CASE STUDY: SEISMIC AND GRAVITATIONAL DESIGN OF 15-STORY OFFICE AND RESIDENTIAL BUILDING ARCHETYPES WITH A SEMI-RIGID CLT DIAPHRAGM AND REINFORCED CONCRETE SHEAR WALLS IN CHILE

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ABSTRACT: This study is part of the "Ciudad Madera" technological consortium, which seeks to promote widespread wood construction in Chile, whose seismic code provides no guidance for cross-laminated-timber (CLT) seismic diaphragms. Currently, this lack of a design methodology for Hybrid Mass-Timber–Reinforced-Concrete (HMT-RC) buildings, has pushed a tendency for structural designers to solely rely on the concrete topping as the seismic horizontal diaphragm element. This study validates HMT – RC and develops a seismic-design method for buildings up to 15 stories high, with CLT floor system performing as the seismic diaphragm. Twenty monotonic/cyclic connector tests and six full-scale 4 m × 4 m Radiata-pine CLT diaphragm tests will calibrate nonlinear models of office and residential building archetypes, analyzed in a 72-case parametric matrix (height, seismic zone, soil class, design philosophy). Findings show: (i) RC-core and coupling-beam detailing governs drift, (ii) flooring size is controlled by a vibration criterion, and (iii) commercial timber column-to-column splices meet seismic inter-story rotation demands.

KEYWORDS: Semi-rigid CLT Diaphragm, Hybrid mass timber buildings, High seismic zone, Experimental testing.

1 – INTRODUCTION

During the last decades, building energy consumption has become consistently more efficient, and as the day-to-day use of energy decreases, attention is now turned to carbon in pre-existing life-cycle stages, specifically raw-material extraction, construction on site, and final end-of-life processing. Traditional construction materials such as steel and concrete dominate worldwide demand for manufactured materials, while at the same time, they also rank among the most energy- and carbon-intensive to produce [1]. D'Amico [1] has quantified that replacing concrete floor slabs for Cross Laminated Timber panels (CLT) in steel buildings for the next 30 years could avoid over 50 Mt CO2 of emissions on average.

To further extend the incentive of more sustainable construction in Chile, "*Ciudad Madera*" Technology Consortium was developed by the real estate company Territoria with the support of CORFO, and the National Center of Excellence for the Wood Industry (CENAMAD), which aims to promote widespread timber construction in Chile. One of the 11 Consortium research projects, the P08, is focused on validating the structural behaviour of Hybrid Mass-Timber Reinforced-Concrete (HMT–RC) buildings of up to 15 stories high, while also establishing a design procedure for these buildings in Chile.

Most mid- to high-rise buildings in Chile are RC buildings, making for 47% of the total net surface constructed between 2019 and 2023 [2]. Furthermore, between 2019 and 2021, over 98% of all authorized building permits for residential buildings over 6 stories high were RC buildings [3].

The country's seismic design standard, NCh 433 [4], mirrors the long-standing reinforced-concrete practice, presuming a mostly rigid diaphragm and a lateral-force-resisting system—both the horizontal

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diaphragm and the vertical elements—made from a single uniform material. Consequently, the NCh 433 [4] code provides no guidance for CLT horizontal diaphragms, overlooking ductility capacity, overstrength, and the seismic demands that panels and their connections must resist. This gap has not halted HMT–RC construction, but it has largely kept projects to low-rise buildings; when greater heights are attempted, Chilean structural engineers typically default to an RC topping slab for the diaphragm action, leaving the underlying CLT panels to carry gravity loads only.

The panelized diaphragm issues have been previously studied for precast concrete. After the 1994 Northridge earthquake, several precast concrete structure floors had failed, forcing several code changes which loads for diaphragm forces to be transferred by a topping concrete slab, leaving precast concrete elements working only as part of the gravitational system [5]. In response to these standard modifications, Fleischman [5] developed a study in which he does from connection and full-scale shake table experimental testing, non-linear detailed modelling and finally proposing a design methodology for precast concrete diaphragms.

This P08 project, "Ciudad Madera" Consortium research project, which is an ongoing project, will be partially based on the Fleischman's precast concrete diaphragm study [5]. The research project aims to fill the current gaps in the national standards and to generate a validated design methodology for HMT–RC buildings in Chile. The research project counts with 20 experimental connection tests, including monotonic and cyclic tests of the inter-panel connections, chords and collectors. There will also be two full-scale specimen diaphragm test campaigns studying different variables, 1 monotonic and 2 cyclic for each specimen, accounting for 6 diaphragms total experimental tests. All results will afterwards be used to calibrate non-linear models and with these results, the research team will be able to deeply understand how the different design methods and connection configurations will affect the building's diaphragm and dynamic behaviour. Finally, all the findings will be used to propose a simplified design methodology of HMT–RC buildings in Chile.

This paper describes procedures and the main results and conclusions from the initial phase of the ongoing project. The information collected in this phase will guide the development of the design methodology, as well as the testing, nonlinear modelling, and analysis in the following stages of the project, all aimed at validating the seismic performance of HMT–RC buildings.

2 – ARCHITECTURE LAYOUT AND DEFINITIONS

To promote timber construction in Chile, the project's architectural layouts had to reflect typical Chilean residential and office buildings. An independent architectural firm was therefore commissioned to design both configurations (Fig. 1). The office building adopts a 6 m × 8 m column grid, while the residential building uses a 3 m × 3 m square grid. Both floor plans incorporate a reinforced concrete (RC) central core and an open-plan layout, offering a high degree of architectural flexibility.

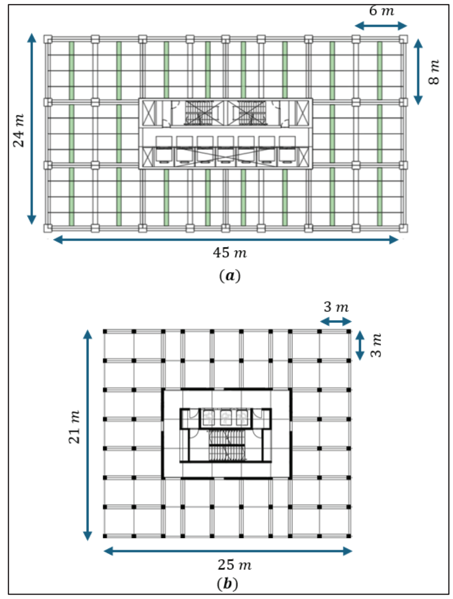

Figure 1. (a) Office building layout. (b) Residential building layout

The project has the intention of reducing the use of concrete as much as possible. As a result, the office building flooring does not use RC topping (raised floor system instead). However, due to requirements of thermal inertia, the residential building does use a 40 mm non-collaborating topping.

2.1 STUDIED VARIABLES AND BUILDING CASES

All information gathered during the project's first phase – including the experimental data – will be used to calibrate and validate detailed non-linear models. These models will then be run across a matrix of 72 scenarios created by varying five parameters: building archetype (residential or office); seismic zone (2 and 3); soil type profile (B, C, or D); number of stories (8 and 15), and design philosophy [Elastic Design Option (EDO), Basic Design Option (BDO) or Reduced Design Option (RDO)]. The resulting dataset will both, highlight the design methodology that offers the best overall performance, and provide a reference library of structural solutions that engineers and architects can expect when planning HMT–RC buildings in Chile.

3 – CONTEXT OF REGULATORY CONSIDERATIONS

This research project is intended to establish a clear design pathway for HMT–RC buildings in Chile, so every structural verification is performed under the pertinent Chilean codes: seismic actions are evaluated with NCh 433 [4], glulam grading with NCh 2165 [6], general timber design with NCh 1198 [7], and the RC

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podium and core shear walls design with NCh 430 [8], while the emerging draft standard prNCh 3743 [9] governs CLT elements. Because Chile's dedicated fire-design standard is still in development, fire resistance of exposed timber members is verified instead against Eurocode5 part 2 (EN1995-1-2) [10]. Horizontal structural components—glulam beams and CLT floor panels—are assessed not only for strength, deflection, and fire performance but also for footfall-induced vibrations; due to the absence of a national regulatory guidelines, this vibration check follows the draft Eurocode5 (prEN1995-1-1) [11] provisions.

3.1 CHILEAN SEISMIC BACKGROUND

Chile is in a high seismicity region. Although non-linear models provide a rigorous solution, they require large amounts of time and computing power. Chilean seismic design provisions, therefore, allow for linear models using a pseudo-acceleration spectrum to estimate equivalent static seismic forces. The base elastic spectrum is established using historical seismic information. However, because the model is elastic, the spectrum is reduced by a structural response modification factor "R". Such a value is determined depending on the lateral force resisting system (LFRS) and its capacity to dissipate energy while maintaining structural integrity.

In terms of lateral displacements, Chilean seismic regulation places two basic requirements. The first is that the maximum allowable relative displacement of story drift at Center of Mass (CM) is limited to 0.002 times the floor height. The second requirement deals with torsional deformations by requiring that at any point along the diaphragm, the drift should never exceed CM drift by an additional 0.001 times the floor height. The aforementioned restrictions are implemented in part to control the structural integrity of buildings, but they're mainly implemented to control the damage of non-structural elements.

Currently, the NCh 433 [4] does not take into consideration the use of different materiality between the LFRS and the diaphragm, so it considers a unique R for both. Since there are no design provisions for the Mass Timber (MT) seismic diaphragm in Chile, structural engineers do not assign seismic responsibility to timber floors elements. The technical and financial viability of HMT–RC buildings is, therefore, dependent on experimental testing and development of proper design methodologies that capture MT diaphragm behaviour and its interaction with RC shear-walls.

4 – METHODOLOGY

4.1 PHASE I: ARCHITECTURAL LAYOUT DEFINITION

Building-layout choices—how many RC cores to use, where to place them with respect to each other and to the façade, and even the cores' shape—strongly govern seismic behaviour. The initial scheme followed Gallegos et al. 24 m × 40 m prototype [12], pairing a single 4.2 m × 16 m central core with perimeter moment frames. Gallegos' all-RC version performed well, so it offered a logical benchmark for a MT hybrid building.

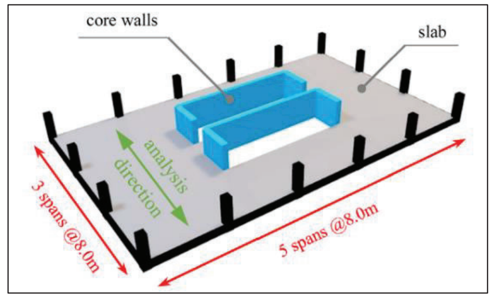

Figure 2. Gallegos et al. analysis building. [12]

In the case of HMT–RC considering the buildings configurations shown in Fig. 1, the analysis with lone core could not control torsional displacements at the corners; acceptable performance would have demanded corner RC shear walls or another perimeter LFRS – an option the project's architect's consultant dismissed. Working jointly with architectural consultants, the engineers therefore proposed a second layout that relocates and couples the cores with a robust coupling beam (Fig 1). This configuration allows the core halves to act together, resolves the previous drift issues, and preserves the desired architectural expression.

4.2 PHASE II: GRAVITATIONAL SYSTEM DESIGN AND DIMENSIONING

All gravitational elements were designed to fulfill the strength requirements under standard ASD tension combinations, considering complete dead and live loads. Fire requirements which were verified using the EN1995-1-2 [10] effective section method, considering 100% dead load and 30% of the live loads.

4.2.1 FLOOR FOOTFALL VIBRATION BACKGROUND

Timber flooring systems tend to require greater consideration in relation to vibration, primarily because of their inherent light weight [13]. In response to this, current international design codes now have straightforward verification procedures requiring minimum out-of-plane stiffness for panels and beams (CSA O86 [14]). The prEN1995-1-1 [11] followed this by supplementing such provisions using a set of manual calculations checking for panel frequency, stiffness, and impact velocity and acceleration. The draft's annex prEN1995-1-1 [11] further includes an approach for modal analysis borrowed from procedures found in *"A Design Guide for Footfall Induced Vibration of Structures"* (CCIP-016) [15], a vibration analysis guide developed primarily for traditional materials in structures.

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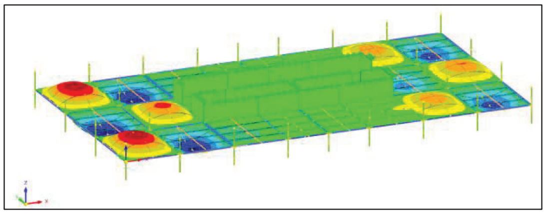

Figure 3. RFEM 6 modal analysis displacement contour results for the first vertical mode (f = 10.37 Hz)

To ensure occupant comfort, the glulam beams and CLT floor panels were not only verified to satisfy strength, deflection and fire requirements, but were also designed to achieve level III vibration floor performance according to prEN1995-1-1 [11]. Level III corresponds to the minimum floor vibration performance suitable for a high-quality choice office building.

Floor vibration design was conducted according to the modal analysis method described in Annex G of prEN1995-1-1 [11]. The FEM model was developed in RFEM 6 in accordance with the guidelines in the Wood Works Mass Timber Floor Vibration Design Guide [16].

After strength, fire-resistance, and vibration checks, the office building with a grid of 8 m × 6 m was finalized with 700 mm × 700 mm glulam columns, primary beams of 600 mm × 550 mm, secondary beams – highlighted in green in Fig. 1 – of 500 mm × 550 mm, and 200 mm-thick, five-ply CLT panels composed of 40 mm laminations. The residential building with a grid of 3 m × 3 m required 40 cm × 40 cm glulam columns, 30 cm × 30 cm beams, and 180 mm-thick, five-ply CLT panels made from 40 mm longitudinal and 30 mm transverse laminations, topped with a non-composite 40 mm reinforced-concrete slab for thermal and acoustic performance. In both projects, beam and CLT dimensions were governed by vibration criteria.

4.2.2 COLUMN – TO – COLUMN SPLICE CONNECTION ANALYSIS AND VERIFICATION

A topic seldom highlighted is that gravity-only vertical elements—such as columns—are still tied to the floor diaphragms and must deform compatibly when seismic horizontal displacements occur. Given that Chilean buildings will face several significant earthquakes over their lifespans, it is essential to verify the resilience of every component, including columns that do not participate in the lateral-force-resisting system.

Column resilience during an earthquake mainly depends on the splice connection. These joints must supply enough relative rotational capacity (see Fig. 4) for the columns to follow diaphragm displacements without inelastic deformations, ensuring that gravity-only elements remain mainly elastic even when they are dragged along by seismic drift.

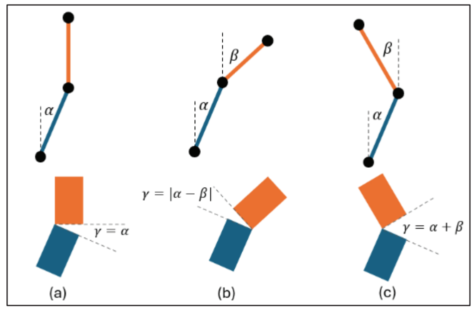

Figure 4. Different types of column relative rotations and their influence on splice deformation.

One of the end results of the project is obtaining the columns' rotation demand from a non-linear detailed model and fully verifying the column rotation requirement. But for a first estimation, a modal analysis was done on an elastic ETABS [17] model which incorporates the reduced stiffness of cracked RC shear walls. The applied seismic spectrum corresponds to the displacement spectrum originally proposed by NCh 433 [4] to verify the required RC shear wall confinement. The spectrum is calibrated as to generate the maximum expected roof displacement, so, even though it is not directly intended for column displacement analysis, it should still give a trustworthy first estimation.

Considering the column splices as hinges with no rotational stiffness, trying to be the most conservative when it comes to the column deformation, the maximum relative rotation that the two consecutive columns endured for the modal analysis was 0.0049 radians. This rotation is within the linear elastic deformation capacity of market connectors, such as the MCT Simpson Strong Tie connector device.

4.3 PHASE III: LATERAL FORCE RESISTING SYSTEM DESIGN

The verification of the seismic provisions and the estimation of the diaphragm forces and deformations is first done using a linear elastic model proposed by Veliz et al. [18], which is based on Moroder approach [19]. This model uses a combination of link and frame elements to capture the in-plane deformation in accordance with the SDPWS (see Fig. 5), as well as the out-of-plane deformation, according to the Shear Analogy Method. This model allowed the engineering team to understand the diaphragm's load distribution (both lateral and gravitational) and determine the chord straps, inter-panel connectors and collector loads.

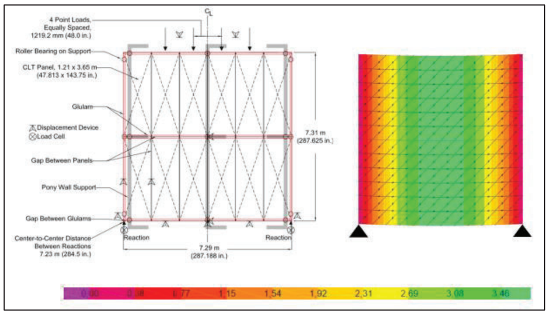

Figure 5. Scheme of CLT diaphragm carried out by Line et al [21] modeled with Veliz et al [18] methodology.

The choice of diaphragm connectors is intended to be followed according to two different design philosophies. On one hand, the SDPWS 2021 [20] approach allows the diaphragm to dissipate seismic energy through its inter-panel shear connectors. Collectors and chords are therefore sized with sufficient over strength to remain virtually elastic. Such approach must be carried out using reduced spectrum to get seismic forces, i.e., using an "R" value to reduce the elastic forces (RDO), obtaining design forces; on the other hand, the elastic methodology (EDO) – which directly employs the elastic spectrum to get the seismic forces – prescribes a fully elastic diaphragm: every component, including the connectors, must stay within its elastic range, and no deliberate energy dissipation is permitted. Finally, an intermediate option between both methods will be defined (BDO), which aims to be less restrictive than the EDO but also to be less flexible than the RDO.

At the current stage of this research project, the diaphragms are composed of Simpson ST LDSS48 as the inter-panel shear connectors, SDCP22100 screws spaced at 10 cm o.c. as the inter-panel shear connector over glulam beams. Chords are composed of double-plated Simpson ST MDCST48 and 6 mm thick collectors, arranged with SDS25312 screws. The properties of the link and frame elements are then calculated accordingly.

4.3.2 COUPLING BEAM INFLUENCE

During the initial stage of the project, in the absence of fully developed structural buildings archetypes (Fig. 1), the structure shown in Fig. 2 was selected as the reference case for initial study. In this preliminary phase, the engineering research team identified the coupling beams between RC cores as critical structural components for controlling the building's global torsional behavior. The high demand on these beams concluded with the necessity of additional RC perimeter walls to mitigate torsional displacements, as illustrated in Fig. 6.

Subsequently, the proper modeling of the coupling beams in the archetypes illustrated in Fig. 1 enabled effective control of torsional drifts without the need for RC perimeter walls, as depicted in Fig. 6.

It is important to note that drifts shown in Fig. 7 are capturing the stiffness differences in the lower stories, where the use of RC podiums has been considered.

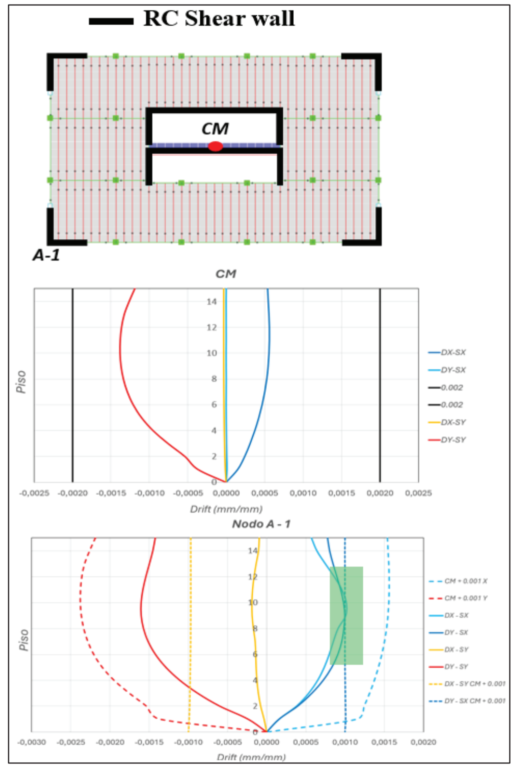

Figure 6. Torsional drifts of preliminary building (Fig. 2).

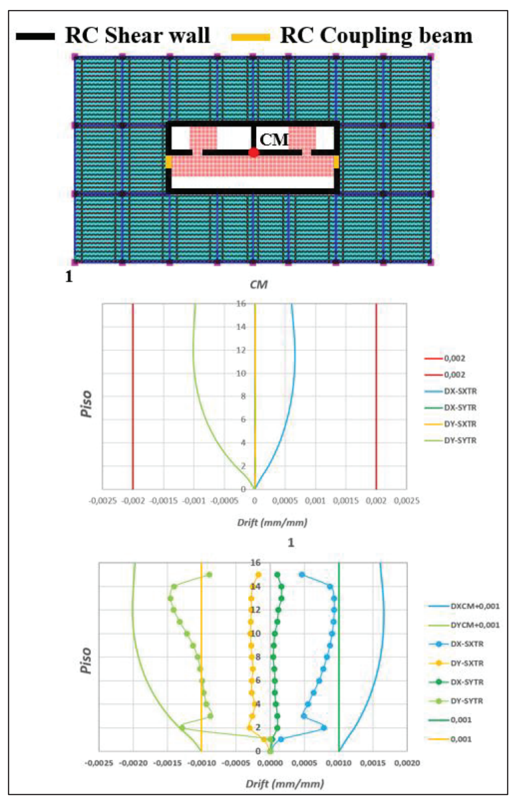

Figure 7. Torsional drifts for Office building (Fig. 1(a)).

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4.3.1 DIAPHRAGM DESIGN METHOD INFLUENCE

As mentioned previously, the timber and steel elements within the CLT diaphragms are highly influenced by the adopted design philosophy, which includes elastic design (EDO), basic design (BDO) and design according to SDPWS [20], also known as RDO in the present research. In this study, the criteria employed to design diaphragm elements include inter-panel shear connectors, tension collectors, shear collectors (MT-to-RC walls connections), chord straps, glulam timber columns, and drift.

A FEM-modeled-buildings matrix is currently under development, in which both the office and residential structures models are subjected to different seismic contexts by varying the number of stories, seismic zone, and soil type. Hence, 72 possible building cases have been identified (for instance, see Fig. 8). Each case must be designed (based on a linear elastic model) to verify compliance with the SDPWS [20] requirements (RDO), BDO or the elastic method (EDO), as appropriate.

The most likely scenario is that after completing the 72 iterations, some cases will not meet all structural criteria and will therefore be excluded from the study matrix. This process will help establish recommended construction limits for HMT–RC buildings in Chile. Meanwhile, buildings that meet the structural criteria will proceed to the second stage of nonlinear modeling, using the method introduced in Chapter 6.

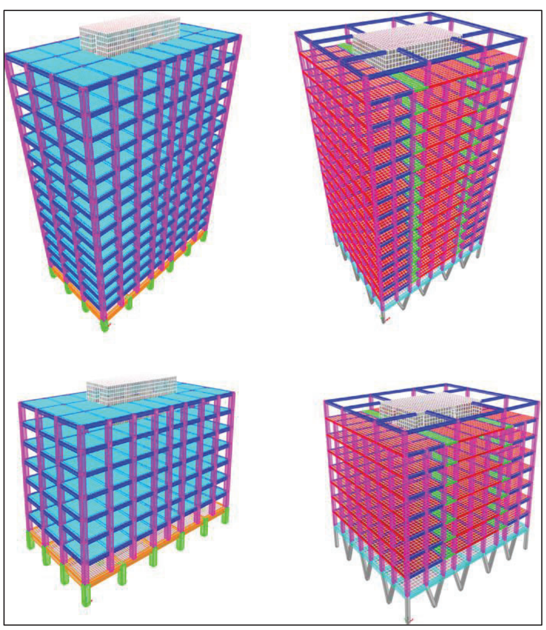

Figure 8. Isometric view of 8 and 15 story ETABS models of Office (left) and Residential (right) buildings.

5 – EXPERIMENTAL CAMPAIGN TESTS

An important objective of this study is the validation of seismic behaviour of CLT MT diaphragm and its components when using Chile's Radiata Pine wood. Such validation will be carried out using a non-linear model (see Chapter 6), which is already being fed by experimental data from small-scale connection tests (see Fig. 9) and is intended to be calibrated with the data from full-scale tests, yet to be performed.

The experimental campaign is being driven in two main steps: (1) monotonic and cyclic testing of individual connectors, depicted in Table 1, and (2) there will be two full-scale specimen tests of 4m × 4m cantilevered CLT diaphragms, each specimen tested to 1 monotonic and 2 cyclic as shown in Fig. 10.

Once the non-linear model from the test shown in Fig. 10 is calibrated against experimental data, a sensitivity analysis will be carried out to completely validate the methodology. Afterwards, both the residential and office buildings will be modeled, and parametric analysis will be performed, considering different soil types, seismic zones, among others.

This stage is planned to initially include 72 study cases, however, some cases are expected to be excluded from the analysis, as their behavior within the linear elastic range – according to the previously presented elastic model – might not be suitable enough for design purposes. Nevertheless, those cases within the matrix that prove adequate for elastic design levels will subsequently be analyzed in the nonlinear range, to assess the relevance of the overstrength factors commonly employed [20].

Table 1: Diaphragm connector experimental testing matrix

Experimental Testing		Load Orientation	
		0°	90° CLT
Inter-panel shear	Monotonic	-	2
	Cyclic	4	3
Chord Tension	Monotonic	-	1
	Cyclic	3	3
Collector Tension	Monotonic	-	1
	Cyclic	-	3

Figure 9. Testing of inter-panel and chord straps connectors.

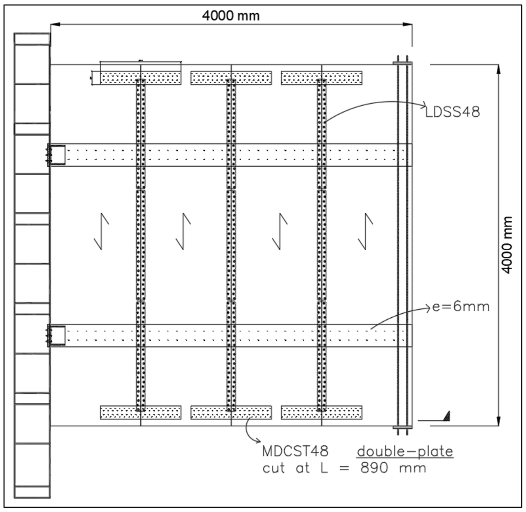

Figure 10. Full-scale diaphragm specimen testing setup.

6 – NON-LINEAR DETAILED MODEL

The buildings MT diaphragm will be modelled with a 2D nonlinear finite element model in ANSYS [22] developed by M. Chacón and P. Guindos [23]. At the current stage of the research project, the model is being applied in simulating the upcoming CLT joints experimental test (see Fig. 11).

The following modeling assumptions are considered in the model: (i) CLT panels are represented using layered-shell elements (SHELL181), featuring six Degrees of Freedom (DOF) per node, where each layer was oriented according to the parallel-to-grain direction of the lamella and exhibits linear-elastic orthotropic behavior; (ii) steel plates for collectors and chord straps were modeled as beam elements (BEAM188) with three DOFs per node, following the von Mises model with kinematic bilinear hardening; (iii) Panel-to-Panel (P2P) connections were simulated with four uniaxial springs (USER300) representing a specific number of fasteners. Two orthogonal elements characterize the behavior of the fasteners (i.e., one represents the shear direction using the ASPID phenomenological-based hysteretic timber model [24] and the other handles the axial direction with a von Mises model featuring bi-lineal hardening) and two orthogonal elements model the contact between panels, utilizing a normal contact for gap-closure that exhibits a smooth exponential relationship – being stiffer and stronger in compression than in tension – and tangential contact for friction exhibiting linear behavior with reduced stiffness; (iv) the Panel-to-Collector (P2C), Panel-to-chord Strap (P2S), and Panel-to-fixation Angle (P2A) connections were represented with two orthogonal zero-length springs for a specific number of fasteners, both employing a von Mises model with bilinear hardening; (v) a compatible mesh has been implemented across all elements, with a maximum element size of 100~mm and an edge size proportional to fastener's spacing in each connection; and (vi) to reduce the number of elements and computation time, each spring models a specific number of fasteners. The zero-length springs were represented by connecting a uniaxial spring (USER300) and an auxiliary rigid beam (MPC184) in parallel (see the detail of P2C in Fig. 11).

The material parameters of the springs related to the shear behavior of the P2P connections have been validated through in-plane cyclic shear tests conducted by Chacón et al. [22]. These tests utilized the same type of fastener, steel spline, and CLT wood class as those considered in this diaphragm test. For the springs in the P2C, P2S, and P2A joints, the yield stress σ_y is proportional to the nominal stress of each connection. In the absence of experimental data with other steel plate thickness, these values were derived from the admissible strength $F_{V,ASD}$ as specified by Johansen's theory [25] and the EN1995-1-1 [25] and further using an overstrength factor Ω_{SC}, i.e., $\sigma_y = \Omega_{SC}F_{V,ASD}/A_f$, where A_f is the fastener diameter. Additionally, the Young Modulus E_0 of these connections was related to the elastic slip modulus K_s following the EN1995-1-1 [26] code, i.e., $E_0 = K_s l_f/A_f$, where l_f is the simulated length of the fastener.

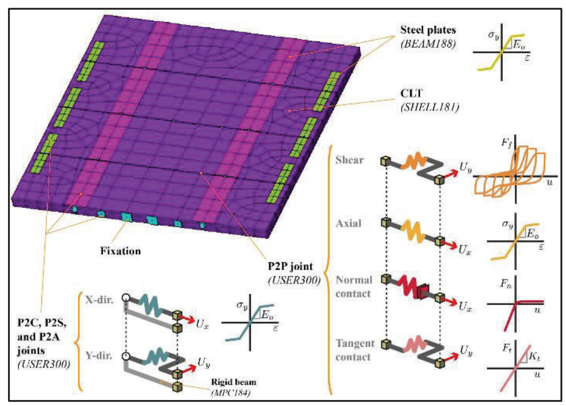

Figure 11. Non-linear detailed ASPID model depiction

7 – CONCLUSIONS

The first relevant conclusion found during the first phase of the research project is the importance of designing a healthy RC core configuration. This decision has an important influence in the viability of an HMT–RC building in Chile. In those buildings, the RC coupling beam has an important role in assuring the building has adequate seismic performance.

The second key conclusion is the validation of commercially available Simpson Strong Tie steel connectors for CLT diaphragms as adequate to withstand lateral loads within the Chilean seismic context. Moreover, the commercial Simpson Strong Tie timber column-to-column splices meet seismic inter-story rotation demands.

The subsequent phases of the project involve 1) finishing the experimental characterization of connections and diaphragms specimens, 2) the nonlinear modelling of both residential and office building configurations archetypes, 3) the evaluation of the building's seismic

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performance through nonlinear time history analysis, and 4) validate the proposed seismic design philosophy procedure for cross-laminated-timber floor system performing as the seismic horizontal diaphragm.

8 – ACKNOWLEGDEMENT

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