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CASE STUDY: A TALL TIMBER BRACE AND CLT SHEARWALL BUILDING IN VANCOUVER, BRITISH COLUMBIA

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ABSTRACT

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This case study describes the structural design approach for the mass timber Keith Drive Office building project located in Vancouver, British Columbia, Canada. The 43-meter-tall structure consists of nine exposed mass timber floors over a concrete podium and four levels of below-grade concrete parking. The upper gravity structure consists of 9ply cross-laminated timber (CLT) floor panels supported on dropped perimeter glulam beams and flush interior steel beams. Perimeter timber brace frames and interior CLT shearwalls complete with supplemental energy dissipation friction dampers make up the lateral system for the building. Resilient Slip Friction Joint (RSFJ) devices, supplied by Tectonus, were designed to localize energy dissipation in a seismic event without damage to the other components of the structural system. To support the supplemental energy dissipation design, a peer review including Performance Based Design (PBD) using Non-linear Time History Analysis (NLTHA) was completed. Performance objectives included life-safety at the design level earthquake, collapse prevention at earthquake levels exceeding the design level hazard, immediate occupancy at the service earthquake and collapse prevention for a post-fire wind event.

INTRODUCTION

Keith Drive is a 43 meter mass timber building built in Vancouver, BC (Figure 1). The building consists of nine levels of exposed mass-timber construction above a concrete podium and four levels of below-grade parkade structure (Figure 2). Exposing the timber was a key architectural design feature for the building. Both the gravity and lateral system consist of exposed wood features that are designed for a 2-hour fire event.

The extent of exposed timber in both the gravity and lateral system exceeds the limitations of the code and therefore the project team used a structural alternative solution that consisted of a full peer review, non-linear time history analysis and performance-based design.

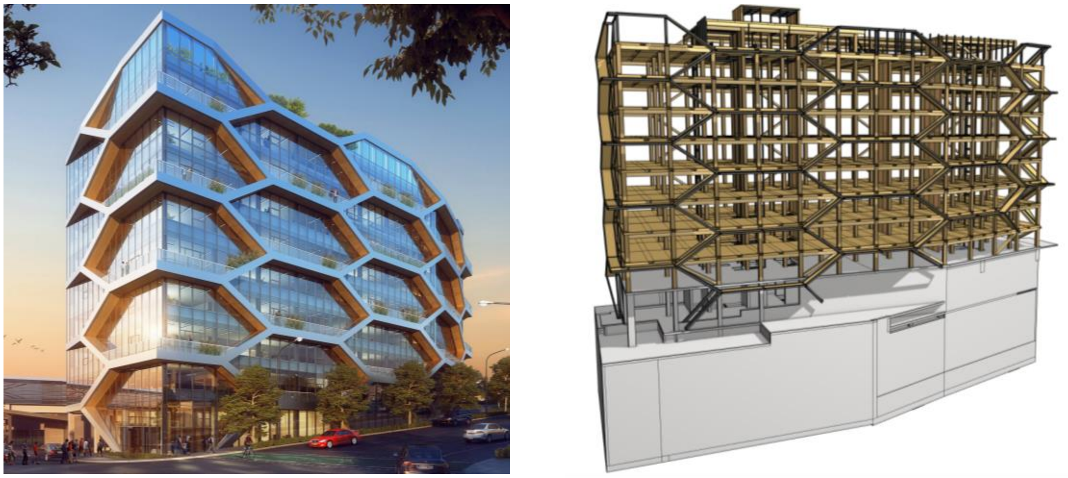

Figure 1. (a)Keith Drive Rendering; (b) Structural Framework Model

GRAVITY SYSTEM

The gravity system on the upper levels consists of 9-ply, 315 mm thick CLT panels supported by interior steel HSS beams and perimeter glulam beams. The beams are supported by glulam columns from Level 2 to the roof. The column-to-column connection, shown in Figure 2, provides a direct load between the vertical elements. The beam-to-column connections at the perimeter use megant form fitted connectors between the glulam beams and glulam columns. The interior steel beams bear directly on the column-to-column end connecitons.

The char design method as per O86-19 Annex B was used to achieve the 2-hour fire rating requirement for the exposed mass timber panels, beams and columns [1]. The megnats are embeded within the glulam member to be protected from char. Drywall encapsulation at the underside of the steel beams was used to achieve the fire rating for the steel members. One side of the CLT walls are encapsulated.

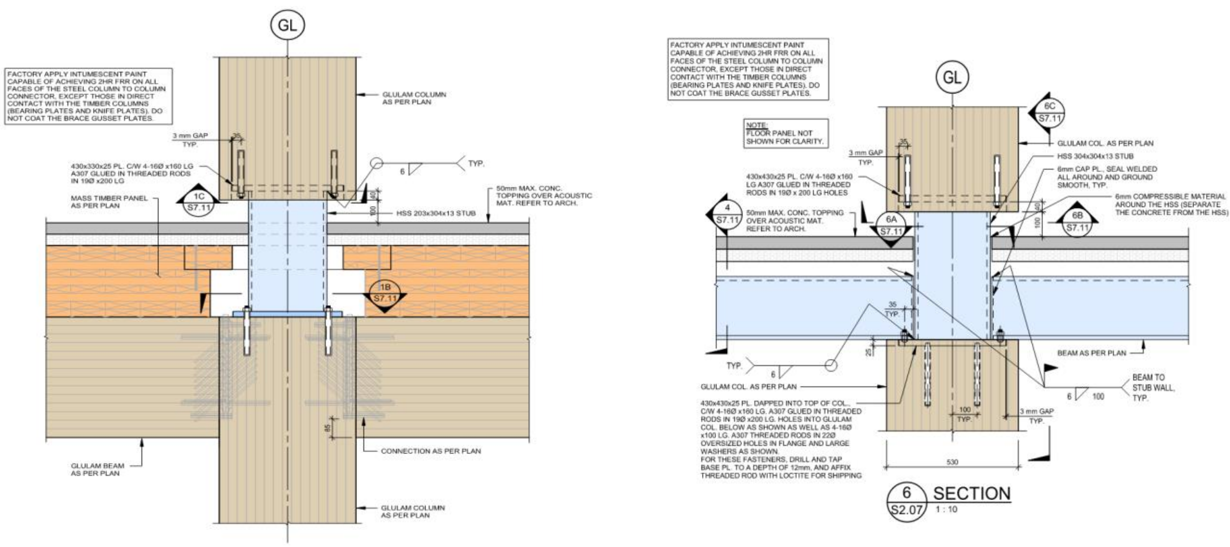

Figure 2. Column-to-Column and Beam-to-Column Connections. Perimeter drop beam connection (left); interior flush steel beam connection (right)

LATERAL SYSTEM

A primary project goal was to provide a fully exposed timber lateral system. Timber brace frames and CLT shearwalls are considered accepted lateral systems within the National Building Code of Canada with defined ductility and overstrength values [2]. The structure height, however, exceeds the height restrictions for these lateral systems in the code. Therefore, an alternative lateral system that uses supplemental energy dissipation devices was used.

The lateral system consists of perimeter timber brace frames and interior rocking CLT shear walls in the upper storeys, as shown in Figure 3. Resilient Slip Friction Dampers (RSFJs), supplied by Tectonus, were used at the end of each brace and as hold-downs at the bottom of the shearwalls, as shown in Figure 4 The RSFJ devices allow for energy dissipation to be localized to within the devices without damage to the other components of the lateral system.

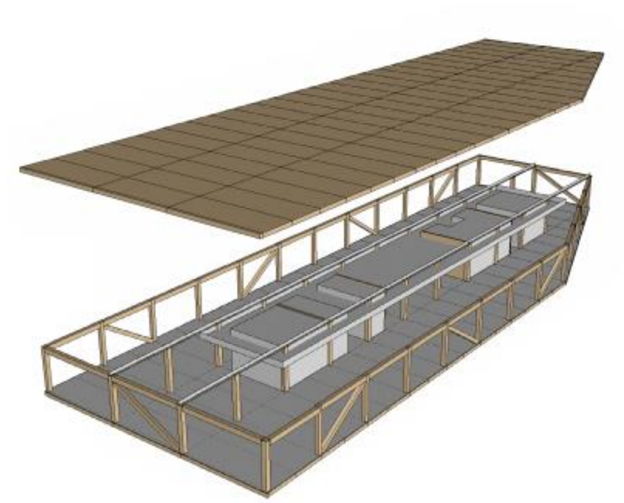

Figure 3. Typical Structure Section

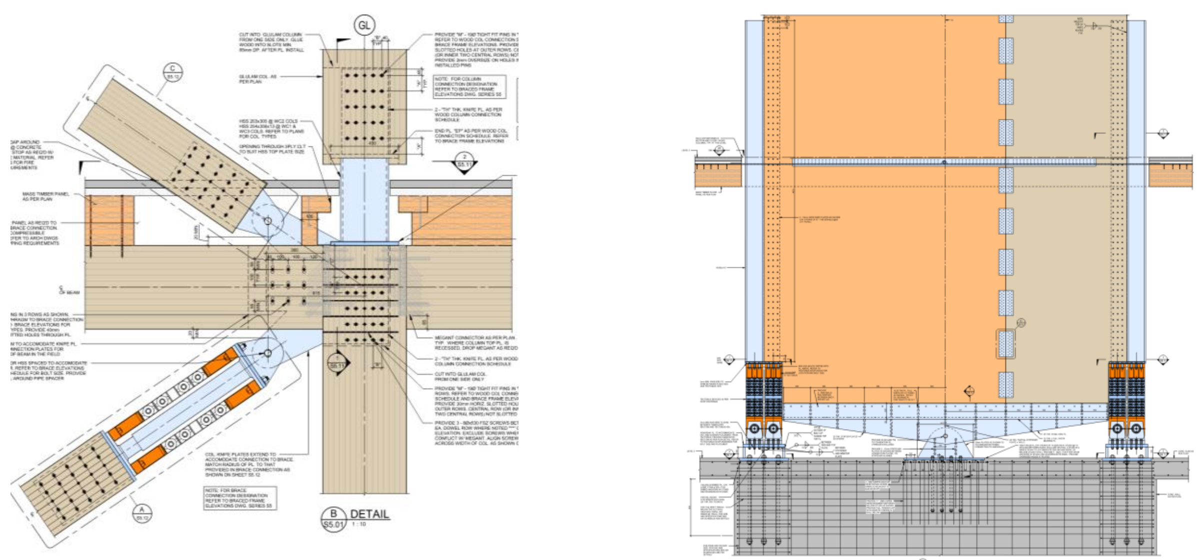

Figure 4. (a) Braced Frame Connection, (b) CLT Shearwall Hold-down

The RSFJ device consists of two outer plates (black) and two center plates (orange) with elongated holes. The plates are grooved and clamped together with high strength bolts and disc springs, as shown in Figure 5. When the applied force overcomes the frictional resistance between the grooved plate surfaces, the center plates will slide and dissipate energy through friction. At unloading, the force from the high strength bolts and compacted disc springs will overcome the frictional resistance of the grooved plates and the device will return to the at-rest configuration. This property helps to provide self-centering characteristics to the lateral system after a lateral event.

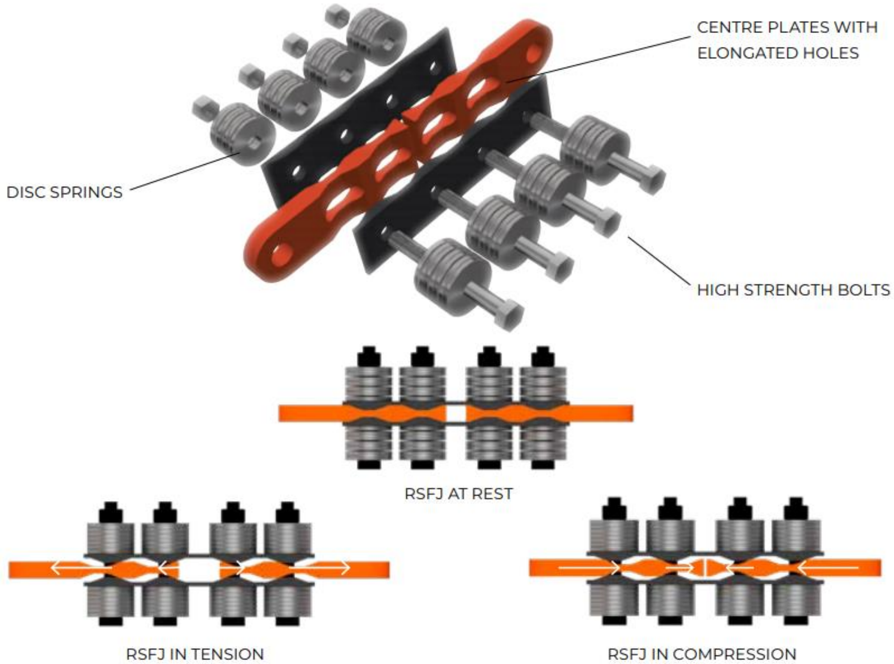

Figure 5. Tectonus RSFJ Device [3][4]

The primary cyclic behavior of the devices is characterized by a flag shaped hysteresis. Each point in the hysteresis (F_{slip}, F_{ult} , $F_{restoring}$, $F_{residual}$ and Δ_{ult}), as illustrated in Figure 6, is uniquely defined for each RSFJ device provided within the building. The device hysteresis can be tuned to the design force and deformation demands by modifying the number or angle of the plate ridges, and/or the number of bolts and disc springs at each bolt.

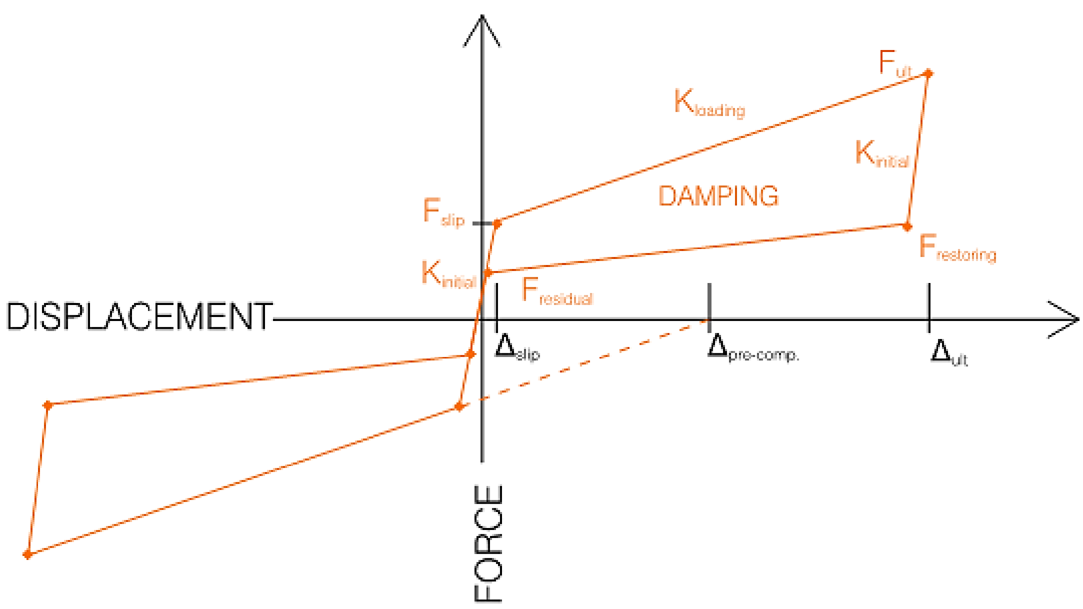

Figure 6. Typical Tectonus RSFJ Primary Hysteresis [3][4]

PROJECT CONTRAINTS

The primarily project constraint was to provide a fully exposed timber structure. The extent of timber in both the gravity and lateral system exceeds the code limitations, therefore a peer review and NLTHA was completed. The main considerations and performance objective covered in the peer review process included:

(a) Model Development

A 3D finite element model was developed and the modelling details and assumptions were peer reviewed. There is general consensus on the methods for modelling more conventional steel structures or concrete structures for NLTHA. The methodology for modeling timber building for NLTHA is still being developed. Key considerations for the model included appropriately modelling the timber frame elements, CLT walls, CLT diaphragms, the stiffness of timber connections, and the non-linear properties of the RSFJ devices. Iteration was required to calibrate the RSFH design properties to the design level earthquake.

(b) NLTHA And Performance Based Design

Three performance levels were considered for the building: life safety performance for the design level earthquake defined by the 2% in 50 year hazard; collapse prevention performance for seismic hazard greater than the design earthquake; and immediate occupancy performance for the service level earthquake defined by the 40% in 50 year hazard.

(c) Fire and Wind Stability Study

Due to the extent of exposed timber in the building, the load case including a wind event following a fire was considered for collapse prevention performance. The three primary considerations for the peer review and performance based design are summarized in the following sections.

MODEL DEVELOPEMNET

Lateral analysis was performed using a 3D finite element model in ETABS 18 Ultimate. The glulam and steel members were included in the model using elastic frame elements.

The CLT shearwalls and CLT diaphragms were modeled as shell elements that were calibrated to a separate detailed model in RFEM with RF-Laminate to determine strength and stiffness properties for these assemblies.

Non-linear friction spring damper type links at the brace ends and the shearwall hold-downs were used to represent the RSFJs. Seven types of hysteresis links defined the RSFJs at the glulam braces (Type 1 through Type 7) and one hysteresis link was used for hold-down RSFJ, as shown in Figure 7.

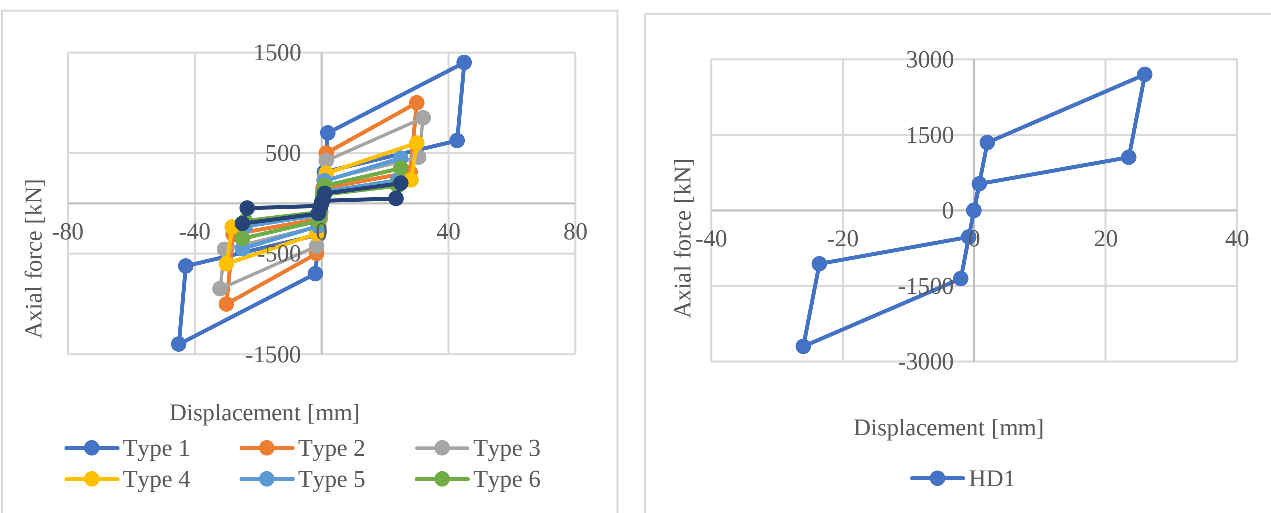

Figure 7. RSFJ Hystersis for (a) Braces (b) Holdown

The glulam beam-to-column connections were modeled with an elastic rotational spring stiffness based on a full-scale testing program. The program was completed as part of the project to understand the rotation capacity of the megant connectors [5]. The interior steel beams were included in the model and were released at both ends of each beam element for all analysis cases except the fire case wind analysis described in the section below.

The concrete substructure was included in the NLTHA model. Sensitivity studies of the strength, stiffness and mass of the substructure were completed to understand the effects on dynamic response of the NLTHA numerical model.

NLTHA AND DESIGN

Because the lateral system is outside of the code, the city agreed that a peer review with NLTHA for seismic would be an acceptable alterantive solution for the building design. As part of the peer review process, it was decided that the building should be:

i) designed for the structural response at the baseline Uniform Hazard Response Spectrum (UHS) level of 2% in 50 years.
ii) assessed for collapse prevension at higher hazard levels past 100% of the UHS design level.
iii) checked at the 40% in 50 years service design level for immediate occupancy.

Design UHS Level

The target building performance at the design UHS level was to satisfy life safety requirements with a low probability of collapse by restricting inter-storey drift to 2.5% (Clause 4.1.8.13). Following the design earthquake, no damage is expected in the braced frames and the shear walls as all timber components have been capacity protected. All yielding and energy dissipation will occur in the Tectonus devices.

For the design of the building, NLTHA was performed using fifteen ground motion sets that each included two horizontal and one vertical acceleration records linearly scaled to the design earthquake level (2% in 50 year). The two horizontal records were applied in two orientations rotated by 90 degrees, resulting in a total of thirty time history load cases. The average responses for the top five records were considered for the acceptance criteria. The design responses were compared against the limits for storey drift, deformation-controlled elements, and force-controlled elements. Iteration of the NLTHA was required to precisely tune the RSFJ properties for the structural response and to limit overstrength of the RSFJ.

The lateral system was designed so that the energy dissipation only occurs in the RSFJ acting as the LFRS fuse at the design level UHS. Capacity design principles were applied to the main framing members of the SFRS, the connections and the diaphragms. Overstrength of the RSFJ is defined as the ratio of the probable capacity of the RSFJ devices (Fult) over the maximum force in the RSFJ from the NLTHA.

The overstrength of the RSFJ was applied directly to the following elements:

- the brace members and connections;
- the beam members and connections;
- the CLT shearwall panels;
- the CLT shearwall base-shear connections;
- the RSFJ hold-down connections;
- CLT diaphragm-to-clt wall connections.

The diaphragm, including the chord and collector elements, was designed based on the maximum defined overstrength at each level.

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. The column design loads are determined by combining the SSRS overturning earthquake with appropriate dead, live, and snow loads.

The concrete structure (Parking Level 4 to Level 2 floor) was modelled separately from the timber structure above using linear static analysis. Backstay effects were studied by following analysis procedures described in Cl. 21.5.2.2.9 and N21.5.2.2.9 of CSA A23.3-14.

Collapse Prevention UHS Level

A collapse prevension sensitivity study was conducted to assess the stability of the building past 100% of the UHS. NLTHA was performed for ground motions scaled to 130% and 150% UHS. The stability of the structure was assessed by reviewing the displacement and force NLTHA response in the Tectonic devices and the maximum inter-storey drift responses.

The hysteresis assigned to the device was altered in the FE model to capture the response of the device at higher drift and force demands than the design ultimate displacement and force. After the device reaches the ultimate displacement the RSFJ secondary fuse is activated where the clamping rods in the dampers yield in tension. If the device is pushed past the "stop displacement" of the secondary fuse (over $1.5\Delta_{ult}$ and $1.25F_{ult}$) the device behaviour is governed by a bolt's in shear response. Figure 8 shows a schematic of the full hysteretic response defined. The stop displacement is defined for the devices in the numerical model at $1.5\Delta_{ult}$. Due to limitations in the numerical hysteretic damper model, the softening loading stiffness ($0.8K_{loading}$) observed for the device in the secondary fuse could not fully be captured. Figure 8 shows the empirical loading stiffness in the secondary fuse in a dashed line and the modeled loading stiffness in a solid line. Due to this difference in the numerical model when the device reaches $1.5\Delta_{ult}$ the force in the device is approximately $1.3F_{ult}$ rather than the empirically observed $1.25F_{ult}$. The point where the devices completely fail would occur in the last elastic phase. The hysteretic numerical model does not have a complete failure point, therefore would not become instable in the NHLTA response. The force and deformation demand provide an indication for when the devices would fail and where instability would occur.

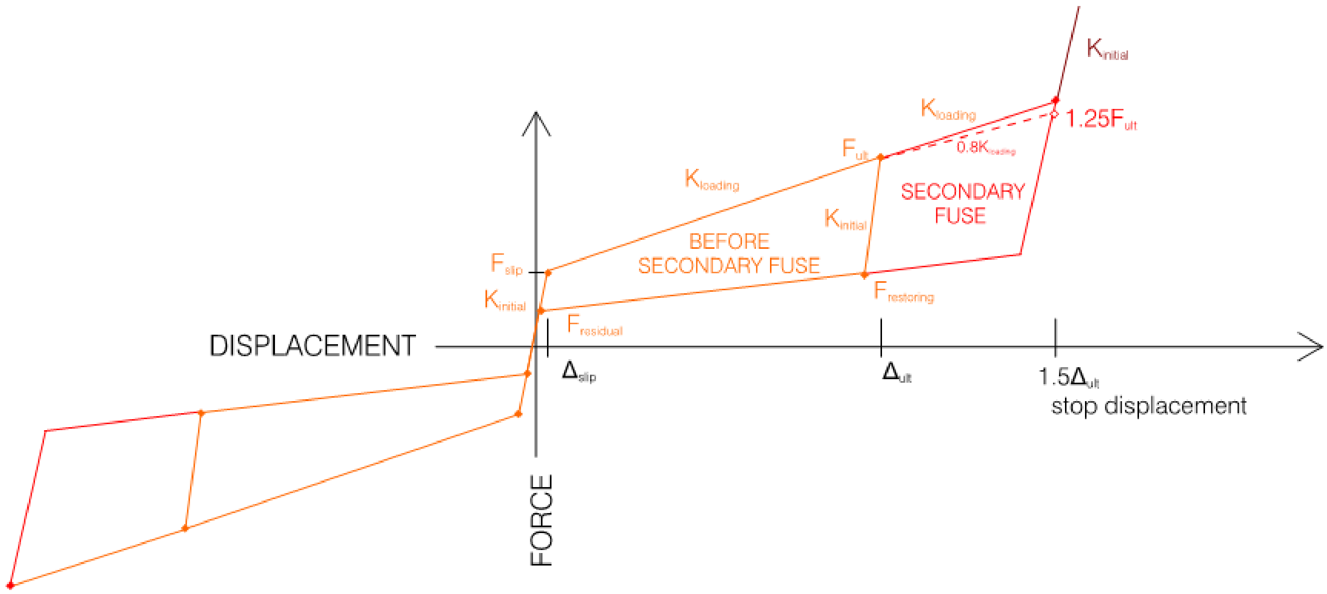

Figure 8. Typical Tectonus RSFJ Primary and Secondary Hysteresis [3][4]

The brace and hold-down link forces and deformations were in the NLTHA were compared to the defined link stop displacement ($1.5\Delta_{ult}$) and stop force ($1.25F_{ult}$). The RSFJ devices are regarded as fully stable below the defined link stop displacement ($1.5\Delta_{ult}$) and stop force ($1.25F_{ult}$). The behaviour of the RSFJ devices pushed past the secondary fuse is governed by bolt shear where brittle failure would start to occur. Failure of the device is an indicator of potential instability and building collapse. The NLTHA brace and hold-down link forces and deformations were reviewed based on the mean of the top 5 link responses (Δ and F) which was considered the main acceptance criteria for the collapse prevention check.

It is confirmed that the mean of the top five link forces and deformations are at or below the defined link stop displacement and stop force for the 130% UHS case within acceptable tolerance. Therefore, the system meets the acceptance criteria for 130% scaled design records.

The link deformations were at or below the defined stop displacement ($1.5\Delta_{ult}$) for the mean of the top five NLTHA responses in the 150% UHS case. However, the force demand exceeded the $1.25F_{ult}$ for the 150% UHS case in several links. The force exceedance could represent failure of the RSFJ device. The mean of the top five response for several devices distributed over all the levels are approximately 10% to 30% over the $1.25F_{ult}$ capacity but remain below the $1.5\Delta_{ult}$ value. Force demand/capacity values remain consistent throughout the floors which suggests that structure is not likely to develop a soft story failure mechanism. As the failure is based more closely on the deformation causing a combined tension and shear failure mechanism, it is unclear if the connections would fail at this loading level. Our analysis does show an even distribution of loading within the units which does not indicate an instability condition or a force concentration.

The drift response from the NLTHA scaled to 130% and 150% was 2.2% and 2.4% drift, respectively. These are within the code-specified drift limits of 2.5% to achieve a low probability of collapse (BCBC Clause 4.1.8.13 [6]) and the glulam beam-to-glulam column connection drift capacity. Therefore, the stability of the building under this check was considered acceptable.

Service UHS Level

Under a service level earthquake, the building is expected to be occupiable with some minor cosmetic damage to non-structural elements. The service design level applied to the model using Response Spectrum analysis and it was confirmed that the forces and deformations in the brace and hold-down links do not exceed the elastic limit.

POST-FIRE LATERAL ANALYSIS

Concern for the potential for a wind event after a full duration fire was raised during the peer review process. This load case is not included in the building or material codes or commentaries, however the project team considered it good practice to include for this building where the lateral stability is provided by exposed wood systems.

The stability of the building was reviewed for an additional wind load case under 1.25DL (or 0.9DL) + 0.5LL + 1.4WL, with a 1/10 year wind load demand following a full 2-hour fire event. All the glulam braces were assumed to be lost within the building before the fire was extinguished. This is considered an extremely conservative scenario as it is unlikely all the braces would be completely ineffective within the building after a fire.

Linear static analysis was completed with a revised finite element model. The glulam beams and columns sizes were reduced by the 2-hr char thickness and the CLT shear wall thickness were reduced by the 2-hr char thickness on one the exposed side. The updated member sizes are provided in Table . Hold-down devices were not affected as these devices were encapsulated.

Table 1: Member sizes after 2-fire

Element	Original Size	Charred Size
Beam 1	315mmx530mm	133mmx348mm
Beam 2	365mmx530mm	183mmx348mm
Column 1	530mmx530mm	348mmx348mm
Column 2	430mmx530mm	248mmx348mm
Column 3	530mmx568mm	348mmx386mm
CLT Shearwalls	245mm	140mm
CLT Floor Panels	315mm	212mm

In the North-South direction lateral stability was provided from the charred CLT shear walls and nominal moment frame behaviour at each end frame.

In the East-West direction the lateral stability of the building relied mostly on the beam-to-column connection rotational stiffness that essentially creates four long moment frames. The rotational stiffness was included in the numerical model for the two main frame connections: the glulam beam-to-glulam column connection and the steel beam-to-glulam column connection, as shown in Figure 2.

The glulam beam-to-column connection were assigned an elastic rotational spring with stiffness consistent with the NLTHA model as discussed in section above on Modelling Approach. The steel beam-to-glulam column connection was assigned a conservative lower bound stiffness approximation of the connection.

The maximum inter-storey drifts are found to be 1.9% in E-W direction and 0.07% in N-S direction. This was deemed adequate for structure's short-term stability after a fire event as the interstorey drift is less than that of the capacity of the MEGANT connectors and is within the maximum allowable interstorey drift defined in the NBCC 2015 for an normal importance building (2.5%).

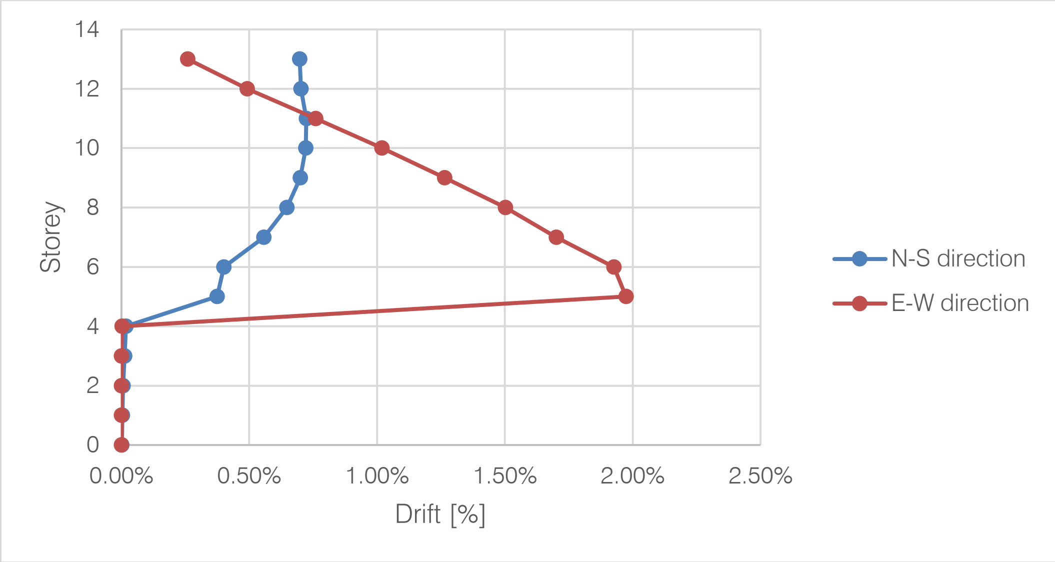

Figure 9. Inter-storey drift from 1/10 yr SLS wind load after fire

The forces and deformation within the frame elements acting as a moment frame were checked against the expected capacity. The column and beam elements have less than a 50% utilization in the 1.25DL (or 0.9DL) + 0.5LL + 1.4WL loading case.

CONCLUSIONS

Keith drive is an example of an innovative structure that primarily lateral system falls outside the code. Through peer review and NLTHA the project team was able to design a code accepted under alternative solution by the city. Performance based design was used to achieve a repairable building after the design earthquake. Additional loading cases and hazards, including a seismic event larger than the design earthquake, service level earthquake and a wind even following fire were completed to ensure that the expected performance of the structure met the owner's and city's objectives.

ACKNOWLEDGMENTS

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