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NONLINEAR DYNAMIC ANALYSIS OF INNOVATIVE HIGH R-FACTOR HYBRID TIMBER-STEEL BUILDINGS

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ABSTRACT: Two innovative hybrid steel-wood seismic force-resisting systems have been developed that utilize the energy dissipation capabilities of advanced structural steel systems to improve the seismic performance and design of heavy timber structures. Conceptual designs of these two systems are presented: first, a buckling restrained braced frame adapted for wood buildings using glued-in rods; and second, a ductile and replaceable steel reduced beam section connection adapted for wood buildings using a steel panel zone and self-tapping screws. To investigate the dynamic behavior of these systems, nonlinear time-history analyses was performed on four six-storey structures designed for Victoria BC: two using the new hybrid steel-wood designs and two using conventional steel-only designs. These analyses were conducted using OpenSees. Each structure is subjected to 44 maximum considered and 44 design basis earthquakes using FEMA-P695 ground motions. Comparable seismic performance was observed for the hybrid and steel-only frames. Weight reduction due to the use of wood resulted in significantly lower base shears for the hybrid structure. Average interstorey drifts remained within allowable limits of 2.5% of storey height as specified in the NBCC (2010) for both the buckling-restrained braced and moment-resisting frames. Storey accelerations were similar for both the hybrid and steel-only frames.

1. Introduction

The high strength-to-weight ratio of wood allows wood structures to be potentially lighter than equivalent concrete or steel structures. This low weight makes wood buildings attractive for use in seismically active regions; however, existing lateral force resisting systems for heavy timber buildings typically have significantly less ductility than seismically-designed steel or concrete buildings. This results in lower force modification factors (R-factors) for wood structures, which, in turn, result in comparatively large seismic design forces, negating the inherent benefit of wood's low weight.

To close this gap, hybrid structures may be used to combine the benefits of timber and steel or concrete. Limited research exists on full scale connections for hybrid systems that provide adequate strength and ductility for seismic design. Popovski et al. (2003 and 2008) have conducted investigations into braced heavy timber frames with a focus on energy dissipation from connection fatigue in frames restrained by heavy timber or glue-laminated bracing elements. However, research into the implementation of steel bracing systems into heavy timber has yet to be investigated. A hybrid timber-steel moment connection using glued-in rods and steel end-plate connections was developed and tested at a small scale by Andreolli et al. (2011); high ductility and energy dissipation was achieved in some circumstances due to end-plate yielding, however depending on the end-plate design the connection sometimes experienced brittle rod pullout failure, limiting ductility and rendering the connection non-replaceable. Additionally, a column-base joint was devised by Humbert et al. (2014) that used steel connectors to increase the ductility and energy dissipation of heavy timber moment frame joints, however strength and ductility were sometimes limited by splitting of the wood member.

In this study, modern timber fasteners and steel detailing are used to significantly increase the ductility of hybrid timber-steel seismic force resisting systems. High performance steel yielding elements are used as localized seismic fuses within a predominantly timber structure. Both the connections between the timber and steel elements and the timber elements themselves are capacity designed to remain elastic during any seismic event. This allows the structures to combine the low weight of timber with the ductility of steel and high seismic force modification factors associated with steel designs.

Two different types of innovative hybrid steel-wood seismic force resisting systems are studied. The first is an adaptation of steel buckling-restrained braces within a timber braced frame and the second is the adaptation of well-detailed steel moment yielding fuses within a timber moment frame. Buckling restrained braces (BRBs) are axial braces designed to yield and dissipate energy in both tension and compression (Della Corte et al. 2011). To enable the brace to yield in compression, a BRB uses a restraint mechanism to laterally support an inner steel yielding core to prevent buckling. In steel moment-resisting frames (MRFs), high ductility may be achieved through plastic hinging in beam elements by using a reduced beam section (RBS) connection first developed by Plumier in the 1990's (Bruneau et al., 1998). Research by others has since resulted in prequalified guidelines for the design of such connections (Chi and Uang, 2002; CISC, 2008). The reduced beam section detail reduces the beam flange width locally at a preferred hinge location, moving the beam's yield location away from the brittle connection components at the column face. The same result is achievable by using a smaller section bolted to the beam element using end plate connections (Shen et al., 2011). Shen's connection was also designed to be replaceable, and can be quickly replaced following a severe ground motion, lowering the time and cost of post-earthquake repairs.

This paper will present proposed details for the two innovative modern hybrid timber structures, which will be experimentally tested in a separate study: one using a hybrid steel-timber buckling restrained braced frame (BRBF) and one using a hybrid steel-timber ductile moment frame (DMRF). The wood elements in both of these systems are glued laminated (glulam) timber beams and columns. Time history analyses of two full six-storey building frames, utilizing these new hybrid timber-steel designs, using the nonlinear analysis package OpenSees (McKenna et al. 2000) will be described. In addition, two full six-storey steel-only building frames, using comparable steel BRBFs and DMRFs, were also designed and analysed for comparison with the timber structures. Results from the hybrid and steel-only buildings will be compared to assess the feasibility of high force reduction factor (R-factor) hybrid buildings. The analyses were performed under two earthquake hazard levels, maximum considered earthquake (MCE) and design-basis earthquake (DBE) having approximate probabilities of exceedance of 2% in 50 years and 10% in 50 years, respectively. The numerical analyses showed that the hybrid and steel-only structures achieved similar performance at both hazard levels. It will be shown that the hybrid structure had similar seismic performance to the steel-only building with respect to interstorey drift, residual drift and storey accelerations, as well as considerable reduction in seismic weight and base shear.

2. Innovative Hybrid Steel-Wood System Designs

When subjected to significant ground excitations, connections in heavy timber structures are prone to bearing failure, excessive fastener yielding and splitting perpendicular to the grain direction. The new systems presented here were designed to avoid such failures in the wood members so that the wood elements in the hybrid system could be easily capacity designed to localize all damage and energy dissipation to the fuse elements.

For the hybrid steel-wood BRB frame design, the beam-column-brace interface joint is replaced by a steel joint assembly as shown in Fig. 1. In a braced frame, the loads in the structural elements are predominantly axial. This design uses glued-in rod (GIRods) connections at the interfaces between the steel and wood elements. These connections are typically comprised of a single or group of threaded steel rods, embedded into the end-grain of heavy timber members by structural grade adhesives. GIRods predominantly resist axial loads, transferred along the embedment length of the fasteners. This enables the transfer of forces into the strong, parallel to grain direction of the glued laminated (glulam) beams and columns. The GIRod connection, steel assembly, and wood members are all capacity designed, based on the behaviour of the brace, to remain elastic during an earthquake. The GIRod connections were designed using the worst-case of the GIROD Project and the LICONS Report Eurocode 5 design proposal, with an additional check completed using the method outlined in the German Design Code (Bengtsson et al. 2002, Connolly et al. 2003, NABau 2004). This braced frame connection is also applicable to other types of bracing systems in addition to the BRB, including friction braces, viscous dampers, and self-centering braces.

For the hybrid steel-wood moment frame design, existing high performance ductile steel fuses are incorporated into heavy timber frames by again replacing the beam-column interface joint with a steel joint assembly as shown in Fig. 2. Since these connections must transfer considerable shear and moment between the beam and column in addition to axial force, the connections between the wood and steel elements use fully threaded self-tapping screws (STS). These screws can easily be installed at an angle to the grain and have high withdrawal capacity and axial stiffness. Limited research exists on the use of STS as a primary load transfer mechanism; however, a beam-column joint system using STS was developed by Closen and Lam (2012). The system used a connection between the steel and heavy timber beam that was similar to that used in this study. Their connection was capable of achieving high strength, but ductility was limited by excessive longitudinal shear and tension perpendicular to grain stresses that led to premature brittle splitting failure in the wood panel zone. In this study, STS were similarly used to form a high-strength moment connection between timber and steel elements as shown in Fig. 2.

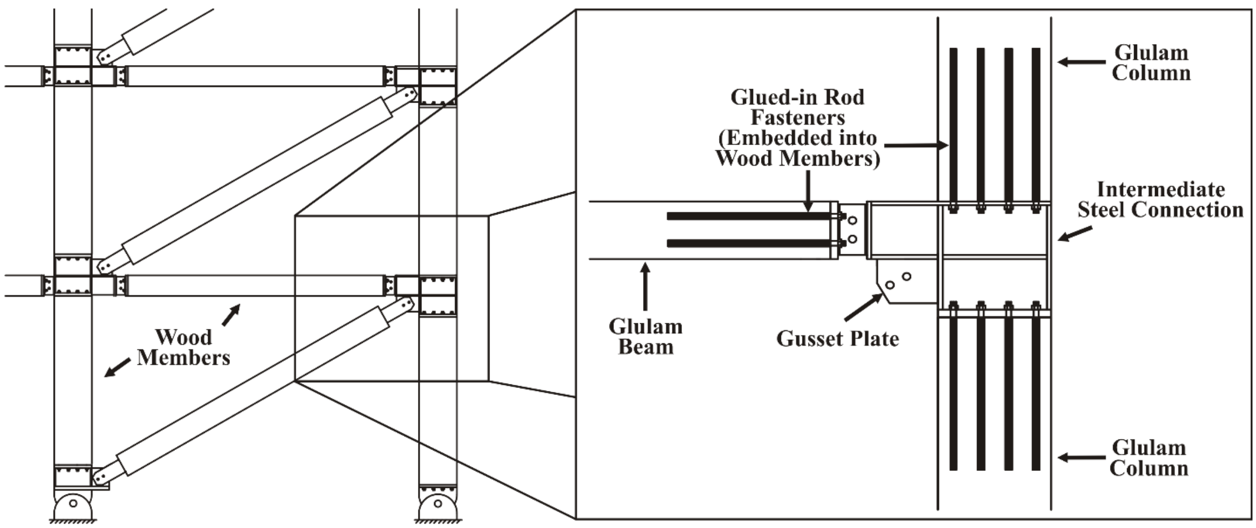

Fig. 1 – Hybrid Steel-Timber Brace Connection

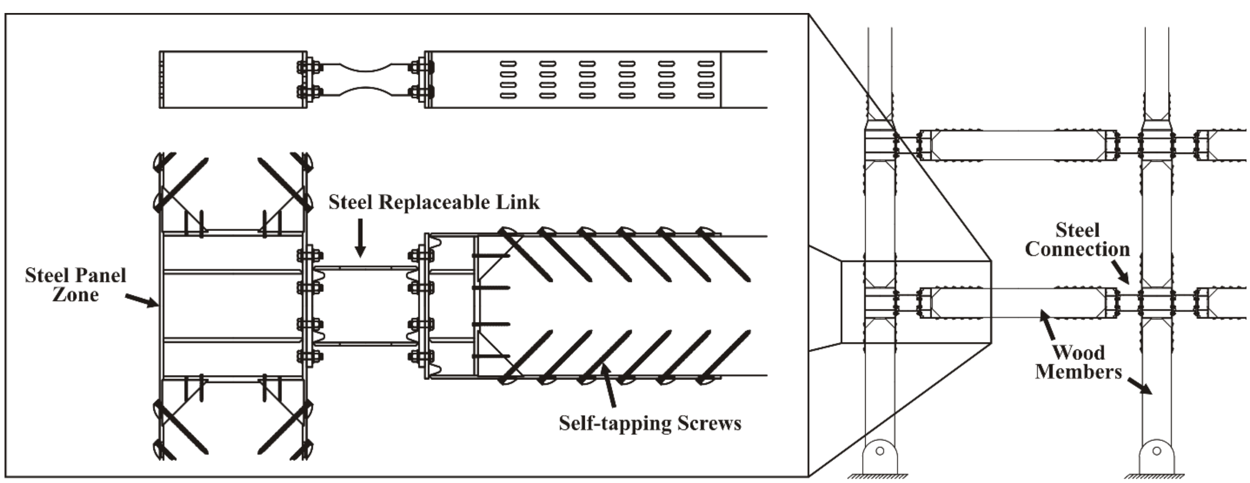

Fig. 2 – Hybrid Steel-Timber Moment Connection with Nonlinear Replaceable Link

The STS connections and wood elements have been capacity designed to remain elastic during an earthquake. If the column were a continuous wood member, it would be susceptible to failing due to shearing of the panel zone or bearing stresses caused by the beam connection, as discussed above in the context of Closen and Lam's (2012) connection. Therefore, the panel-zone area of the column was replaced by a custom steel panel zone resembling that of a steel-only MRF, which is also designed to remain elastic. In addition, a reduced beam section (RBS) link has been included within the connection as shown in Fig. 2 to provide a stable energy dissipating mechanism. The only inelastic part of the assembly is the yielding replaceable RBS link, which will accommodate all of the inelastic demand during an earthquake. The timber beam is connected to this link using a steel spacer section to allow a replaceable bolted end-plate detail to be used. The wood connections are designed to avoid the formation of moisture induced shrinkage cracks at the fastener locations.

3. Building Design

For this study, two full six storey buildings were designed. Each building has a moment-resisting frame (MRF) in the east-west direction, and a buckling-restrained braced frame (BRBF) in the north-south direction, as shown in Fig. 3. One building was designed using the two new hybrid steel-timber systems, and the other was designed using similar systems in a steel-only structure. Both buildings had identical geometry to facilitate comparison and they were both designed for Victoria, BC. The storey height was 3700 mm and the bay width was 6500 mm in both directions. The steel MRF used RBS connections, while the hybrid steel-wood MRF incorporated the specially designed hybrid system using steel nonlinear replaceable RBS links as shown in Fig. 2. The layout of the structural elements is shown in Fig. 3.

Both buildings were designed using the response spectrum analysis method as specified in the NBCC (2010), accounting for both accidental torsion and P-delta amplification. The design spectrum for Victoria was determined assuming site class C. The steel structure was assumed to have concrete floor slabs on all levels, while the timber structure used a 131-5s cross-laminated timber (CLT) one-way slab floor system. This significantly reduced the seismic weight of the hybrid structure relative to the steel building, which resulted in lower seismic design forces as shown in Table 1. Table 1 also provides the design period for each structure.

Table 1 – Seismic Design Properties of Prototype Structures

Structure	Frame	Seismic Weight W (kN)	Design Period T (sec)	Design Base Shear V_d (kN)
Wood Building	BRBF	18 700	1.11	1110
	MRF		0.90	933
Steel Building	BRBF	31100	1.11	2130
	MRF		1.30	1060

Since the ductile mechanisms in both the hybrid and steel-only structures are steel, identical R_d and R_o factors were assigned to both. This reflects the fact that all of the plastic behaviour will occur in the ductile BRBs and RBS links. The values for R_d and R_o were 4.0 and 1.2, respectively for the Ductile Buckling Restrained Braced Frames, and 5.0 and 1.5, respectively for the Ductile Moment-Resisting Frames. The elastic elements (connections, beams and columns) were capacity designed to protect the brittle glulam members. Experimental validation of the hybrid connection system is ongoing to confirm that this R-value is appropriate for such hybrid timber-steel systems.

For the capacity design of the BRBFs, an overcapacity of 10% was assigned to the probable tensile and compressive brace forces, as dictated by CSA S16-09 (2010). The structural member sizes and yielding core areas of the BRBF frame are provided in Table 2 below.

The moment resisting frames were designed using the strong column-weak beam design approach provided in CSA S16-09 (2010). Both the hybrid and steel MRFs used RBS connections with radial flange cuts. These connections were designed using the Moment Connections for Seismic Applications guidelines (CISC 2008). A fifty percent flange width reduction was assumed at the centre of all radial flange cuts. The RBS design parameters are provided in Table 3. Appropriate doubler and continuity plates were used to help ensure elastic behaviour of the panel zones during severe earthquakes, according to both CSA S16-09 (CSA 2010) and CISC (2008) guidelines. For most stories of both frames, it was necessary to increase beam member sizes to meet design drift requirements, since MRFs are inherently flexible the design of the beams was typically governed by drift criteria as opposed to strength. For the steel-only MRF, this necessary beam increase for stiffness resulted in higher capacity-design forces since the strength of the RBS is proportional to the beam size. However, in the hybrid system the strength and drift designs were uncoupled due to the replicable link. Therefore, the wood beams could be increased without effecting the RBS link sizes, resulting in a more economical design. The member sizes for each MRF frame are provided in Table 3.

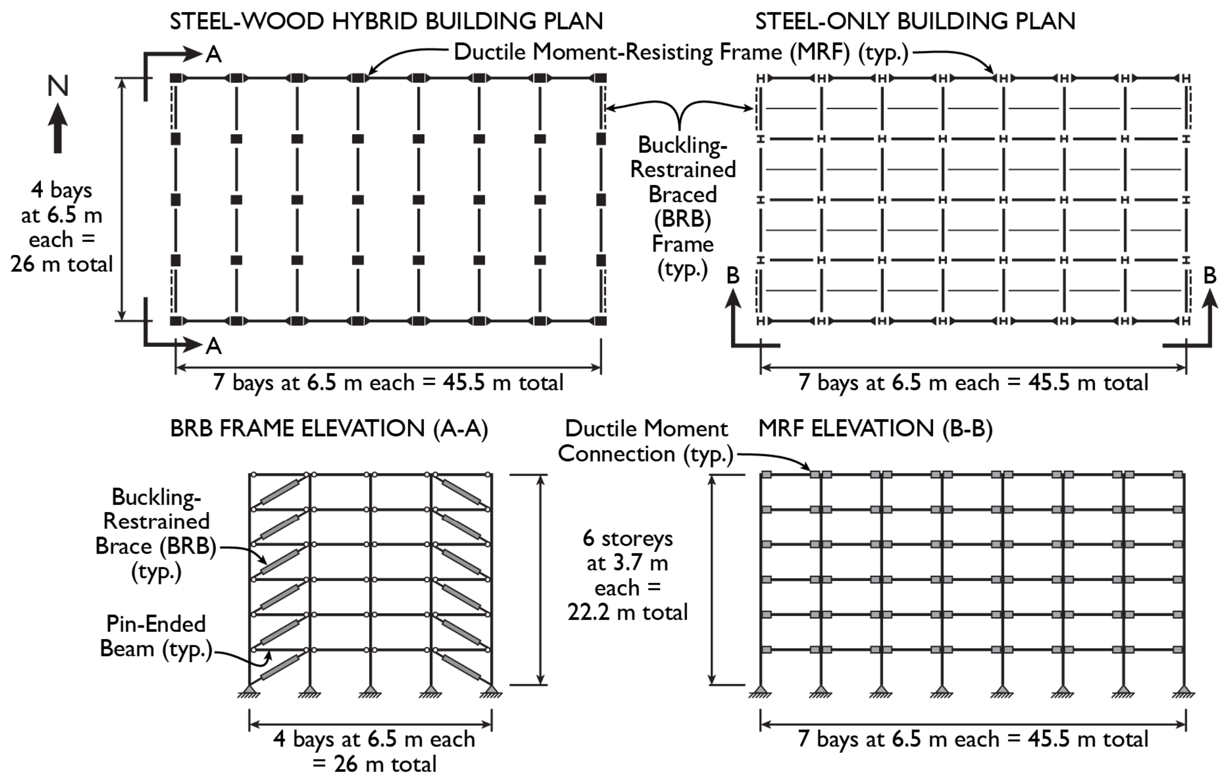

Fig. 3 – Plans and Elevations of Building Designs

4. Modeling

Nonlinear dynamic time-history analysis of each building was performed using OpenSees (McKenna et al. 2000). Full-scale 2-D frame models were constructed to evaluate their seismic performance when subjected to earthquakes at the MCE and DBE hazard levels, scaled to meet the NBCC design spectrum for the Victoria region. Leaning gravity columns were assigned the building weight and gravity loads not directly tributary to the studied frames to account for inertial forces and P-delta effects. The gravity loads and frame mass tributary to the frames were lumped at the appropriate column nodes on each storey. The concrete and CLT slabs were modeled as rigid diaphragms by constraining horizontal displacement of the frame nodes at each storey to the leaning column node at that storey. Large displacement analysis was considered to account for p-delta effects through the application of P-delta geometric transformations on all columns. In the BRB frame, rigid joint offsets were used to simulate the rigidity of the intermediate steel connections at the beam-column interfaces. Rayleigh damping of 3% was considered in modes 1 and 6, which is consistent with previous studies conducted on steel structures, which is also a conservative assumption for wood structures (Welch et al., 2014). As previously discussed, the frames were capacity designed so that all beams and columns remain elastic. For this reason, in the models these members used elastic beam column elements with strength and stiffness properties based on the designed member properties.

Buckling-restrained braces (BRBs) were modelled using spring elements. The uniaxial Giuffre-Menegotto-Pinto material in OpenSees was used to represent the steel brace core because it accounts for kinetic and isotropic strain hardening properties of steel. The strain-hardening and isotropic hardening parameters for the BRBs in the models were calibrated based on the experimental results of Trembley et al., (2004). Increased strength in the compressive direction due to friction effects was not considered. The initial series stiffness of the BRB was calculated based the geometry of the inner steel yielding core. The geometry and lengths of the BRB core and connecting elements were determined based on brace geometry equations and industry modeling information provided by Star Seismic.

Table 2 – Buckling-Restrained Brace Frame Member Sizes and Brace Properties

	HYBRID FRAME				STEEL-ONLY FRAME			
Storey	Glulam Columns		Glulam Beams	BRB Core Area (mm^2)	Steel Columns		Steel Beams	BRB Core Area (mm^2)
	Exterior	Interior			Exterior	Interior		
1	265x570	465x456	532x215	1140	W360x122	W250x131	W410x46	2042
2	215x418			1040				1682
3	175x418	465x380	418x215	920	W360x64	W200x100		1487
4	175x342	342x315		795				1306
5		228x215		720	W360x51	W200x52		1111
6			342x215	510			W310x21	796

Table 3 – Moment-Resisting Frame Member Sizes and RBS Properties

Storey	Glulam/Steel Columns		Glulam/Steel Beams	RBS Link			
	Exterior	Interior		Section	a* (mm)	s* (mm)	c* (mm)
HYBRID FRAME							
1	265x570	265x684	265x684	W460x60	95	295	38
2	215x418	265x532	175x570	W360x39	75	230	32
3	175x418	175x532	175x494	W360x33			
4	175x342	175x494					
5		175x456	175x418	W250x25	70	170	25
6			175x380	W250x22			
STEEL-ONLY FRAME							
1	W360x122	W360x216	W610x101	W610x101	114	392	57
2			W460x52	W460x52	76	293	38
3	W360x64	W360x91	W410x46	W410x46	70	262	35
4							
5	W360x51	W360x72	W360x39	W360x39	64	230	32
6			W310x33	W310x33	51	203	25

**a* is the distance between the column face and the beginning of the RBS
s is the length of the radial flange cut, *c* is the width of flange reduction (per side)

When modelling the MRFs, only the replaceable link elements and the panel zone were assigned a nonlinear behaviour. These elements used a distributed plasticity model with force-based beam-column elements. For the distributed plasticity, the behaviour of each integration point along the element was evaluated using a fibre section model. The flange width along the RBS part of the link was assumed to be the flange width at the centre of the reduced section. The optimized element configuration consisted of 5 integration points per element, containing 10 fibres across the web depth, and 3 fibres across each flange depth, determined through a sensitivity analysis. To model the plastic behaviour of the steel material, a bilinear elasto-plastic material was specified with strain hardening of 2% to capture the hysteretic behaviour of the element. The section located between the RBS and the column face was modelled using an elastic beam-column element. The panel zones in the MRF frames were modelled using the method suggested by Gupta and Krawinkler (1999).

5. Ground Motions Considered

The nonlinear time-history analysis study of the structural models used the full set of FEMA P695 far-field ground accelerations (FEMA P695, 2009). This far-field earthquake record set consists of two-directional horizontal ground accelerations recorded 10 km or more from the point of fault rupture. The earthquake record scaling procedure presented in the FEMA P695 documentation calibrates the ground accelerations based on the median spectral value, instead of calibrating the individual spectral intensities, to evaluate building performance (FEMA P695, 2009). The suite of 44 records was scaled such that the median spectral acceleration at the building first mode period for each 2D frame matched the design spectral acceleration for Victoria BC. at the corresponding period, as suggested by FEMA P695 (2009). These scaled records represent the seismic hazard level associated with the MCE. The records were then further scaled by a factor of 2/3 to attain an additional set of ground motions that approximates the seismic hazard level associated with the DBE.

6. Results and Discussion

Two-dimensional nonlinear dynamic analysis was conducted using the suite of 44 MCE level records and 44 DBE level records to compare the seismic performance of the six-storey hybrid and the six-storey steel-only structures. For the hybrid structures, both types of systems experienced collapse for only a single record: the MCE level 1979 Imperial Valley earthquake. The BRBF steel-only system also experienced collapse during this ground motion. Since none of the other 43 earthquakes caused collapse for any of the structures, this single record was removed from the calculations of the mean values of the response parameters to avoid skewing the results (since the drift level for a collapsed building is not defined).

The first and second mode periods, scale factors, and base shear of the two structures are compared in Table 4 below. It is important to note that the seismic weight of the hybrid structure is approximately 60% of that of the steel-only structure. This resulted in a reduction in total base shear of 59% and 51% in the hybrid BRBF and MRF respectively, relative to the steel-only structure. Lowering the seismic weight decreased the overall demands on each frame. This improves the feasibility of potentially expensive hybrid alternatives due to the reduction in lateral design loads and, hence, member size. In addition to the decreased weight, the lateral stiffness of the hybrid building is also lower than the steel building. This results in the two different building designs having similar periods in each direction. This variation had a direct effect on the earthquake scale factors of each system, since first mode periods where used to determine the scale factors for each frame independently.

Table 4 – Seismic Design Properties of the Modelled Frames

Structure	Frame	Periods		EQ Scale Factors		Mean Base Shear	
		T_1 (sec)	T_2 (sec)	MCE	DBE	$V_{b,\,MCE}$ (kN)	$V_{b,\,DBE}$ (kN)
Wood Building	BRBF	1.51	0.54	1.21	0.81	1510	1420
	MRF	2.63	0.95	1.46	0.98	1800	1550
Steel Building	BRBF	1.59	0.56	1.25	0.83	2560	2440
	MRF	2.51	0.89	1.41	0.94	3500	3020

The overall performance of the hybrid and steel-only buildings were similar with respect to the drift and acceleration demands. Fig. 4 shows the drift and acceleration of each frame in terms of the mean response from the DBE and MCE hazard level earthquakes in addition to the mean plus one standard deviation to account for variation in earthquake magnitudes. Overall, the mean from all the earthquakes of the peak interstorey drift responses (presented in Fig. 4a) remained within the allowable NBCC specified limit of 2.5% under both hazard levels (NBCC, 2010). For the BRBFs, the hybrid and steel-only structures had similar mean MCE-level peak interstorey drifts of 1.2% and 1.1%, respectively. The largest interstorey deformations occur in the lower stories of the BRBF due to drift localization caused by P-delta effects acting on the concentrated gravity loads in those stories. The DBE level response also showed similar drifts for the two types of BRBF structures. When observing the mean peak interstorey drift for the MFRs, more significant variation was seen between hybrid and steel-only frames. This is due to the reduced stiffness of wood as compared to steel, since MRFs depend primarily on the stiffness of the beams and columns to resist lateral movement. For the MRFs, the mean of the peak interstorey drifts at the MCE level was 2.5% and 2.4% for the hybrid and steel-only MRFs respectively. The DBE response of the hybrid and steel-only MRFs were more similar, and peak interstorey drifts were found to be 1.81% and 1.75% respectively. As expected, the BRBFs experienced significantly lower interstorey drifts than the MRFs, since the MRFs are a more flexible system and member selection is often governed by drift limits.

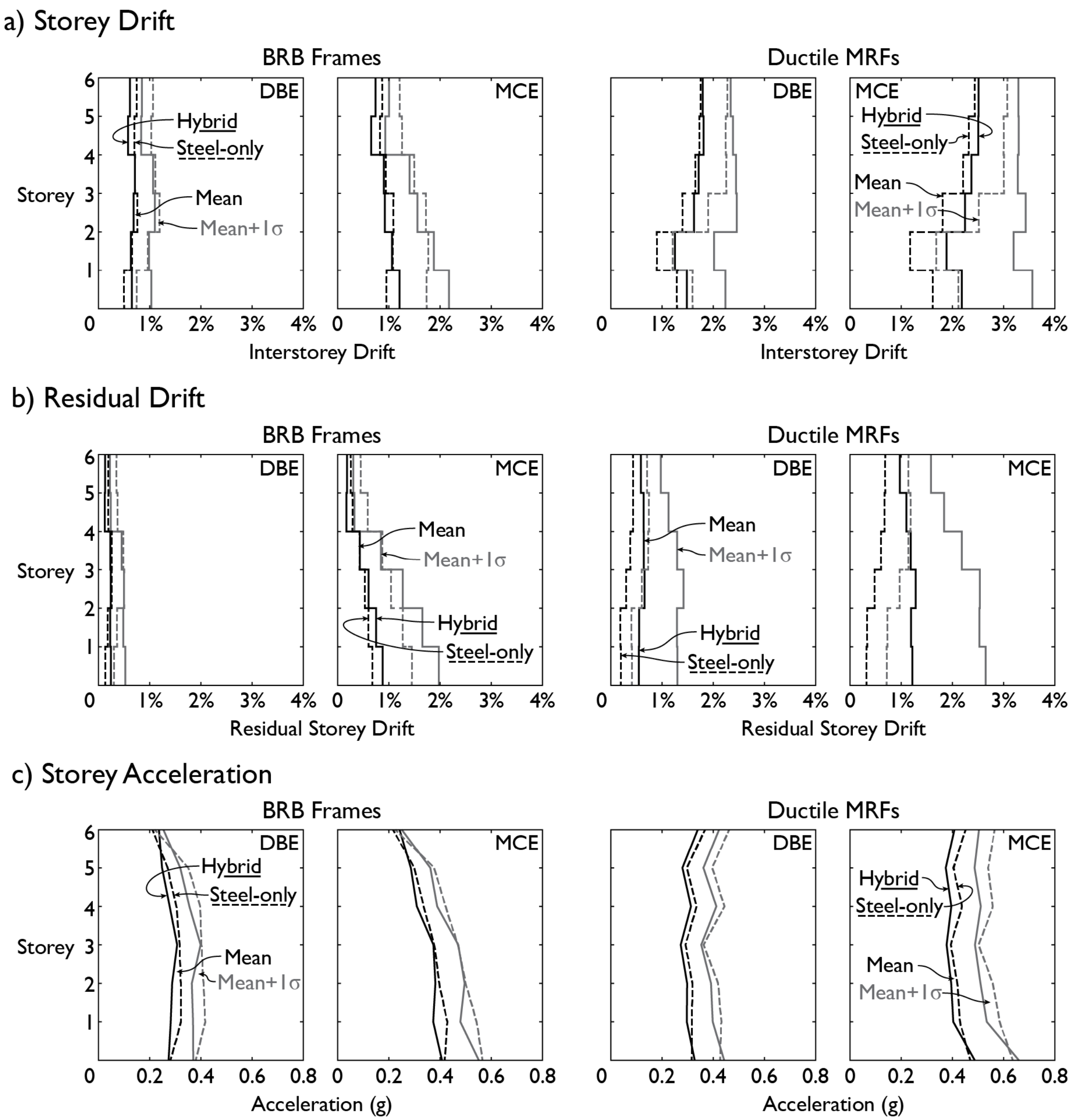

Fig. 4 – BRB Frame and MRF Response Plots: a) Interstorey Drift; b) Residual Drift; c) Storey Accelerations

The residual drift responses presented in Fig. 4b may be used to assess the post-disaster performance of the buildings. The DBE level ground motions resulted in small mean residual storey drifts, limited to approximately 0.25% for both types of BRBF. At the MCE level, these residual drifts increased to 0.9 and 0.7% for the hybrid and steel-only frames respectively. Overall, the hybrid BRBF exhibited equivalent performance with respect to residual drift to that of the steel-only system. The residual drifts were significantly larger in the MRF structures because these structures had larger peak interstorey drifts. The residual drift at the MCE level was 1.3% in the hybrid frame and 0.7% in the steel-only frame. When designing structures to meet NBCC drift limits, it is possible to develop high residual drifts without violating code-based design criteria since there is no simple method of approximating residual drift. Further refinement of the design will be completed in the future to attempt to limit the residual drift of the MRFs.

Mean absolute storey accelerations are plotted in Fig. 4c. The BRBFs and MRFs at both hazard levels experienced maximum absolute storey accelerations that were similar for the hybrid and steel-only systems. For the BRBFs, the frames were exposed to scaled mean peak ground accelerations (PGAs) of 0.41 and 0.42 g for the hybrid and steel-only structures respectively, resulting in peak storey accelerations of 0.38 and 0.43 g respectively. For the MRFs, the absolute peak storey accelerations at the MCE hazard level were found to be lower than the corresponding scaled average PGAs. The hybrid and steel-only MRFs experienced maximum mean storey accelerations of 0.48 and 0.45 g respectively. The distribution of accelerations throughout the height of the buildings were well controlled for both BRBF and MRF frame types, varying only slightly between adjacent storeys.

7. Conclusion

This study provides insight into the seismic performance of hybrid timber-steel structures designed with high R-value seismic force-resisting systems. Conceptual designs have been developed for the adaptation of existing advanced energy dissipative steel systems into predominantly heavy timber structures. These designs are made possible through the application of hybrid steel-wood connections utilizing advanced fastener technology at the forefront of heavy timber design.

Two six-storey buildings were designed in Victoria, BC.; one using the newly developed hybrid systems, and one using conventional steel construction. Two-dimensional nonlinear dynamic time-history analysis was conducted on four lateral load-resisting frames; two frames used buckling-restrained bracing (BRB) systems (hybrid and steel-only), and two frames used moment-resisting frame (MRF) connections with reduced beam sections (RBS). The hybrid MRF system used nonlinear replaceable link elements in combination with typical RBS details. The seismic performance of the four 2D frame systems was evaluated using OpenSees by subjecting the frames to 44 MCE level grand motions and 44 DBE level ground motions.

Each frame was evaluated based on overall lateral load imparted on the system, maximum interstorey and residual drift and storey acceleration. The results show that the reduced weight of the hybrid structure significantly lowers imparted seismic design forces, resulting in smaller structural elements and offsetting the increased cost of timber construction. In the BRB frames, the maximum and residual drifts of the hybrid and steel-only frames were similar. The hybrid MRF experienced slightly higher drifts than the steel-only MRF due to the increased flexibility of timber and the MRF’s dependency on flexural stiffness to resist lateral deformations. Both the hybrid and steel-only MRFs had higher peak drifts and residual drifts than the BRBFs due to lower lateral stiffness; however, in all cases the mean values of the peak interstorey drifts stayed within 2.5% of storey height. In most cases, storey accelerations were found to be lower than the corresponding average peak ground acceleration, and kept below 0.5 g for all frames.

This research shows that the implementation of ductile steel systems into heavy timber frames, creating a hybrid steel-timber structure, could potentially overcome the disadvantages of timber-only seismic force-resisting systems. Designing these systems using high R-values equivalent to steel-only structures results in similar seismic performance and reduced seismic weight compared to such steel-only structures.

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