3 in the Y-direction (Table 8). Moreover, the system remains in the elastic range for this level of demand.

Table 8: *Maximum capacity (Fmax) and overstrength factor (Ω) in X and Y directions*

Design Base	X-Direction		Y-Direction	
Shear (kN)	Fmax(kN)	Ω	Fmax(kN)	Ω
142.6	763.2	5.4	1181.1	8.3

4.3 INCREMENTAL DYNAMIC ANALYSIS

For the IDA curves construction, a set of 540 nonlinear time-history analyses were performed. The structural damage index considered is the maximum inter-story drift demand, while the first mode elastic pseudo-acceleration spectral coordinate is used as the intensity index.

Using the results of the IDA, fragility curves are calculated for each direction considering four levels of structural damage, which are defined in terms of the inter-story drifts and set at 0.5% for Immediate Occupancy (IO), 1.0% for Operational (O),1.5% for Life Safety (LS), and 2.5% for Near Collapse (NC) (Figure 8). These inter-story drift limits are defined arbitrarily but consistent with the pushover results (Figure 7).

Fragility curves results show that the elastic pseudo-acceleration necessary to reach a 100% probability of achieving the NC state is larger than 7.5 g in and 9.0 g for the X and Y directions, respectively. On the other hand, in the range of elastic pseudo-accelerations between 1.5 g to 2.0 g it is achieved a 100% probability of reaching the IO level.

Moreover, the Collapse Margin Ratio (CMR) is evaluated for different collapse probabilities to determine the level of seismic safety. The CMR values are defined as the ratio between the elastic pseudo-acceleration required to achieve a certain NC state probability (S_{CT}) divided by the maximum expected earthquake demand (S_{MT}). According to Chilean codes (NCh 2745 [15]), S_{MT} demand level can be defined as the elastic design spectrum (S_E) amplified by 1.2 (S_{MT}=1.2S_D). Figure 9 presents the S_E and S_{MT}

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demand spectra, as well as the SMT acceleration coordinates obtained for each analysis direction.

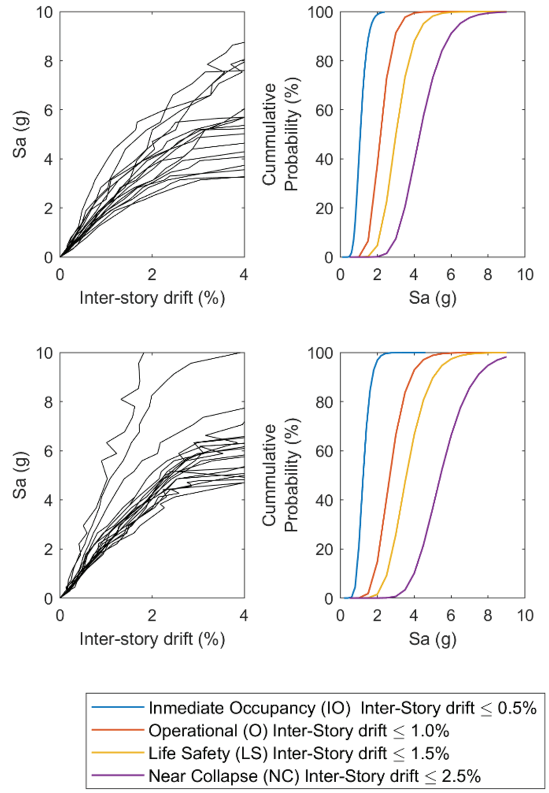

Figure 8: *IDA curves and fragility curves for X-direction (top) and Y-direction (bottom)*

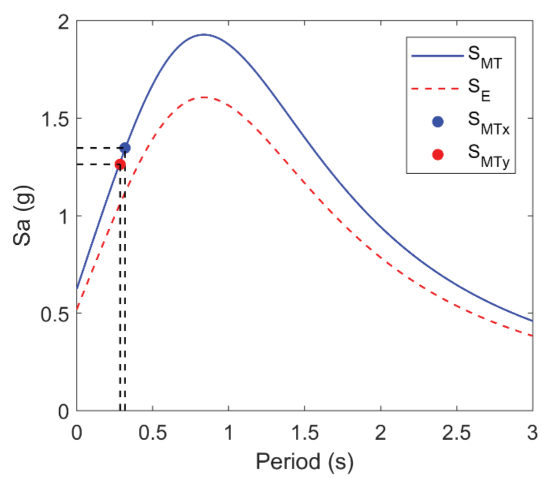

Figure 9: M*aximum considered earthquake spectrum (S_{MT}) and the elastic spectrum (S_a).*

Table 9 presents the obtained CMR values. It is observed that the Y-direction has higher collapse margin ratios due to lower S_{MT} demands and larger S_{CT} at all probability levels considered. However, for both directions, the CMR tends to be high, suggesting that the building's design is able to provide a significant safety margin against earthquake loads (CMR>3 for a 50% NC probability).

Table 9: *Collapse Margin Ratio (CMR) for different NC probabilities*

NC	X-Direction			Y-Direction		
probability	S_{CT}	S_{MT}	CMR	S_{CT}	S_{MT}	CMR
25%	3.63	1.34	2.71	4.62	1.26	3.67
50%	4.30	1.34	3.20	5.45	1.26	4.32
75%	5.05	1.34	3.76	6.46	1.26	5.13

4.4 INTER STORY DRIFT VS ROOF DRIFT RELATIONSHIP

Using the IDA analysis results, it is possible to establish the relationship between the maximum roof drift and maximum inter-story drift. Figure 9 shows the relation obtained and the linear fits for each direction.

The results of Figure 9 show a strong correlation between the maximum roof drift and inter-story drift for the Pymelab building. The linear fits point out that the slope angle is 42° and 36° for the X and Y directions, respectively. These angles suggest that the inter-story drift distribution among the building stories tends to be more uniform X-direction, hinting that the Y-direction could be more prone to concentrated story deformations and eventual soft story mechanisms. Furthermore, linear fit slope angles are close to Gohbarah (2014) [16] findings for reinforced concrete buildings.

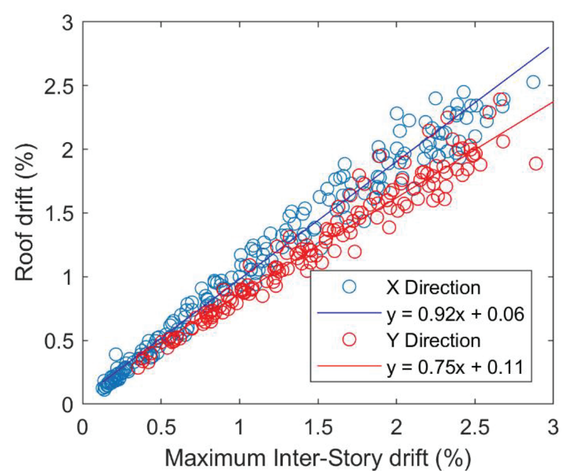

Figure 9: *Relationship between maximum inter-story drift and maximum roof drift of the Pymelab building*

5 CONCLUSIONS

This research analyzes the seismic response and safety of a CLT building designed through traditional force-based methods under seismic codes that do not recognize the intrinsic wooden building characteristics. Complementary, aiming to achieve better earthquake performance, capacity-based design principles were also considered.

Results appear to show that the building is able to provide significant lateral load strength, but its deformation capacity can be limited. This fact is based on the large overstrength values (Ω >5) and the low displacement ductility ratios (μ<2.5) obtained. The latter might be a consequence of the large seismic force demand considered in the design due to the small load reduction factor (R=2) established by the Chilean code for CLT structures.

Regarding the seismic safety, fragility curves show that the probability of achieving a Near Collapse state is close to 0% for the maximum expected Chilean seismic demand (elastic pseudo-acceleration of 1.36 g). In addition, collapse margin ratio values suggest that the safety margin of the Pymelab Building against earthquake demands is appropriate.

Finally, results suggest that current Chilean force-based design regulations promote safe CLT structures, but it appears that there is still a wide room for the optimization of the designs. Nevertheless, capacity-based design principles should complement the design process in order to promote a suitable failure mode.

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