A 10-STOREY TIMBER BRACED FRAME AND CLT STRUCTURE IN VANCOUVER, BC (CASE STUDY)

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ABSTRACT

This paper outlines the structural design approach used for the Keith Drive Office Building project, a 10-storey mass timber building in Vancouver, British Columbia, Canada. The office building includes exposed timber throughout most of the building, including timber brace frames along the perimeter, and cross laminated timber (CLT) shearwalls at the interior near the elevator and stair cores for seismic and wind forces. The project deployed resilient slip friction joint (RSFJ) dampers as energy dissipative devices on both the timber braced frames and CLT shearwalls. Non-linear time history analysis (NLTHA) was completed to evaluate the performance of the building. Detailed modelling and calibration of 2D shell elements was undertaken to ensure a reasonable understanding of the impact of the semi-rigid CLT diaphragm.

KEYWORDS: Mass Timber, Tall Timber, Tectonus, non-linear time history analysis, CLT shear wall, timber brace

Figure 1: Keith Drive Rendering

1. INTRODUCTION

The Keith Drive Office Building is a 10-storey mass timber structure and construction began in the fall of 2022 in Vancouver, British Columbia, Canada. The 43-meter-tall structure is comprised of nine floors of mass timber construction over a concrete podium and four levels of below-grade concrete parking. The timber gravity structure consists of 9-ply cross laminated timber floor panels supported on dropped perimeter glulam beams and flush interior steel beams. The Seismic Force Resisting System (SRFS) consists of perimeter timber braced frames and interior balloon framed CLT shearwalls. See Figure 1 and 2 for building renderings.

2 DESIGN

The project intent was to create a mass timber building. The project was awarded significant external funding through the Green Construction in Wood Program from NRCAN allowing it to push the boundaries of timber engineering and design in Canada, the project design quickly headed in a direction of a 'pure' mass timber superstructure complete with a timber lateral system. Exposing the timber to view wherever possible was a key goal for the architect and owner, providing a unique experience for occupants.

The extent of exposed timber in both the gravity and lateral system exceed code prescribed limitations, requiring the project team to work closely with the city to address all concerns. Alternative solutions were provided to address the various issues; the approach for the structural alternate solution included peer review and non-linear time history analysis.

The base of the structural system consists of four levels of below grade concrete parkade and one level of concrete podium to accommodate the sloped site. The timber superstructure springs from the second floor concrete transfer slab as shown in Figure 2. From Level 2 to Level 10 CLT floors are supported on dropped perimeter glulam beams and interior flush steel beams. Interior perimeter timber braced frames and interior balloon framed CLT shearwalls provide the SFRS.

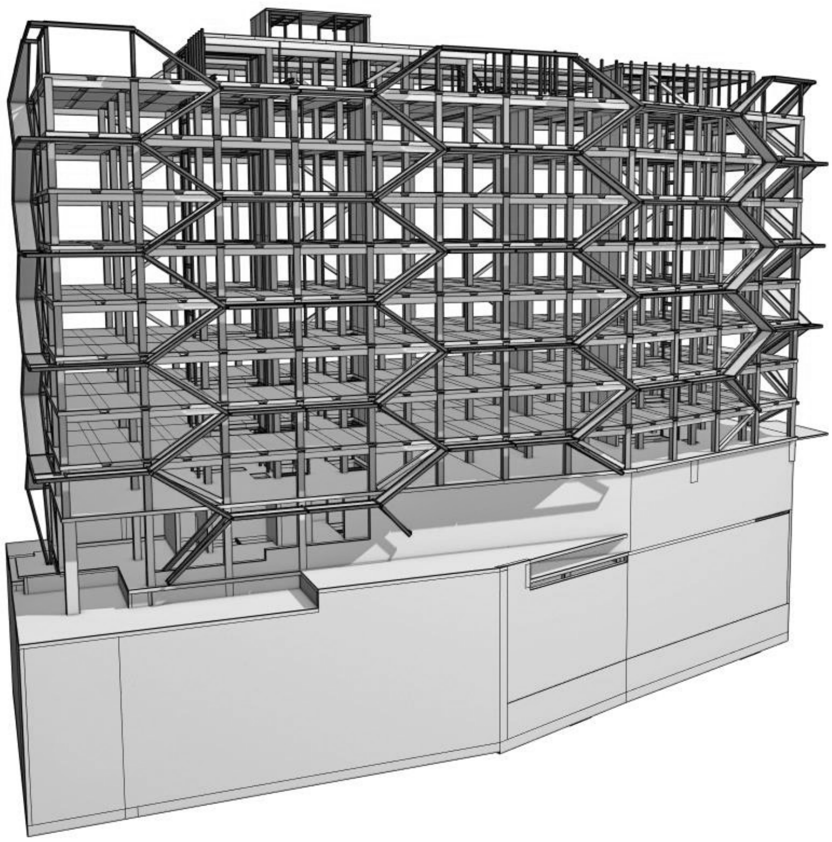

Figure 2: Structural Framing Model

2.1 GRAVITY SYSTEM

The gravity system is comprised of 9-ply, 315mm thick CLT panel with a non-structural concrete topping spanning between glulam perimeter beams and flush steel interior beams. The interior steel beams are comprised of an HSS with an extended bottom plate welded to the underside, acting as a seat for the CLT; this flush framing system allows for unobstructed mechanical distribution and reduces overall building height. Both steel and glulam beams are supported by glulam columns carried down to the concrete podium at Level 2.

The building requires a 2-hour fire rating requirement (FRR) for all structural gravity members. The timber gravity framing achieved its FRR via char design as per the O86-19 Annex B design method [1]. The steel beam achieves the required FRR by way of drywall encapsulation at the underside, matching the aesthetic of other service chases adjacent.

The central area of the building accommodates a dropped ceiling to allow the mechanical, electrical and plumbing (MEP) to be routed as required, with the flush steel supports avoiding any conflicts. Beyond this central zone, the CLT soffit is exposed throughout, with all MEP routing passing within planned spline zones in the CLT floor as shown in Figure 3. 3-ply, 105 mm thick CLT panels act as splines connecting the 9-ply panels.

The 9-ply, 315mm panels are fully exposed with double outer layers at the top and bottom to accommodate a 2-hour FRR over the 8.2m span, while also increasing panel stiffness for deflection and vibration response. Similarly, the supporting glulam columns and perimeter beams are all designed to achieve a 2-hour FRR through char without encapsulation. The 3-ply, 105mm CLT splines are fire protected with type X gypsum board. The flush steel beams are similarly encapsulated to protect the steel withough intumsecent paint or fire spray.

The column-to-column connection, shown in Figure 4, is configured to provide direct load transfer between the vertical elements rather than transferring forces through the beams or the floor plans. The beam-to-column connections at the perimeter make use of Knapp megant form fitted connectors at all locations, embedded to be protected from char, and the interior steel beams bear directly on the column-to-column end connections. Glulam columns will arrive on-site with a steel connecting plate and Hollow Square Sections (HSS) stubs fastened to the end-grain with glued-in rods.

The glulam column above will have a similar connection with smaller diameter HSS stubs. Stubs are connected using glued in threaded rods bolted to the stub, which allow for simple installation and a tension connection in the extreme event where a column below is eliminated, according to progressive collapse principles. CLT floor panels will be notched around the HSS sections and bear directly on the column below.

2.2 LATERAL SYSTEM

The lateral system consists of perimeter timber braced frames and interior CLT shear walls. The perimeter timber braced frames mimic the architectural expression of the honeycomb on the building's façade. The interior CLT shearwalls are placed near elevator or stair cores to accommodate the large open floor plan desired by the architect and owner to provide flexibility around the egress stair shafts for more

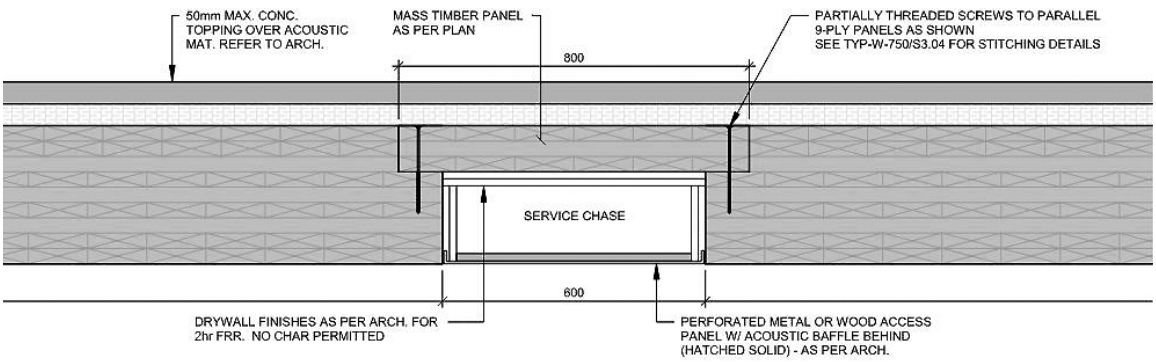

Figure 3: 3-ply CLT spline for MEP routing

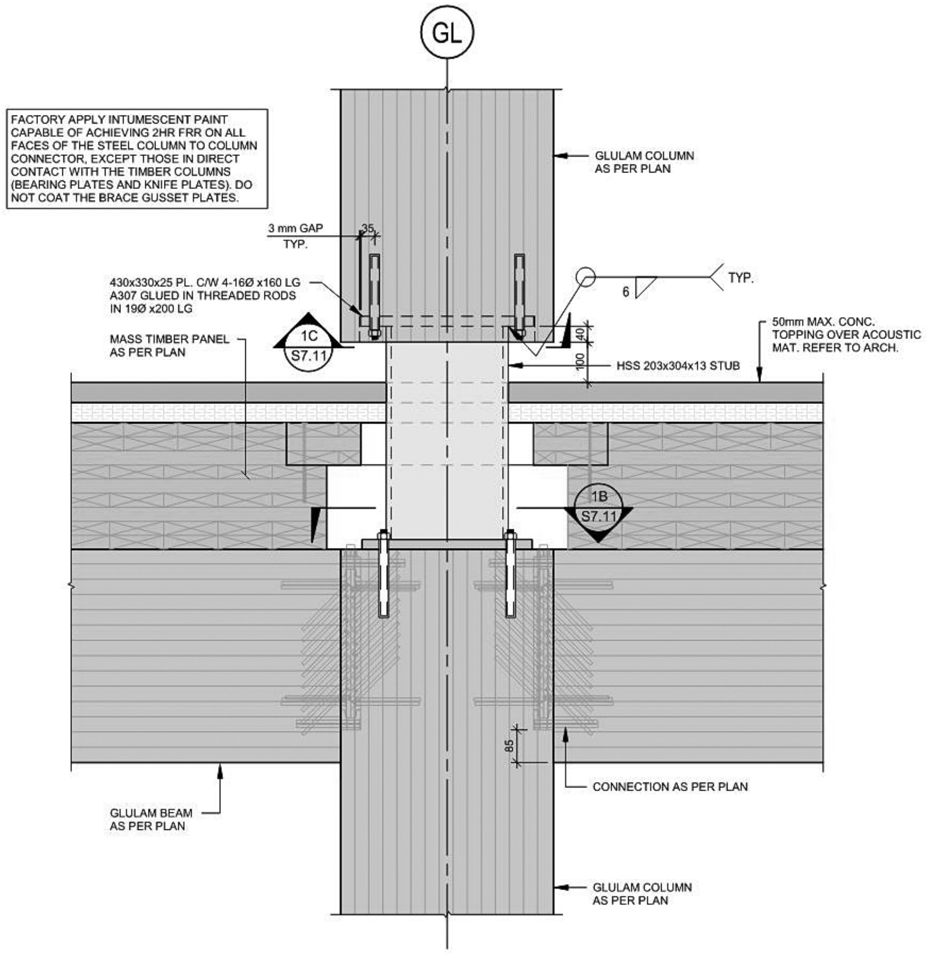

Figure 4: Column-to-column and beam-to-column connection at perimeter

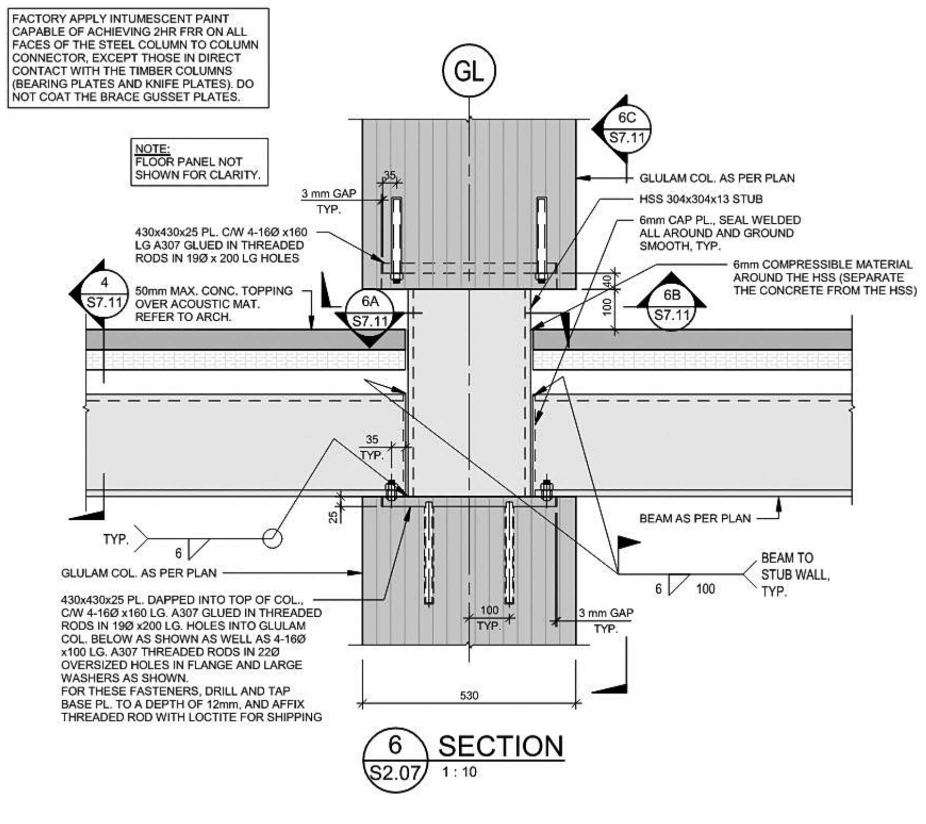

Figure 5: Column-to-column and beam-to-column connection interior

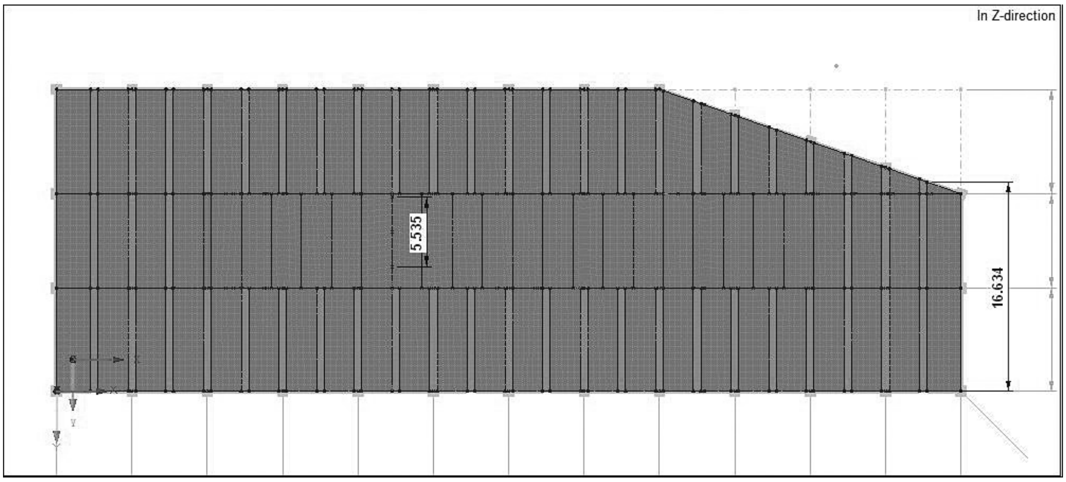

Figure 6: Plan view of SFRS elements within the timber levels

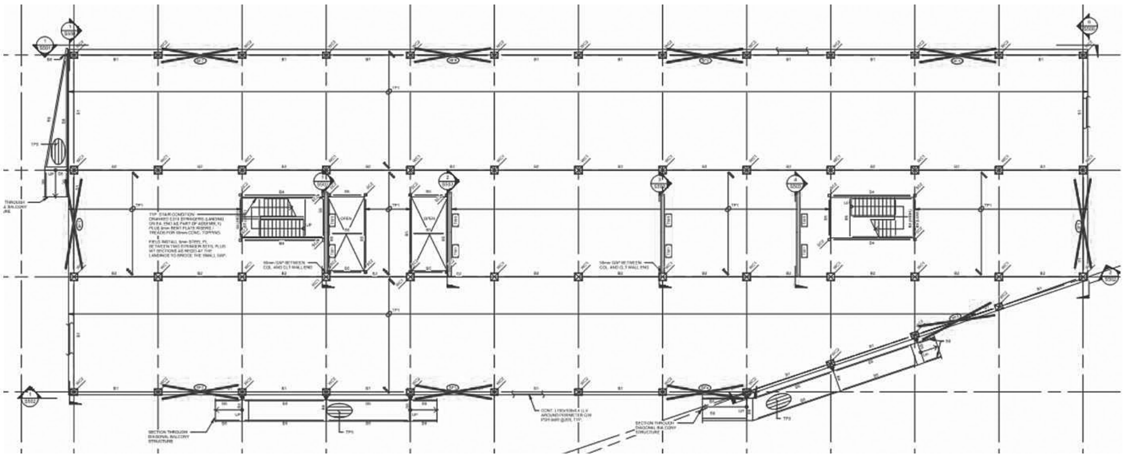

Figure 7: Timber Braced Frame Connections

interior glazing. Refer to figure 6. To achieve energy dissipation without damaging the structural members or connections, Resilient Slip Friction Joints (RSFJ's) are provided at each brace (one end), and at the ends of the CLT shearwalls, as hold-downs., allowing energy dissipation without damage to the structural system as shown in Figures 7 and 8.

Both timber braced frames and CLT shearwall SFRSs are codified systems in the most recent version of National Building Code of Canada (NBCC 2020), with defined ductility and overstrength values and height limits [2]; however, the prescribed height limits are significantly less than the actual building and the ductility and overstrength values associated with the codified systems are not relevant for a system with RSFJ's.

Therefore, to support the SFRS design, an NLTHA was completed, alongside an onerous third-party peer review in accordance with NBCC Structural Commentary J [2]. Iterative NLTHA was used to optimise the RSFJ design for the 2% in 50-year design earthquake. NLTHA was also implemented at 130%, and 150% of the design earthquake to review the collapse risk for an earthquake exceeding the design earthquake. Linear dynamic analysis was performed at service level earthquake (40% in 50-year) to ensure the RSFJs remain in the initial linear portion of the hysteresis for low level earthquakes.

2.3 NLTHA APPROACH

Fifteen ground motion sets, scaled to the design earthquake level (2% in 50-year) were used with each set including two horizontal and one vertical acceleration records. The two horizontal records were implemented in two orientations, rotated by 90 degrees, resulting in total of thirty time history load cases. The average responses for the top five records are considered for the acceptance criteria. The maximum response for the top record was also reviewed; while some devices exceeded the ultimate

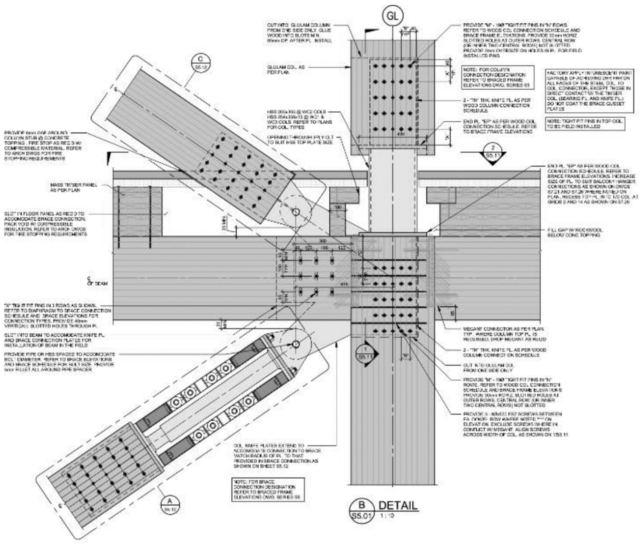

Figure 8: CLT Shearwall Base Connection

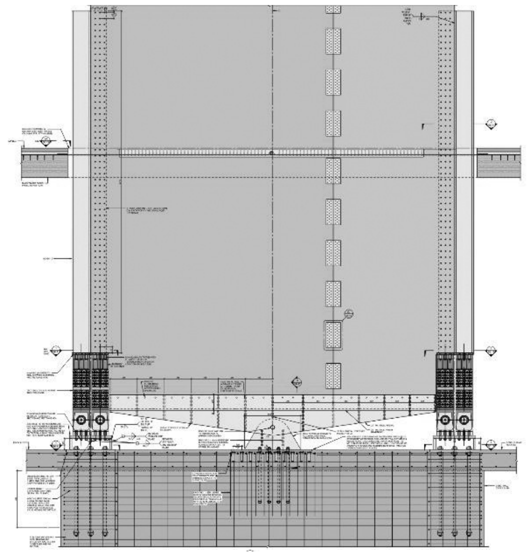

Figure 9: Detailed Diaphragm Model shell type layout

force (F_{ult}) and ultimate displacement (Δ_{ult}) limits, they were within the secondary fuse response, not exceeding the stop force (1.25 F_{ult}) and stop displacement (1.5 Δ_{ult}) limits.

Additional NLTHA was conducted using ground motions scaled to 130% and 150% of the design earthquake level to assess the structure's stability at higher hazard levels. For the 130% of the design earthquake level ground motions, the force and deformation demand of the RSFJ devices remained primarily below the stop force (1.25 F_{ult}) and stop displacement (1.5 Δ_{ult}) for the average of the top five responses. For the 150% of the design earthquake level ground motions, the force demand of the RSFJ devices remained below the stop displacement (1.5 Δ_{ult}) but the force demands were 10% ~ 30% above the stop force (1.25 $F_{ult)}$ for the average of the top five responses. Force demand-to-capacity ratios remained consistent throughout all RSFJ devices in braced frames, suggesting the structure is not likely to develop a soft story failure mechanism or force concentration.

The braced frames were modelled with glulam beam, glulam brace, glulam column and non-linear friction spring damper type links representing RSFJ device. These non-linear springs were active in axial direction and fixed in all other degrees of freedom.

The CLT shearwalls were modelled as 3 panels of 245 mm thick shell elements connected using rigid links representing spline plates. Non-linear friction spring damper type links were modelled at each end of the CLT shearwall bottom on Level 2 representing hold-downs. These non-linear springs were active in axial direction and fixed in all other degrees of freedom. At the centre of the bottom of the CLT shearwall on Level 2, a stiff frame element was modelled representing the shear connection to the concrete structure below. This frame element was released in rotation about the axis perpendicular to the face of the shearwall to represent the true-pin condition. This connection allows the shearwall to rotate about its center and transfer shear forces to concrete wall below.

The CLT shearwall shell element's stiffness properties were calibrated against CLT shearwall modelled in RFEM RF-Laminate.

2.4 DIAPHRGAM CONSIDERATIONS

CLT diaphragms are complex and provide significantly stiffer behaviour than that associated with a flexible diaphragm given the large format stiff billets used throughout. Despite the relatively high stiffness of the panels, the stiffness of the diaphragm is much lower than that of the individual panels due to the flexible nailed or screwed connections between panels. These discrete connection locations between panels result semi-rigid behaviour. Evaluating the rigidity of CLT diaphragms is directly related to the panel sizes, the number of joints, and the stiffness of the joints between panels, as well as the chord and drag connection stiffnesses.

Understanding and representing the diaphragm behaviour in the NLTHA model was a critical component of ensuring a realistic building response in the model. The shared behaviour of the CLT splines, connections to the steel and timber beams and drag connection into the lateral elements is critical due to the comparatively high stiffness of the CLT panels and low stiffness of the connections between elements. Detailed diaphragm models can be developed to include all the elements, but that represent a large computation burden. If implemented in a larger NLTHA they would significantly slow the model processing. To resolve this problem a detailed diaphragm model was developed for a for a single floor to study the diaphragm behaviour and calibrate a simple shell element, which is adjusted to match the load distribution and deformation coincident with the detailed model, via membrane stiffness modifiers. That simple shell element was then implemented in the larger NLTHA model to simplify analysis.

The detailed diaphragm model is composed of a series of shell elements representing each individual panel with properties associated with the panel types described in the structural drawings as shown in figure 9. Drag and chord elements are also incorporated into the detailed diaphragm model. The joints between the panels, drags, and chord elements are represented with linear line springs with stiffnesses representative of the connection type. Both 3ply 105V panels, and 9ply 315V XL panels with layups based on APA product approvals are implemented in the model. The layout of the individual shell elements and the panel implementation is shown in the light blue panels represent 3 ply CLT panels, and the red panels represent 9 ply CLT panels. The layup for each panel type has been defined using RF-Laminate

The linear springs are designed based on the stiffness properties of the screwed connection between CLT panels and steel drags to CLT panels based on screwed connection stiffness as provided Eurocode 5 (EN 1995-1-1:2004+A1) as described in clause 7.

Wood-wood Stiffness = $\rho_m^{1.5} dl23$
steel-wood Stiffness = $2(\rho_m^{1.5} dl23)$

The stiffness is generally applied in both the x and y directions are shown in Figure 10. These correspond to the axes in the plane of the CLT floor panel at the spline, drag, and chord connections.

The simplified diaphragm model is comprised of a single shell element tuned to represent the combined properties of the complex model as discussed above. The shell element is defined with orthotropic properties modified as needed to allow the load distribution to match that found from the detailed shell element. Additionally, the bending properties in the weak direction of the individual panels (X-direction in the drawings) has been reduced to near zero.

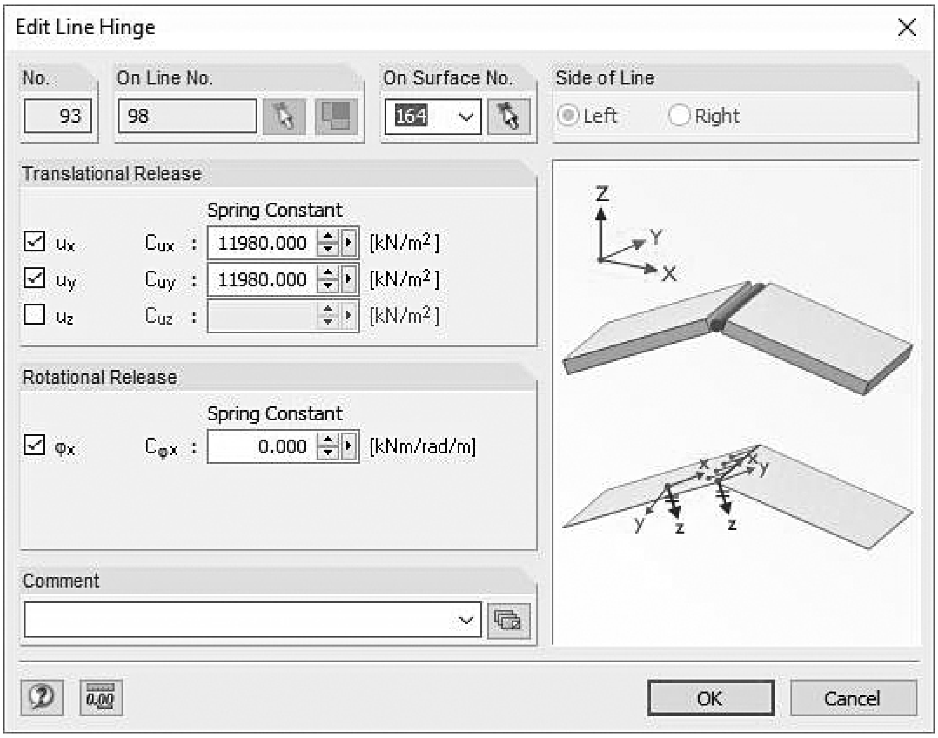

Figure 10: line spring axes

This is a conservative simplification to ensure that the shell element does not resist significant weak axis moment, as minimal moment would be transferred across CLT- CLT splines joints. Property modifiers for weak axis bending and torsion for the shell are thus reduced to near zero.

An iterative process was implemented to bring the force distribution for each lateral element to within 5% of the detailed diaphragm model for a unit applied area load. To verify the behaviour, the reactions of each LFRS elements were compared for both the complex and the simplified shell diaphragm model. For all cases the variation in load distribution was within 5% or less. A summary of the variation in LFRS shear is provided in Table 1.

Table 1: Variation in reaction SFRSs

	LFRS Location	Variance
Y-dir.	Brace @ GL 2	4.80%
	Brace @ GL 14	3.80%
	Wall @ GL 5	2.00%
	Wall @ GL 6.5	2.05%
	Wall @ GL 9	2.50%
	Wall @ 10.5	0.50%
X-dir.	Braces @ GL D	0.2-0.9%
	Braces @ GL A	0.2-1.0%
	Braces @ GL 15	1.5-2.3%

2.5 BEAM-TO-COLUMN CONNECTIONS

The glulam beam-to-column connections required a 2-hour FRR. To achieve this with exposed members, Knapp Megant form fitting connectors are specified with sufficient timber encapsulation to provide the necessary FRR. Inter-storey drift tolerance of these connections was investigated with testing was undertaken to show the inter-storey drift tolerance and established rotation stiffness and strength of the joint.

Perimeter glulam beams connected using Knapp Megant concealed hangers were the connection of primary study, as the direct bearing connection of the flush steel beams at the columns should not pose a problem. The drift capacity of these hangers was not explicitly available from the supplier or fabricator but it is critical to the behaviour of the system. To better understand the behaviour of the hangers, testing was completed to determine the rotational capacity of the hanger under the design loads of the system and localized drift studies and sensitivity studies were completed to assess the behaviour of the frames [5].

2.6 THE RESILIENT SLIP FRICTION JOINT DEVICES

The Resilient Slip Friction Joint (RSFJ) is a friction device that dissipates the seismic energy through friction while providing re-centring force. Friction dissipation occurs via sliding movement of clamped plates. An RSFJ device consists of 2 outer plates and 2 centre plates with elongated holes, which are grooved and clamped together with high strength bolts and disc springs (refer to Figure 13).

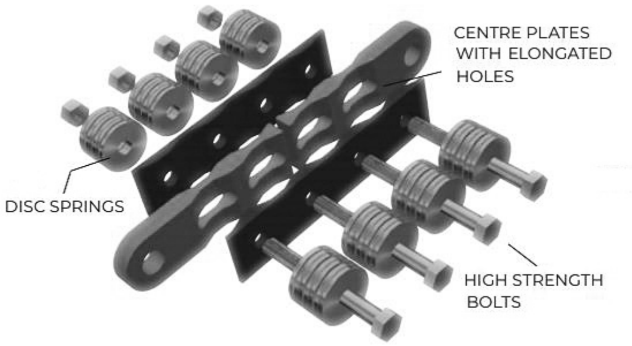

Figure 11: The RSFJ components

When the applied force overcomes the frictional resistance between the sloped bearing surfaces, the centre slotted plates start to slide, and energy is dissipated through friction during cycles of sliding. During unloading, the reversing force induced by the elastically compacted disc springs is larger than the friction force acting between the facing surfaces, providing self-centring characteristics

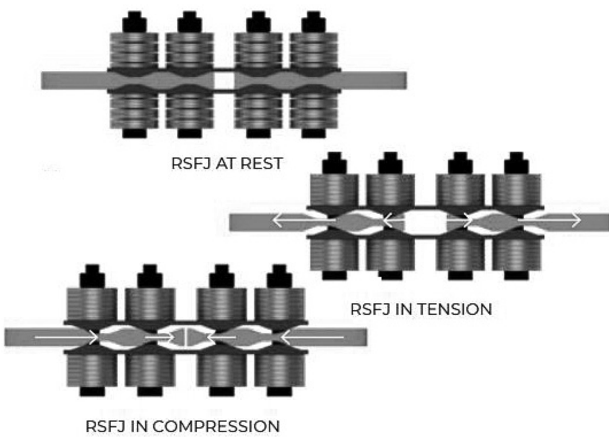

Figure 12: RSFJ cyclic loading behaviour

The hysteretic shape of the response of this type of RSFJ device is a flag shape common for friction based dampers. Each point in the hysteresis is a uniquely defined parameters (F_{slip}, F_{ult} , $F_{restoring}$, $F_{residual}$ and Δ_{ult}) as shown in Figure 15. The number of tapered friction planes and associated bolts/spring washers is flexible, allowing for a 'tunable' connection in both force and displacement.

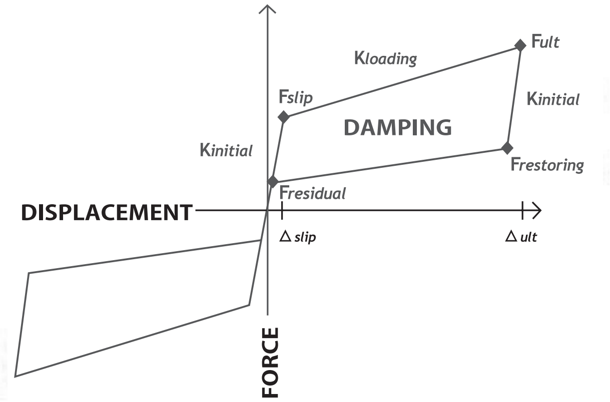

Figure 13: Typical RSFJ Hysteresis

The flag shape of the hysteresis can be tuned as needed to meet the design requirements to achieve the desired load-slip response up the F_{slip}, and F_{ult}. This is achieved by changing variables like the number or angled of the ridges in the plate, and/or the number of bolts and disc springs at each bolt. The behaviour and performance of the RSFJ has been extensively evaluated via joint component tests and large-scale experiments. With testing showing that the performance of the RSFJ is stable and that the theoretically predicted hysteretic behaviour (see Fig. 3(a)) matches well with test results [3]. This technology has been studied and tested for different configurations and applications. Experimentally tested full-scale rocking Cross Laminated Timber (CLT) and Laminated Veneer Lumber (LVL) walls with RSFJ hold-own connections. The mass timber lateral systems showed repeatable and damage free performance without any strength or stiffness degradation. The damage observed in the timber elements or connections was insignificant and negligible given that all non-fuse members were capacity-protected [3] [4].

The RSFJs also have 25% additional capacity beyond the F_{ult} provided in resulting from bolt bending in the RSFJ bolts. This additional capacity prevents failure after the ultimate strength is reached but will no longer represent a self-centering system without damage.

The RSFJs strength and deformation capacities were chosen through multiple iterations of Non-Linear Time History Analysis. The final hysteretic flag shapes for each device are shown in Figures 14 for the braces. The hold-downs are consistent across all 4 walls as shown in Figure 15.

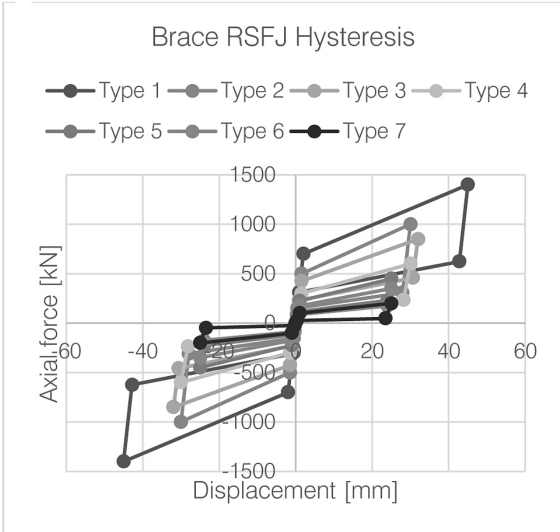

Figure 14: Hystersis for brace RSFJs at various plan and storey locations

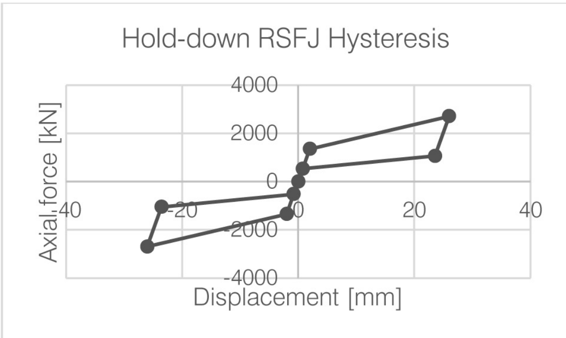

Figure 15: Hysteresis for CLT wall RSFJ hold-downs

Each RSFJ is tested (Figure 16) to ensure that it achieves the specified hysteretic forms as shown in figure 14 and 15. This testing ensures that the ultimate and slip strengths and deformations match the specified values and also is critical in minimizing the overstrength required for capacity protected elements as there is little variability prior to the yield in the bolts.

To achieve the varied ultimate strengths and hysteretic forms shown the RSFJs are tuned with a varying number of spring discs and angle of the saw-

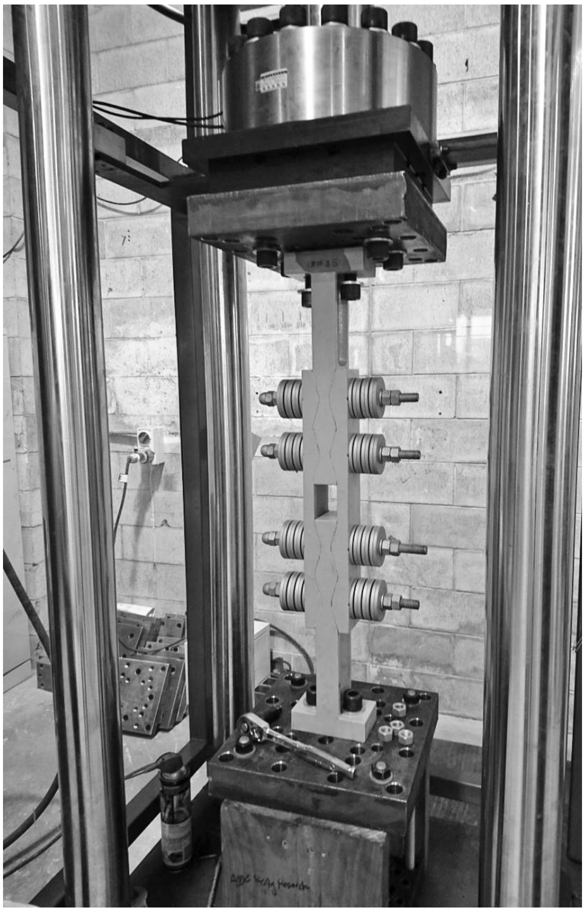

Figure 16: Testing of RSFJ for brace at upper level of Keith Drive project

tooth plates as previously described. At the CLT walls, the system is stable without the presence of the RSFJ due to the base connection and the connections to the diaphragms at each level, allowing the RSFJs to fastened directly to the CLT panels without additional components for stability as shown in Figure 8. At the braces, the RSFJs interrupt the length of the brace; to avoid issues with instability in the brace member under compression, two RSFJ devices are paired with central elongating tubes called Anti-Buckling Tubes (ABTs) to address this issue, without compromising the hysteretic behaviour (Figure 16).

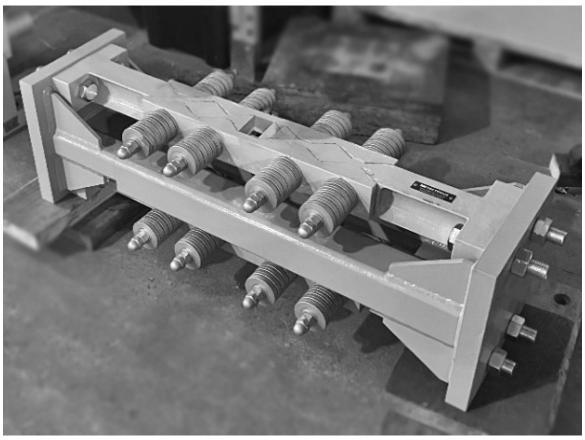

Figure 17: Brace RSFJ for upper-level braces at the Keith Drive Office Building

These assemblies are then bolted to highly coordinated end plates that are either connected in to braces frame nodes at the beam to column connections, or into the ends of timber braces as shown in Figure 18.

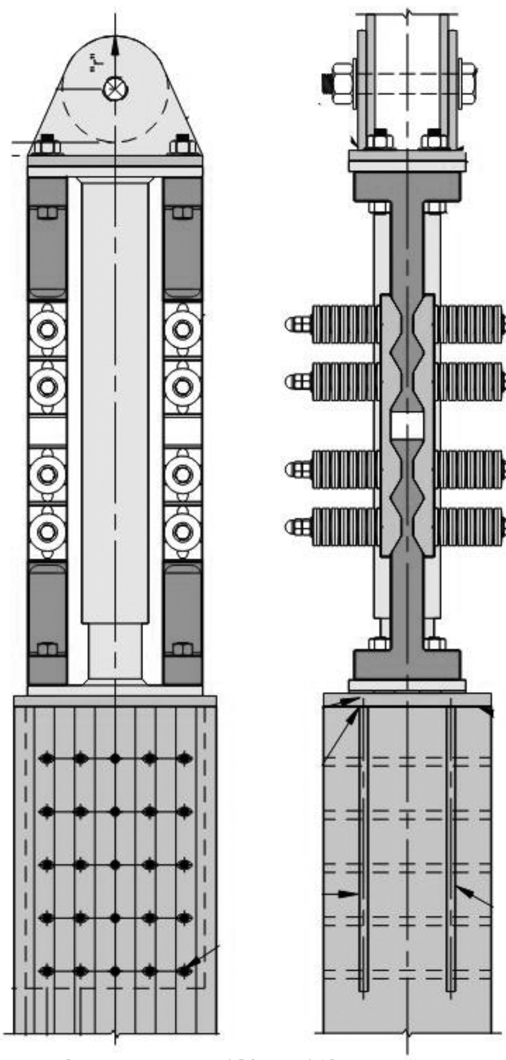

Figure 18: Brace connection of RSFJ and ABTs

2.7 CAPACITY DESIGN

Capacity design principles are applied based on the overstrength of the RSFJ devices, overstrength of the RSFJs is applied to the ultimate strength (F_{ult}) of the RSFJs within the SFRS element in the system. This direct overstrength is applied to: Timber Brace members and their connections to both the RSFJ and the frame, beam members and their connections in braces based, CLT shearwall panels, CLT shearwall base-shear connections, RSFJ hold-down connections the Shearwall, and all other connections between CLT panels in the shearwall. At the CLT shearwalls, because the RSFJs were able to be tuned within a few percent of the ultimate RSFJ strength, the overstrength factor could be applied directly to the NLTHA results.

The glulam column and glulam column-to-column connection design loads have been determined based on the maximum of the probable capacity of the RSFJ devices and the NLTHA results increased by the RSFJ overstrength. As the likelihood of each device within a brace frame simultaneously reaching its probable capacity is low, a Square Root Sum of Square (SRSS) approach is applied to all the columns and column-to-column connections in the brace frames (i.e. components subject to determine the tension/compression design forces in the brace frame columns. The column design loads are determined by combining the SSRS overturning earthquake with appropriate dead, live, and snow loads.

The diaphragm panels and connections are also capacity protected. The design loads for the diaphragms are based on the capacity of the braced frames and CLT shearwalls, at that level, such that the 'fuse' remains the RSFJ, rather than the diaphragm fasteners themselves.

3 CONCLUSIONS

The innovative timber braced frame and tall CLT shearwall system proposed for this project significantly exceeds prescriptive approaches for timber systems in the NBCC. With the implementation of self-centering, zero damage dissipative RSFJ devices, this building will achieve a high level of performance and will represent a first-of-its-kind timber lateral system in a seismic zone for a tall timber building in North America. By performing detailed NLTHA modeling including the non-linear link elements to represent the dissipative RSFJs and linear elements to test drift tolerance of the gravity structure, and accurate shell elements representing the true behaviour of the semi-rigid diaphragm, the overal behaviour of the system has been tuned through performance based design

To achieve a realistic distribution of loads between SFRS elements, a detailed diaphragm model was developed with the development of a simplified, calibrated, shell element to simplify the NLTHA model and analysis.

The building is currently under construction, targeting concrete completion up to level 2 by December 2023, and timber superstructure completion by end of summer 2024

ACKNOWLEDGEMENT

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