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SEISMIC RESILIENT MASS TIMBER STRUCTURES USING INNOVATIVE CONNECTIONS: LATEST RESEARCH, DESIGN METHODS AND CASE STUDIES

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ABSTRACT

There is increasing public pressure to have seismic-resistant structures made with massive wooden panels such as Cross Laminated Timber (CLT). It is a well-known fact that the type and behaviour of the connections govern the overall seismic behaviour of the system. Thus, special attention must be paid to the connections when designing such buildings under lateral loads. Previous experimental studies showed that although CLT structures with conventional seismic detailing could survive design-level earthquakes, the extent of damage in connectors could be severe. Therefore, it is necessary to have resilient connection systems if a low-damage performance is desired.

This paper presents the latest research, testing and developments about the application of innovative and resilient energy dissipative connectors in mass timber structures. Results of large-scale testing are presented and discussed. This paper also presents case studies of low-damage mass timber structures where innovative and resilient connections are used instead of conventional high-damage/pinching connectors to firstly introduce energy dissipation to the structures (without damage) and, secondly, provide a self-centring behaviour. Three different case studies in Canada are presented and discussed. The case studies include the Yukon building (4-storey mass timber building), the ON5 building (three-storey mass timber building with rocking CLT core) and the Keith Drive Building (10-storey mass timber structure) in Vancouver, Canada. The design approach, challenges, and aspects of erection/construction are discussed. Furthermore, the significant advantages of low-damage mass timber buildings over conventional buildings are discussed. The findings of this paper demonstrate great potential for low-damage mass timber structures in high seismicity areas such as BC.

Key words: Mass timber, Resilience, Energy dissipation, Damping, Low damage

INTRODUCTION

In recent years, mass timber elements have widely been used for different types of timber buildings such as offices, commercial buildings, public buildings and multi-story residential complexes. For conventional mass timber structures, traditional steel connectors with dowel-type fasteners such as nails and screws are extensively being used. Despite the acceptable seismic performance of these structures in seismically active regions, the permanent inelastic deformation of the steel brackets under cyclic loading has made them vulnerable to aftershocks and/or future events. One of most extensive experimental researches about the seismic performance of mass timber structures to date has been conducted within the SOFIE project [1]. That project included quasi-static and shake table tests on different types of CLT buildings. The results confirmed that the CLT structures with traditional connections are relatively stiff and could survive the design level events (although significant damage was observed in some connections). Some of the steel connections (such as nailed hold-downs and nailed shear brackets) yielded in bending and some withdrew from the timber elements. More importantly, high response accelerations (mainly in the upper levels) with a maximum of 3.8 g were recordedPopovski et al. investigated the seismic response of the CLT wall panels with various arrangements and connection layouts for application in mass timber structures [2], [3]. The data would seem to suggest that these walls have satisfactory lateral resistance when nails or slender screws are used together with steel brackets. Moreover, the use of nailed hold-downs at the corners of the walls was proven to improve the resistance to overturning from the lateral forces. That can be attributed to the increased moment lever arm for the wall panels. Gavric et al. experimentally investigated the cyclic behaviour of single and coupled CLT walls with different connections [4], [5]. The test results suggested that the layout and design of the connections govern the overall behaviour of the wall.

Popovski et al. conducted a series of full-scale quasi-static tests on a two-story CLT platform house [6]. No global instability was observed not even at the maximum design lateral force. Regardless of the rigid connections between the floors and walls, rocking movement of the wall panels was not totally restricted by the floor above. Yasumura et al. studied the mechanical performance of low-rise CLT platform buildings with large and small panels subjected to reversed cyclic lateral loads [7]. It was concluded that in the buildings with small panels, the rotation of the panels was the major cause of the total deformation of the building. They also proposed a numerical model to predict the seismic behaviour of such structures based on the connectors used. Blomgren et al. [8] performed a series of shake table tests on a full-scale, two-story mass timber building with rocking CLT shear walls. The shear walls were equipped with sacrificial components (fuses). The test results showed that the system could meet the design performance indexes and the damage was limited to the designated components that could quickly be replaced after the event.

The use of friction devices for mitigating the seismic damage dates back to 1980s where Pall et al. [9]–[11] introduced them for the application in reinforced concrete panels and steel braced frames. Later on, Popov et al. [12] and Clifton et al. [13] proposed the use of friction connections for steel moment-resisting frames. For timber structures, Filiatrault [14] investigated the application of friction damping devices at the corners of traditional sheathed timber shear walls that demonstrated a promising improvement in the hysteretic behaviour of the wall system. Their experiments showed a significantly enhanced seismic performance compared to traditional systems in terms of stability and hysteretic damping. Hashemi et al. [15], [16] expanded the concept of slip friction connections to the CLT coupled walls and hybrid timber-steel core walls. It was shown that despite the reliable low damage seismic performance of the proposed systems, an additional mechanism and/or vertical gravity loads is required to provide a self-centering behaviour.

The paper presents the implementation of innovative friction-based resilient conceptions instead of conventional timber connectors for low damage design of mass timber structures. The technology used is briefly described and the details of the implementation are outlined. This paper provides an insight to engineers, designers and different parties in the construction industry about the journey ahead to adopt a novel solution for mass timber structures.

THE RESILIENT SLIP FRICTION JOINT (RSFJ)

The Resilient Slip Friction Joint (RSFJ) technology was invented by Zarnani and Quenneville [17]. This device is a friction device that can dissipate the seismic energy and provide a re-centring behaviour with a flag-shaped hysteresis. Figure 1 depicts the configuration and the load-deformation relationship of the RSFJ. Similar to conventional friction joints, the RSFJ dissipates energy via sliding movement of the clamped plates. Moreover, on account of the special profiled shape of the sliding plates (grooved plates), the elastic energy conserved in the semi-pressed disc springs allows the plates to return to their original position without depending on any external mechanism.

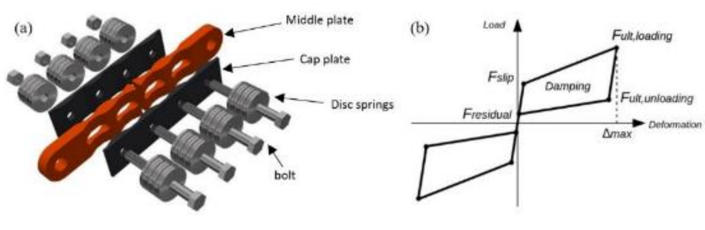

Figure 1. Resilient Slip Friction Joint (RSFJ): (a) configuration (b) hysteretic behavior

Figure 1(a) shows the assembly and different components of a RSFJ specimen. The hysteretic parameters (F_{slip}, $F_{ult,loading}$, $F_{ult,unloading}$, $F_{residual}$ and Δ_{max}) shown in the figure can be determined in accordance with the design requirements. In other words, almost any desired load-slip response can be designed by tuning the different variable parameters of the RSFJ such as the angle of ridges, number of bolts and stiffness of the disc springs. The behaviour and performance of the RSFJ has previously been verified via joint component tests and large-scale experiments.

This technology has been studied and tested for different configurations and applications. Hashemi et al. experimentally tested full-scale rocking Cross Laminated Timber (CLT) (Hashemi et al. 2017) and Laminated Veneer Lumber (LVL) [19] walls with RSFJ hold-own connections. Figure. 2 displays the test setup, close-up view of the hold-downs and the reversed-cyclic test results. As can be seen, the mass timber structural components were able to demonstrate a repeatable and damage-avoidance performance without any degradation in stiffness and strength in the system. Moreover, the damage to the timber elements was insignificant and negligible given that all non-fuse members were capacity-protected. The results of the abovementioned experimental studies gave confidence to practitioners to start exploring the use of RSFJ units in real-life projects.

CASE STUDY: YUKON STREET BUILDING, VANCOUVER CANADA

Project Intro

Having spent most of its history in a two-storey custom-built office block on West 1st Avenue in the Armoury District of Vancouver, BC, Fast + Epp's rapid growth over the last few years sparked the decision to seek out new premises in the Mount Pleasant area of the city in 2016. In view of the ever-increasing migration of people to the suburbs of Vancouver for lower housing costs, there was a desire to shift closer to a rapid transit nexus that would ease transportation challenges for the firm's staff. A few years later a 12m x 37m corner site was purchased – a short walk from what promises to become one of Vancouver's busiest transportation hubs with the existing north-south Skytrain transit link and the forthcoming underground Broadway Line providing an east-west link across the city. Over time, it became apparent that the future Fast + Epp Home Office building would present a prime opportunity to showcase and test contemporary hybrid mass timber office construction coupled with state-of-the-art seismic technology. In view of the firm's involvement in high profile mass timber projects such as the 3 million square foot Walmart Home Office campus in Bentonville, Arkansas and 10-storey Arbour at George Brown College in Toronto, Ontario, along with the desire to push the design envelope on projects, it was incumbent on Fast + Epp to walk the talk when presented with the challenge of designing their own space.

Further, it was imperative to implement resilient earthquake resistance to ensure a safe, occupiable structure during and after an extreme event. Fortuitously, a presentation by Professor Pierre Quenneville from the University of Auckland during Fast + Epp's design investigations led the design team to a solution that will be implemented for the first time in North America.

While having to shoehorn 1,500m^2 of the permissible area into a tight site was not without significant planning challenges, the design collaboration between Fast + Epp and f2a architecture yielded a 4-storey building with generous daylighting at the north, south, and west sides, ample balcony space arising from setbacks at the north and south end of the 4th floor, a 2-storey central atrium connecting the 3rd and 4th floors, and a single-storey underground parking level. Given the site constraints and minimal laydown area, it was critical to prefabricate as many structural components as possible to facilitate a short, seamless erection period – one of the primary advantages of mass timber construction.

Structural Design: Gravity Load Resisting system

Many prefabricated timber and hybrid timber-steel panel options were considered for the floor construction, particularly solutions that would integrate and conceal mechanical and electrical components within the prefabricated assembly in a shop-controlled environment. However, in this instance, simplicity won out over complexity and Fast + Epp elected to use glue-laminated timber beams clear-spanning 12m at 3m spacing supporting cross-laminated timber floor panels. In the spirit of the building becoming a living laboratory, Fast + Epp reduced the size of the glulam beams to a 608mm depth, satisfying strength requirements while pushing the limits on vibration performance. Typically, a disconnect between theoretical and real-life vibration characteristics is found. This was an opportunity to carry out exhaustive testing to better understand these differences. A vibration testing program using accelerometers was established to test the impact of various building elements on the performance of mass timber floors. The testing is in its final stages and initial results from full-scale mock-ups and in-situ testing (pre interiors fit-out) demonstrating acceptable performance.

The floor panels consist of 3-ply of cross-laminated timber panels with a total thickness of 105mm at Levels 2, 3 & 4, and 87mm at the roof. A non-composite 50mm thick concrete topping layer and 10mm thick acoustic mat is added on top of the cross-laminated timber. Fire-resistance of up to two hours is achieved by reinforcing the concrete topping and relying on the contribution of both cross-laminated timber and topping. The underside of the cross-laminated timber panels remains exposed in most of the desk areas, with mechanical ducts, sprinkler lines, and electrical conduits strategically located to ensure a clean and tidy ceiling expression. At one end, the glulam beams are supported on steel columns. While glulam columns were also contemplated, the larger sizes were required to achieve up to a 2-hour fire rating at the ground floor, hence intumescent-coated, round 168mm diameter steel columns became the preferred option. The steel not only lends a lighter feel to space but also provides a contrast to the generous amount of exposed timber surfaces. The opposite ends of the glulam beams are supported on glulam columns connected to a 5-ply cross-laminated timber firewall at the zero-lot line.

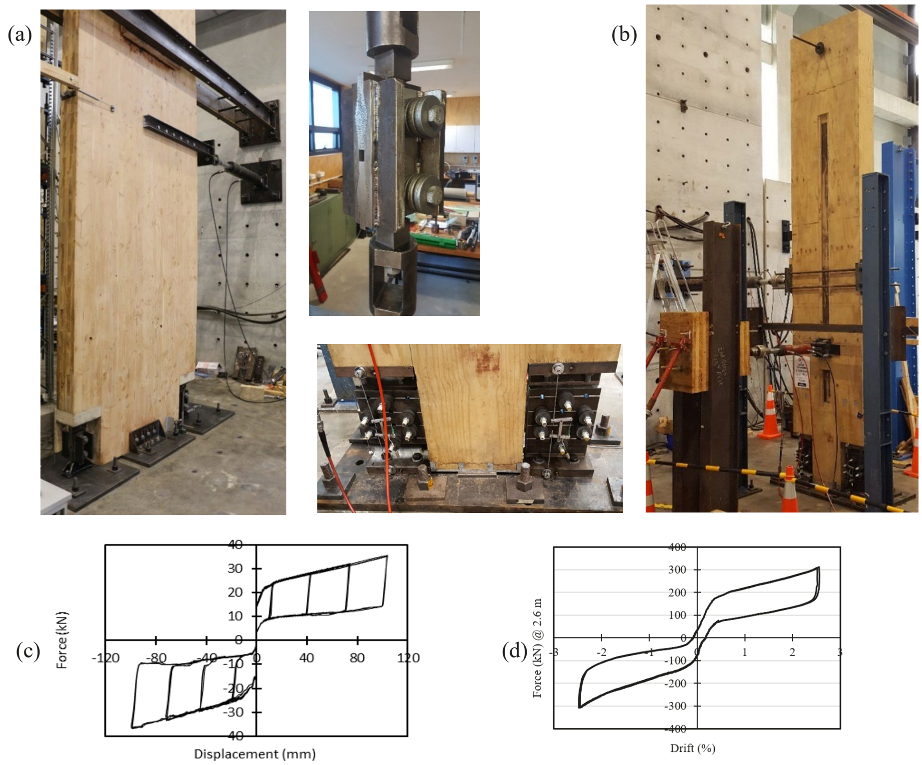

Figure 2. Experimental testing of the RSFJ units on mass timber structures: (a) rocking CLT wall with in-plane RSFJs and resilient shear key (2016) (b) Rocking LVL column with bidirectional RSFJs (2018) (c) in-plane performance of the CLT wall (d) bi-directional performance of the LVL column

Figure 3. Fast+Epp new head office

Figure 4. Gravity system

Structural Design: Lateral Load Resisting System

The lateral force system for the structure is provided in the east-west direction by a series of four narrow CLT shearwalls and one steel braced bay all located on the east side of the structure, and by a CLT shearwall running the entire length of the building along the eastern gridline. The CLT wall seen on the west face of the structure is only nominally connected to the concrete base and is not designed to participate meaningfully to the lateral strength and stiffness of the structure. Due to the emphasis of lateral elements on the east side of the building, the building does have strong torsional behaviour when loaded in the long direction, and the west wall, if included in the seismic system, would attract a significant amount of load from direct loading and from the torsional response. The loading, in this case, would exceed the reasonable design of hold-downs and hence the narrower lateral elements are used to resist the inherent torsion. The wishbone steel braced bays were included for programmatic reasons as this bay required corridor access on multiple floors. The CLT floor panels act as diaphragms at each level, with steel angle chords and drag elements along the perimeter and connecting into the CLT shearwalls. The 3rd floor atrium presents a large diaphragm opening. The use of resilient, self-centring devices within the narrow shearwalls resisting direct shear & torsional shears mitigates the impact of having a torsionally sensitive structure.

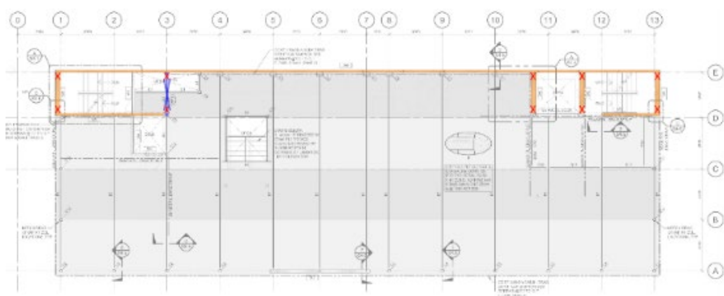

Figure 5. Lateral system layout

The use of mass timber panels for shearwall elements is not without precedent, though it has only recently begun to enter Canadian building codes. CLT in particular makes for a reasonable choice as lateral element due to the glued cross layers being able to transfer shear stresses between adjacent laminations, making it stable under in-plane loading. Many yielding mechanisms exist for CLT shearwall systems, though they generally consist of some manner of metal fastener yielding with timber elements capacity-protected due to the brittle nature of wood failure mechanisms. CLT panels in shearwall systems tend to act as rigid elements with minimal panel shear deformation, and deformation is primarily exhibited in the hold-downs and spline connections. The Canadian timber design code (CSA 086) recommends using a ductility factor (Rd) of 2.0, and an overstrength factor (Ro) of 1.5 for CLT shearwall systems in platform-style framing, subject to various aspect ratio and connection limitations. For reference, a light wood frame shearwall system using plywood panels and common nails has an Rd=3.0 and Ro=1.7. The ductility factor (Rd) from the Canadian code is applied in a similar manner to the ductility factor μ used for seismic design in New Zealand. The overstrength factor Ro is approximately equivalent to the inverse of the Structural Performance (Sp) factor used in New Zealand design, intended to account for the likely increased strength and damping of materials and non-structural elements than what is assumed in the analysis. In this case, the Ro = 1.5 would be approximately equivalent to an Sp= 0.67; however, it is noted that they are not a direct swap for each other.

The system at Yukon Street uses a long array of CLT wall panels on the east property line where spline fastener yielding and hold-down deformation will be the primary contributors to system ductility, though given the squat aspect ratio of this wall assembly (16m high, 35m long) the deformations are not expected to be significant. In the opposite direction, the shearwalls are a single panel wide (3m). Though the building does not strictly fall within the design limits for CLT shearwalls in CSA 086, the recommended Rd, Ro of 2, 1.7 were used for the design of the structure. The effect of the RSFJs at the base of the narrow CLT shearwalls in providing energy dissipation through the hold-downs was deemed in reasonable alignment with the philosophy for code-prescribed CLT shearwalls where base anchors are intended to provide all the yielding for these single panel walls. The designers recognize that though these devices have been used in the narrow CLT shearwalls, the building is not a purely damage-free system due to the spline and yielding hold-down mechanism of the long east wall. The connections between the RSFJs and CLT wall panels were designed as double-knife plates with tight-fit bearing pins (see Figure 7). The connections we designed to the overstrength of 1.35 determined through previous testing [20]. Concrete embed devices were designed for the overstrength of the devices. The horizontal spline connections at level 2 were designed for overstrength forces to ensure that rocking occurs at the base of the panels only and prevent a secondary rocking plane at level 2.

Analysis

Three methods of analysis were used as part of the design, including Equivalent Static Analysis (Linear static), Response Spectrum Analysis (linear dynamic), and Pushover analysis (non-linear static). The analysis software used in the lateral design is primarily Dlubal RFEM 5.17 and ETABS. Initial models were created in RFEM with the RF-Laminate plugin to determine the initial period of the structure and perform a basic equivalent static analysis. The RF-Laminate extension allows for reliable modelling of the CLT section properties, taking into account the orthotropic material properties. More detailed lateral analysis models were created in ETABS 17 for response spectrum and pushover analyses. CLT wall behaviour and material/section properties in ETABS were calibrated to equivalent deformation and properties using RF-Laminate in RFEM. The material calibration is discussed in more detail below.

The following table provides an overview summary of the analysis types that were used and the purpose for each model. Further descriptions of the modelling procedure for material properties, model components and the global model are provided in the following section. The building was initially modelled in RFEM with RF-Laminate being used to accurately represent the CLT shearwalls and floors. Glulam columns, beams and braces were also input with built-in material properties. This model was used for initial period checks of the building and equivalent static lateral analysis to determine the magnitude of forces.

An ETABS model was created for equivalent static and modal response spectrum lateral analyses to further assess the system behaviour, primarily the torsional response and load distribution to shearwalls. ETABS does not have built-in material properties for Glulam and CLT elements, so a calibration model of a 3m wide wall was created in RFEM and a similar wall created in ETABS. Material properties used for the design and analysis of CLT, glulam and steel members are as per the Table 2. The RFEM model with RF-Laminate was considered to provide the most accurate calculation of the deformation of CLT panels. The input materials of the ETABS model were set to an anisotropic material with the elastic and shear modulus set to that of the base lumber that is used in the CLT panel. The results of the calibration analysis, for the panel height/length ratio of approximately 4:1, showed that a good approximation of CLT could be made in ETABs by modifying the thickness of the material to include the longitudinal layers only and ignore the transverse layers. In this case, this resulted in a 245mm thick 7 ply CLT being modelled as 175mm thick. Thin-shell elements were used for both shearwall and diaphragm elements. At the north and south stair cores, walls perpendicular to the narrow CLT shearwalls were connected with rigid links transferring only horizontal axial forces, as walls were not intended to be coupled. The base of the narrow CLT shearwalls was modelled using a shear only connection at the panel centre (reflecting the actual design detail of a concrete shear lug in this location) with linear spring elements at either end representing the RSFJs.

Ultimate hold-down forces determined from the response spectrum model along with allowable rocking displacement of the CLT walls were provided to Tectonus Ltd. engineers in New Zealand, to calculate the device properties. Tectonus provided key values of the flag-shaped hysteresis to define the non-linear springs of the pushover analysis. The pushover analysis was carried out in the across direction only and used to validate the load distribution, the tension in each device and the displacement of the walls. The flag-shaped load-deformation response of the specified RSFJs are presented at the last section of the paper.

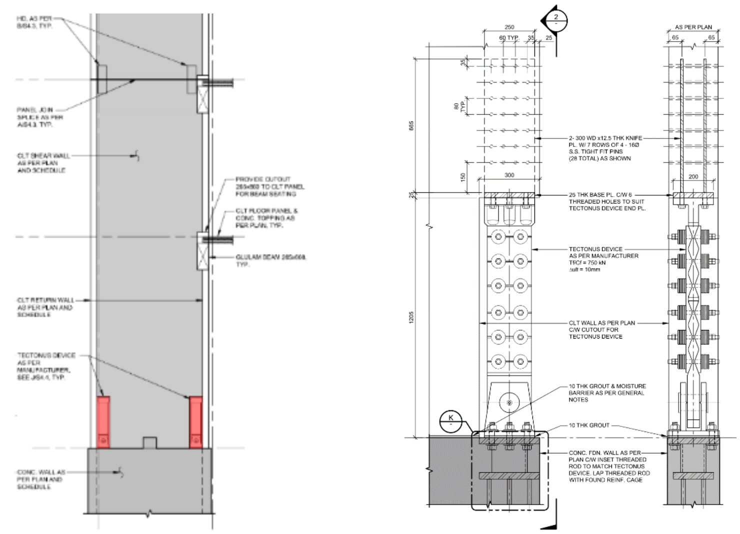

Figure 6. CLT wall elevation and RSFJs detailing

Device Performance Testing

The RSFJ devices are fabricated and tested by Tectonus Ltd. As per the specified characteristics. All devices for this projected were tuned and tested to ensure that the load-slip performance is aligned with the assumed link properties in modelling. During the production testing, three fully reversed cycles were performed on each device until a repeatable hysteresis is achieved.

Figure 8 shows the test setup and the cyclic test results related to a hold-down connector and a tension-compression brace. The performance tolerance for production testing was -/+ 5%.

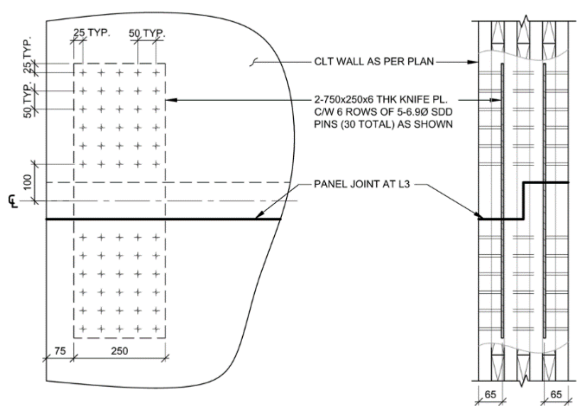

Figure 7. Capacity-protected connection between the CLT panel and the RSFJs

Table 1. Analysis Summary

Purpose of analysis	**Type of analysis**	**Structure modelled**	**Software**
Initial sizing of lateral system	Equivalent static seismic analysis	Entire structure	Dlubal RFEM
Calibration of CLT wall material properties	Static analysis with notional horizontal load	Single wall, full-height	Dlubal RFEM & ETABS
Detailed design of the lateral system	Modal response spectrum seismic analysis (torsional response)	Lateral structure and diaphragms (gravity loads not included)	ETABS
Verification of the RSFJ devices	Non-linear Push-over analysis	Lateral structure and diaphragms with RSFJs specific elements	ETABS

Table 2. Summary of material properties.

Material	**Grade**	**Density (kN/m^3)**	**Modulus of Elasticity (MPa)**	**Shear Modulus (MPa)**
CLT (E-W walls)	E1M5	0.44	12,400	775
CLT (N-S walls)	V2.1	0.44	9,500	594
CLT (floors)	V2M1.1	0.44	9,500	594
Glulam (beams)	D.fir 24f-E	0.55	13,100	813
Glulam (columns)	Spruce 12c-E	0.44	9,700	631
Steel Elements	350W	77	200,000	77,000

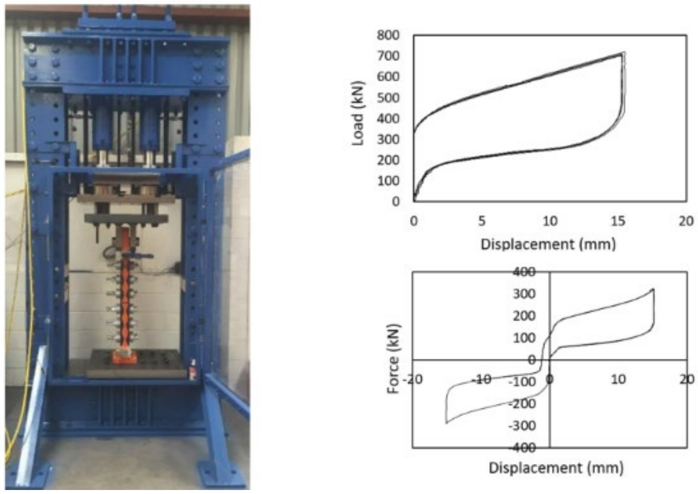

Figure 8. Testing of the seismic devices: (a) testing frame (b) shearwall hold-down performance (c) timber brace device performance

Construction

Due to the narrow lot situated on a busy city corner, the timber elements were sent to a staging yard outside the city where exterior insulation was pre-installed and RSFJs were test fit. Trucks loaded with mass timber were sent to site first thing in the morning for pick-and-place installation of the walls and columns first, followed by glulam beams, and finally CLT panels. Each floor took ~1 week, and steel plate drag straps and miscellaneous metals were installed after the entire wood structure was in place. Timber panels were stripped of their wrapping just prior to installation and placed without any additional temporary moisture management measures. The installation took place during dry summer months and hence the need for additional temporary material was deemed unnecessary, and re-finishing of the exposed wood has not been extensive. The majority of the wood structure is left exposed, including several of RSFJs located in the north and south stair shafts as a demonstration of the novel technology employed within the structure.

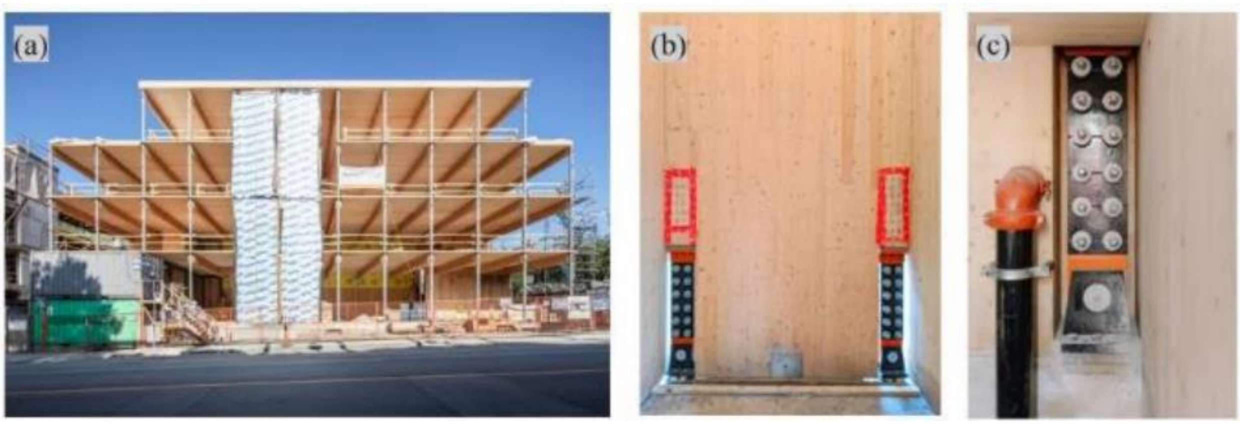

Figure 9. (a) full building elevation (b) CLT wall base (c) installed seismic devices

CASE STUDY: KEITH DRIVE BUILDING, VANCOUVER CANADA

Gravity Load Resisting System

The Keith drive project is a 10-storey mass timber office building supported on a concrete podium with 4 levels of concrete parking below grade located in Vancouver, BC, Canada. The gravity system above the concrete podium consists of glulam and steel post-and-beam construction supporting long-span CLT floors. The CLT floors are supported on perimeter dropped glulam beams, and interior flush steel beams, all supported on large glulam column sections (see Figure 10).

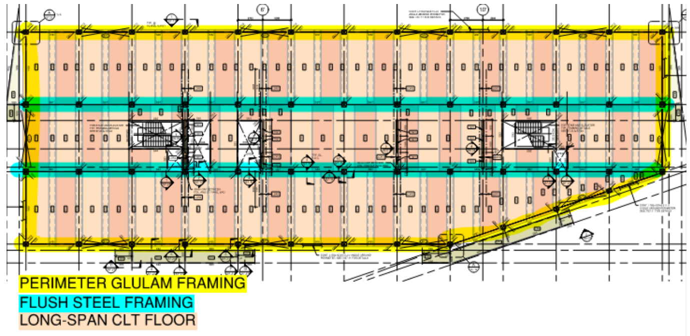

Figure 10. Plan view of timber framing

These flush beams accommodate simple mechanical and electrical implementation, like any flat slab concrete building construction. The exterior perimeter dropped glulam beams, glulam columns, and CLT floor plate are all design to be exposed throughout the building to accommodate the clients desire for wood expression through the building.

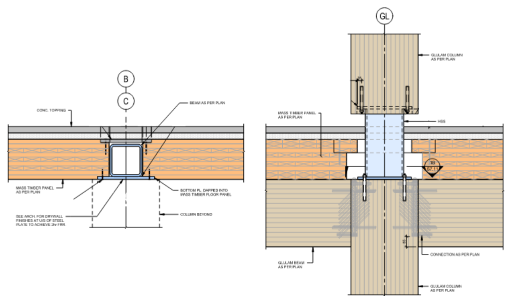

Figure 11. Connection layout

Lateral Load Resisting System

The Keith Drive office building represents a first of its kind in North America with respect to the Lateral Force Resisting System (LLRS) implementation. The project implements a combination of interior CLT shearwalls and perimeter timber braces frames as the lateral force resisting system in a seismic zone (see Figure 12). For the timber braces, Tectonus friction damping devices are implemented at the end of each brace member at each level. These friction damping devices are connected to the timber braces with capacity protected tight fit pin connections using shop installed double knife plates into the brace system. The brace member and RSFJ is then connected into the timber frame with a single capacity protected steel-steel pin connection, facilitating true pin behaviour in the system overall. The timber frame connections are also capacity protected. All the brace members and connections are designed to be capacity protected based on the probably ultimate strength of the Tectonus devices past the restoring force limit.

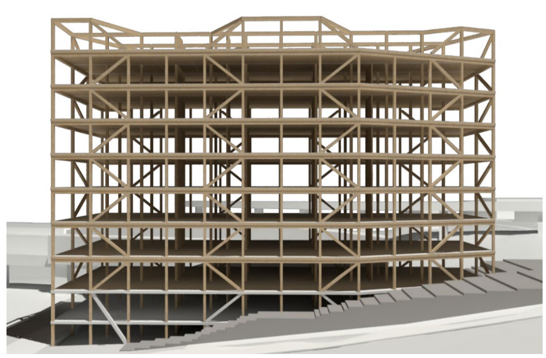

Figure 12. Keith Drive mass timber structure

For the CLT shearwalls, Tectonus dissipators were implemented at the base of the shearwalls at hold-downs only. The hold-downs are connected to the CLT shearwalls using double knife plates and tight fit pins. Stiffness compatibility over the length of the connections has been considered by providing a large W-section facilitating the connection over its entire length to accommodate the long connection length required to accommodate the high forces and prevent break-out. the walls system is designed to act as a rigid body with capacity protected vertical spline connections, rocking about a stationary point at the centre of the shearwall base. The shear connection at the base provides both a true pin to facilitate the desired rocking behaviour around a central point, as well as a series of shear lug plates to facilitate the capacity protected shear load transfer. This approach allows the Tectonus devices at both ends of the shearwall to be engaged simultaneously.

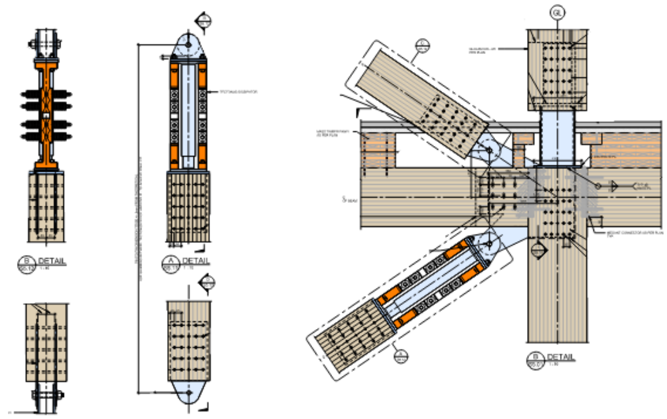

Figure 13. Detailing of the brace connections

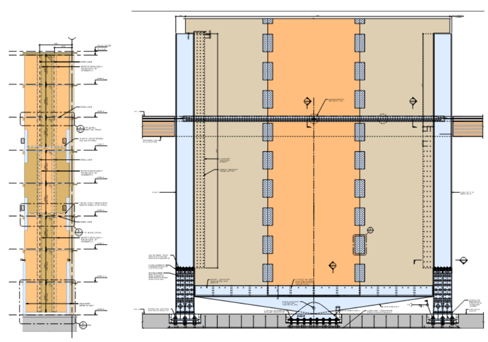

Figure 14. Detailing of the wall connections

The full building was implemented in a non-linear time-history analysis (NLTHA) in ETABS. The Tectonus devices were implemented with non-linear hysteresis as outlined in previous sections of this paper. The glulam framing was implemented with true material properties. The CLT shearwalls were implemented with calibrated shell elements to accurately represent the behaviour of the system, and effectively rigid links at all capacity protected connections. The diaphragms were implemented with tuned shell elements created to provide equivalent stiffness of the true diaphragm based on a complex diaphragm, including all CLT spline connections completed in Dlubal RFEM and accurate representations of the CLT panels using RF-Laminate. Several iterations of the NLTHA were completed in ETABS to tune the Tectonus damping devices and to establish capacity protected load requirements. Additionally, partial incremental dynamic analysis was completed to both evaluate the behaviour of the system for a seismic event with a 40% in 50-year return period, as well as a 1% in 50-year return period. The lower seismic event (former) was evaluated to determine the drift occurring and what damage the building might see at this more frequent return period. The higher than code-required seismic load was applied to evaluate the risk of collapse for seismic events somewhat larger than code required.

CASE STUDY: ON5 BUILDING, VANCOUVER CANADA

Gravity Load Resisting System

Named for its location near the intersection on Ontario Street and East 5th Avenue in Vancouver, Canada, oN5 is an innovative four-storey building designed and constructed to showcase the potential for commercial mass timber. The four-storey building, 840 m2, designed by Hemsworth Architecture and Timber Engineering Inc. (formerly Equilibrium consulting Inc.), is scheduled to be completed in 2022. Figure 15 shows photos of the building during construction. The building is the new home for Timber Engineering Inc., an engineering firm with a worldwide reputation for its advanced timber engineering expertise.

Figure 15. The oN5 Building during construction (Credit KK Law Courtesy – Naturallywood.com)

Podium construction

To optimise the seismic performance of the top three-storey mass timber portion, a two-stage analysis approach as per the National Building Code of Canada [21] was followed. The high stiffness of the reinforced masonry walls and concrete core, relative to the CLT system, enables to obtain a rigid box at the platform level. Using a two-staged approach, where $k_{lower}>3\times k_{upper}$, oN5 building was analysed as two separate structures. Herein, k_{upper} is the stiffness of the upper portion, and k_{lower} is the stiffness of the lower portion of the building, below the transfer slab. The design of the lower portion included the addition of the forces generated by applying the lateral capacity of the upper portion. Based on the capacity design approach, the lateral design of the upper mass timber portion was designed using $R_d\times R_o$ (ductility and overstrength factors) equivalent to the considered CLT shearwall system, whereas the lower stiffer portion would be designed elastic using $R_d\times R_o$=1.3.

CLT Diaphragms

Like the diaphragm analysis, CLT shearwalls analysis and designs are governed by the connections, assuming that the panels themselves mostly as rigid bodies and analysed using suitable mechanics. The 2019 Canadian standard [22] recommends that CLT shearwalls act in rocking or in a combination of rocking and sliding. For the oN5 building, all CLT wall panels are balloon-type, continuous from the concrete transfer slab to the roof, with the floor panels connected to their sides. Typical three-storey and two-storeys CLT wall panels are approximately 11mx3.0m and 8.7mx3.0, resulting in aspect ratios of 3.6 and 2.9, respectively. These aspect ratios ensured the ductility of the building through the desirable rocking mechanism of the CLT panels when subjected to lateral loads. The CLT core is composed of a single panel, with rocking as the kinematic mode. The behaviour of the CLT walls in the long direction is governed by coupled-panel kinematic behaviour, where the panel-to-panel joints allow each panel to rotate about its respective point of rotation. For CLT panel in balloon-type, the components contributing to the lateral deflections due to horizontal forces are: i) bending of the panel; b) shear of the panel; c) rotation of the panel; d) sliding of the panel; and e) slip of existing panel-to-panel joints. Given the aforementioned zero-lot-line site constraints, the design limits the total drift of the building to approximately 2.0% drift, with 1.5% considered as the design target for ultimate limit state design. The oN5 building uses innovative hold-down components, the Resilient Slip Friction Joint (RSFJ) developed at the University of Auckland to reduce the total lateral drift of the building while meeting the target ductility by ensuring that rotation (rocking) of the walls governed with other deformations deemed negligible. The inherent self-centring characteristic of these devices ensured the global re-centring behaviour of the core and the buildings. This concept has been successfully tested for rocking CLT [18] and LVL [19] walls. As shown in Figure 16, four (4) RSFJ were installed at every corner of the CLT central core as the main seismic LLRS providing ductility in both orthogonal directions. Hold-downs were not used for the long CLT walls, assuming energy dissipation is provided through the panel-to-panel joints.

PERFROMANCE-BASED SEISMIC DESIGN AND ANALYSIS

This building is designed for a design life of 50 years with an importance factor of 1.0 (normal occupancy). The 3-storey mass timber building has a footprint of 34.1 m by 8.2 m an inter-storey height of 3.6m, and top storey of 3m. As mentioned, the lateral load resisting system in one direction is fixed base CLT walls and in the other direction is a rocking CLT core. The maximum allowable drift for the building is 1.5% for the rocking direction to target a low damage performance. The building is in Vancouver, and the soil type is classified as Type C. The following sections will describe the method used and details of the analysis. The lateral elastic forces are calculated as per the NBCC cl.4.1.8.11 using the lateral force method. The parameters for site-specific seismic hazard spectra for a 5% damped horizontal acceleration of 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 sec periods, with PGA and PGV for a 2% probability of exceeding in 50 years were taken from Table C-3 of NBCC2015 (annual probability exceedance of 1/2500). The site-specific design spectral values (S(Ta)) are calculated from the PGA to determine the base shear. For the initial analysis, the R_o and R_d shall be taken from Table 4.1.8.9 of NBCC2015 for timber walls.

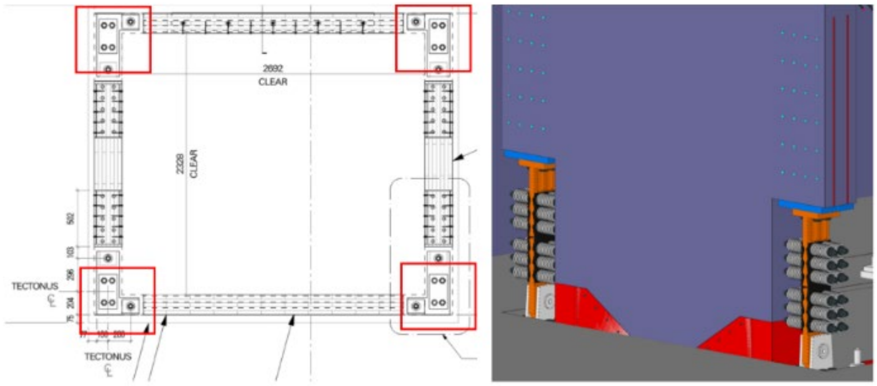

Figure 16. Location of RSFJ hold-down plan view (left), and isometric view of single panel (right)

A Direct Displacement Based Design (DDBD) procedure is used to specify the RSFJs [16]. In this procedure, the structure is represented as an elastic Single Degree of Freedom structure (SDOF) with effective stiffness and effective period to predict the inelastic response. The pushover curves are generated for the structure using the two critical load patterns. These Pushover capacity curves for the building is then converted to SDOF pushover to produce the demand Acceleration Displacement Response Spectra (ADRS) curves [23]. When both capacity curves intersect the scaled demand spectrum, the performance points are interpolated to determine the building performance and calculate the base shear. Accordingly, the equivalent R_d factor is determined to design the RSFJs. Note that RSFJs can be designed for various levels of damping as per the demand of the structure and various values of Rd as per the required performance. Figure 17 displays the capacity curves and the performance points.

A potential difficulty with the pushover analysis is that it represents a static distribution of the seismic forces acting on the frame. Conventionally, an inverted triangular distribution of lateral seismic forces up the height of the frame could be assumed (as per FEMA 356 [24]), but this approach does not consider higher mode effects or changes in displaced shape post-yield. Accordingly, two load patterns were considered for the analysis (see Figure 8).

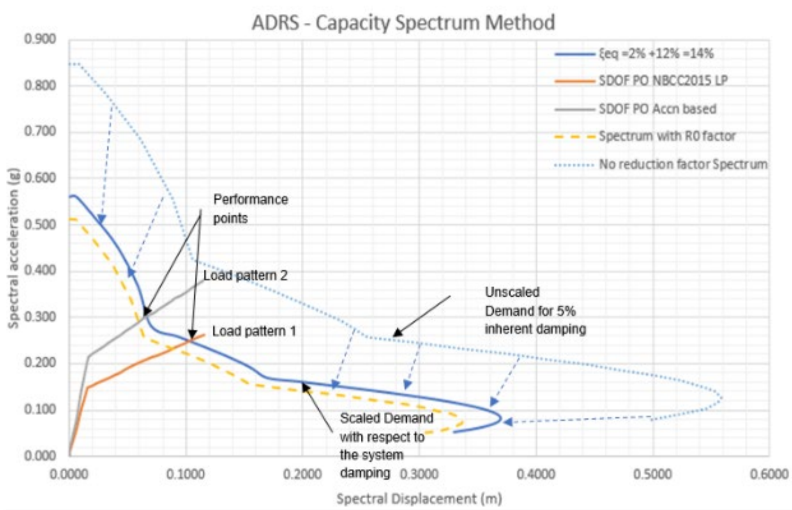

Figure 17. Capacity and performance curves

CONCLUSIONS

This paper provides the details and implementation of a novel connection system for a mass timber structure in a seismically active region. Resilient slip Friction (RSFJ) devices were used as hold-down connector and tension/compression braces for the Fast+Epp head office, Keith Drive building and on5 building located in Vancouver, Canada. The design approach, construction sequence and observed challenges for the projects are discussed. The findings of this paper show that with a robust design and careful detailing, a seismic resilient mass timber system is achievable even if this type of structures is not covered by the current international standards and guidelines. The finding encourages researchers and engineers to start considering seismic resilient mass timber structures.

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