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WOOD-STEEL HYBRID SEISMIC FORCE RESISTING SYSTEMS: SEISMIC DUCTILITY

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ABSTRACT: North American building codes currently provide strict limits on height of wood structures, where for example, in Canada wood structures are limited to 4 or 5 storeys. This paper examines wood-steel hybrid system to increase seismic force resistance beyond current limits, up to 10 storeys. The use wood-steel hybrid systems allows for the combination of high strength and ductility of steel with high stiffness and light weight of timber. This paper examines one type o wood and steel hybrid system: a steel moment frame with infill crossed Laminated Timber (CLT) shear walls. A detailed non-linear model of a 2D wood-steel hybrid seismic force resisting system was completed for 6, and 9 storeys; with two different steel frame designs, and four different placements of the infill walls. The static pushover response of this type of hybrid seismic force resisting system (SFRS) has been completed and compared for all cases. The results indicate that preliminary values for ductility (R_d) and overstrength (Ro) for this type of system are 2.0 and 1.7, respectively, similar to a plain wood wall system. Low ductility frames benefit the most from the addition of CLT shear walls as they do not lose the ductility in the system.

KEYWORDS:

1 INTRODUCTION

Hybrid systems are commonly used throughout the world, and are present in many types of structures with many different types of material. A hybrid system is any system that combines two or more structural materials. Steel and concrete hybridization is most common; these include concrete on metal deck supported on steel beams as a floor system, also steel frame buildings commonly use concrete elevator shafts and/or stair wells for lateral resistance. Steel and timber hybrid systems are less common but do exist; for example, Quebec and Northern Ontario have many steel and wood hybrid bridges (Krisciunas, 1996).

Effective timber and steel hybridization creates a system where steel is used minimally only where high strength and ductility are needed. Steel is much stronger and provides significant post-yield deflection capability, known as ductility; steel moment frames are extremely ductile, with large deflections during seismic events. Wood is comparatively much weaker usually requiring larger members, resulting in stiffer systems; wood does not produce post-yield deflection, especially when loaded perpendicular to the grain. Several issues are immediately obvious with this type of seismic force resisting system, the largest being the incompatibility associated with the difference in material properties; the incompatibility of steel and timber mean the connections are an important problem. Despite this, the light, cheap, and environmentally friendly nature of wood makes it a good material to pair with stronger, more ductile steel. Many options exist for hybridization of steel and timber within a vertical seismic resistance system. To effectively create a hybrid system it is important to understand the properties of both steel and wood. The characteristics of wood and steel are summarized in Table 1.

Table 1: Material Properties for Steel and Timber

Material		Steel	Timber
Density (kg/m^3)		7800	400-600
Mod. Elasticity (MPa)		200 000	8000-11000
Strength (MPa)	Comp	400 - 1000	Parallel: 30 Perp: 8
	Tens	400 - 1000	Parallel: 6 Perp: 1
	Yield	350	N/A

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It is important to note that timber is a material with less reliable strength characteristics than steel and concrete because it is a natural material. Timber characteristics are extremely dependent on the species of tree and specific qualities of the wood harvested; growing conditions can have a large impact as well as local imperfections in the wood, such as knots (Keenan, 1986).

Current design philosophy, as detailed in the National Building Code of Canada 2005 (NRC, 2005), is force based design. In other words, the force exerted on the building given a fully elastic response for a given seismic hazard index is calculated; the elastic force is then reduced by the ductility factor, R_d, and the overstrength factor, R_o, to design force allowing for plastic behaviour. The NBCC 2005 provide R_d and R_o factors for many seismic-force-resisting system (SFRS) types but currently provides no information for steel and wood hybrid systems.

First we will review the types of hybridization: component and system hybridization, and how they can be applied to steel and wood. Some case studies of existing hybrid systems are also reviewed. Finally, initial analyses were completed on one type of seismic force resisting system, a steel moment frame with structural infill wood shear walls. The proposed system is described, along with each of its components; preliminary results for static NBCC loading and static pushover analysis confirm the contribution to strength and stiffness, compared with either material working alone. This system performs similarly to the common hybrid seismic force resisting system: concrete moment frames with masonry infill shear walls.

2 Hybrid System Case Studies

Although the combination of steel and wood in buildings is not as common as hybrid steel and concrete, there are still some existing case studies to inform the our direction. Two such case studies are reviewed here.

2.1 Kanazawa M Case Study

One case study is the existing Kanazawa M building in Japan (Koshihara *et al*., 2005). It combines several kinds of hybridization both at the component and system level. The first storey is a reinforced concrete shear wall structure, and the second to fifth storeys are a steel and timber hybrid frame system. This represents a vertical mixed system, while also using component hybridization with the flitch type members in the seismic force resisting system in the upper levels.

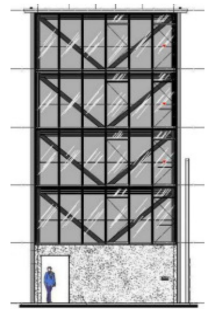

Figure 1: Building elevation

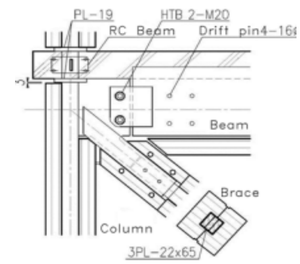

Figure 2: Hybrid braced frame connection detail

The frame members are made of hybrid steel and wood components. All the columns, beams, and braces are constructed from engineered timber hybridized with steel plates or bars.

2.2 Scotia Place Case Study

Another case study is the Scotia Place building in New Zealand constructed from steel concentrically braced frames and wood floors on steel beams (Moore, 2000). The wood floor acts as the slab for gravity, as well as the diaphragm for seismic and wind loading. The vertical structural system in this building is purely a steel frame system.

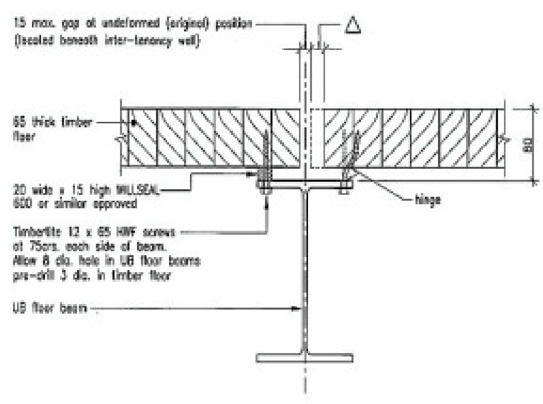

Figure 3: Hybrid floor assembly

The use of wood floors significantly reduced the mass in the building; the effect was significant on the design for both gravity and seismic load resistance. The steel columns, concrete basement columns, and piles on which the building is supported were all made smaller as a result of the use of wood floors instead of concrete. However, the mass reduction resulted in wind governing the vertical lateral design; only the wood diaphragm was governed by the seismic loading.

3 HYBRIDIZATION TYPES

A hybrid building or structural system uses two or more materials in combination, ideally to work with the best of each material's attributes. Generally, two types of hybridization are used individually or in some combinations: component level and system level hybridization (Stiemer *et al*., 2012).

3.1 COMPONENT LEVEL HYRBIDZATION

Component level hybridization is when more than one material used in a single member. In timber systems, there are several types of component level hybridization. Flitch beams are one type of steel-timber component hybridization. A flitch beam consists of one or more steel plates or other steel members are sandwiched between pieces of timber. There are several advantages to this type of system: the steel beam has significantly higher strength than the timber members, but is susceptible to lateral torsional buckling; the wood provides lateral restraint. The steel and wood is connected using bolts to transfer shear along the length; this is important to ensure distribution of the load without causing splitting in the wood.

3.2 SYSTEM LEVEL HYRBIDZATION

This type of hybridization involves members that are timber and members that are steel. The connections between these members are frequently the most complex issue.

One of the simplest forms of system hybridization is a simple vertical mixed system; the bottom stories are constructed out of strong and stiff materials, like steel or concrete, with the upper storied constructed from lighter, weaker material, such as wood (Khorasani, 2010). The mix of vertical systems has also been used to extend the storey limits imposed by building codes like the NBCC 2005. It is important to address the vertical stiffness change in this type of hybrid system resulting in two building periods, and the possibility of a weak storey; shake table test on concrete-wood specimen suggests that seismic responses are greater for specimens with smaller stiffness ratio than that with larger stiffness ratio (Xiong and Jia, 2008).

4 MODEL OF HYBRID INFILL WALL

The most common types of hybrid infill wall system are masonry infill walls in low ductility steel or concrete moment frames. They effectively stiffen and strengthen the moment frame considerably. Many studies show the benefits: increased strength, stiffness, energy dissipation, and resistance to incremental collapse; conversely the ductility of the system is significantly reduced by their inclusion in the structure (Kodor *et al*., 1995).

Frequently, non-structural partition walls are added as infill to moment frames; these walls can have serious impact on the seismic response of the structure due to the high comparative stiffness of the infill partitions to the moment frame despite being non-structural components. Yousuf and Bagchi's (2009) study on the impact of non-structural infill partition walls on ductility type ductile (D) steel moment frames found that the impact of infill partition walls results in decreased deflection and ductility in the structure, and in some cases hinging in the columns.

This study will look at steel moment frames designed for a variety of ductility levels and compare the behaviour to similar frames with the addition of wood infill shear walls. Hybrid wood and steel infill wall systems would not provide the stiffness of a masonry infill wall but has the advantage of significantly less added weight to the system, and using crossed laminated timber (CLT) shear walls we can obtain a stiffness much great than that of typical OSB or plywood shear walls.

A simplified floor plan and elevation are shown in Figure 1. The plan has 4 x 3 bays. The 2D frame elevation taken in the horizontal plan direction will be used as the base steel moment frame for analysis. The dead loads are based on concrete on metal deck for the floors for a total dead load of 4.05 kPa, and 3.4 kPa dead load for the roof.

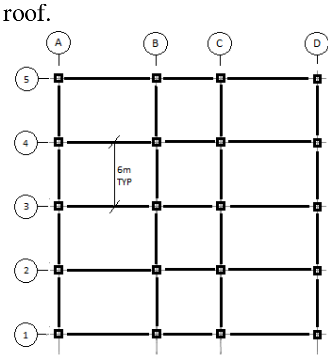

Figure 4: Test building floor plan

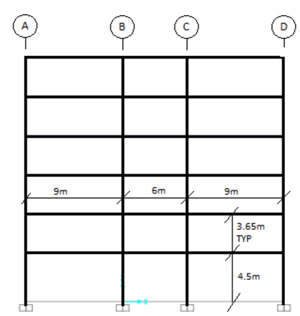

Figure 5: Test building elevation for 2D analysis at 6 storeys

The steel frames at 6 and 9 storeys are designed with different sets of member sizes, one to provide a base moment frame ductility type ductile (D), and limited ductility (LD) as specified in the CSA S16 code (CSA-S16, 2009). The placement of the infill walls shown in Figure 2 will help determine the contribution of infill walls on the overall response of the structure. The sensitivity of the system to the area of shear wall is an important factor in the implementation of the system.

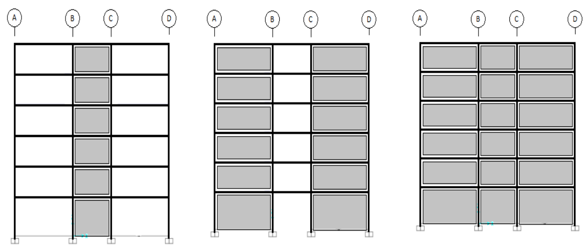

Figure 6: Infill wall locations within moment frame

The system will be modeled in SAP2000 with the frame modeled as elastic frame elements with hinges matching ASCE-41 modeled at the end of each member.

4.1 CROSSED LAMINATED TIMER WALLS

Crossed laminated timber is layered and glued in alternating directions to approximate an orthotropic plate (Mohammad, 2010). A large scale shake table test of mid-rise building constructed from CLT slabs and walls was performed in Japan. High intensity shaking, with a peak ground acceleration of 0.82g, was applied and the building responded with minimal structural damage (Dujic *et al*., 2010). CLT walls combine two behaviour types: overturning or rocking, and panel shear (Schneider, 2009); deformation in the panel provides minimal contribution to the hysteretic behaviour. The CLT without the contribution of the connections is generally treated as either a linear elastic material, or even a rigid body. Based on testing done at FPInnovations, R_d of 2.0 and R_o of 1.5 are conservative estimates for CLT structures with nailed and screw connections (Popovski and Karacabeyli, 2011). Further, the behaviour is superior to that of braced timber frames given similar seismic factors, CLT constructions “is not susceptible to the soft storey mechanism as the panels (that are also vertical load carrying elements) are virtually left intact in place even after a “near collapse” state is reached” (Popovski and Karacabeyli, 2011). This is also true in a hybrid infill shear wall system.

The actual panels were modeled as orthotropic elastic thick shell elements; representing a 3ply, 94mm, panels with a central laminate of 34mm. Values for the properties of CLT panels are dependent on the type of wood used and the number and orientation of the layers. Using the values developed by KLH, the material properties in Table 2. Modelling CLT panels as elastic element is more complex than most analytical studies of CLT panels, where the panels are modeled as rigid bodies.

Table 2: CLT Elastic Material Properties Parallel to the Grain (KLH, 2008)

Elastic Modulus (E)	12000 MPa
Shear Modulus (G)	250 MPa
Tensile Strength	16.5 MPa
Compressive Strength	24 MPa
Crushing Strength	30 MPa
Shear Strength	5.2 MPa

Note that the grain direction of the cross laminated panel is to be taken as the grain direction of the face layer of the panel. The strength of the walls will then be compared with the stress values in measured in the system.

4.2 CONNECTIONS BETWEEN FRAME AND WALLS

The connection between the frame and the shear wall has two important features: the brackets connecting the shear walls to the frame, and the confinement of the wall from the frame (e.g. see Figure 7). Testing was completed at FPInnovations on nailed steel brackets in CLT walls; the resulting hysteretic envelope is shown in Figures 8 and 9.

Figure 7: CLT shear wall bracket (Schneider, 2009)

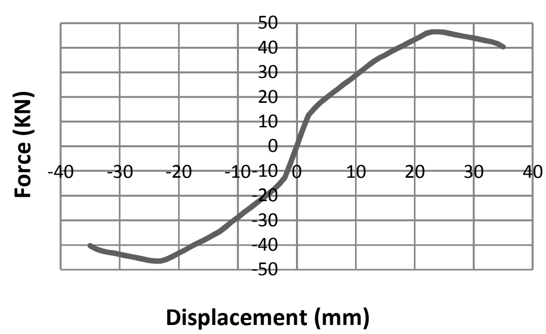

Figure 8: bracket experimental force-displacement results perpendicular to CLT wall edge (Schneider, 2009)

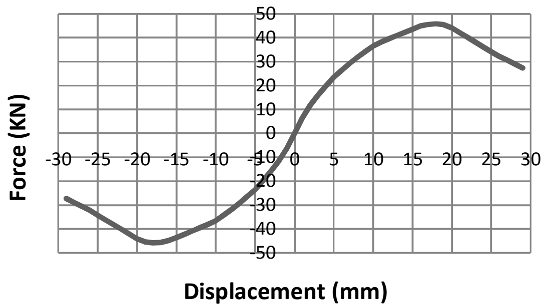

Figure 9: bracket experimental force-displacement results parallel to CLT wall edge (Schneider, 2009)

The connections were modeled as non-linear plastic links along all sides of the wall at 800c/c spacing. Additionally, the confinement of the wall provided by the steel frame has been modeled using gap link elements with an initial gap of 12mm. This will allow deformation in the brackets before the wall comes in contact with the frame. This will maximize the energy dissipation in the brackets..

5 STATIC ANALYSIS RESULTS

Static modal pushover analysis is a commonly used analysis method for initial strength and ductility quantification. FEMA P695 uses static pushover response curves to help determine the ductility and overstrength of a system. They break down overstrength and ductility into two factors, similar to the NBCC. The first factor Overstrength (Ω, R_o) is defined as the ratio of the maximum base shear resistance (V_{max}) and the design base shear (V) (FEMA, 2009):

$$\Omega = R_o = \frac{V_{max}}{V} \tag{1}$$

Because we are interested in the innate overstrength in the system, we will assume perfect design and consider the design base shear as the base shear at first yield.
Ductility (μ, R_d) is defined as the ratio between the ultimate roof drift (δ_u) and the yield roof drift (δ_y):

$$\mu = R_d = \frac{\delta_u}{\delta_y} \tag{2}$$

The yield drift of the system has been determined analytically.
The ultimate roof drift is determined as the point where the system has had a 20% strength loss, which could be considered failure. Failure is the "near collapse" states, or the state where the system is no longer stable. The infill wall systems are unlikely to experience a true collapse while the walls remain in the bays. It is important to note that 3% deflection is generally considered failure for a steel moment frame; we will take the failure of the frames occurring at 3% drift.
A pushover analysis is performed for each ductility and infill configuration. The pushover analyses compare the plain frame to increasing bays of infill shear walls. The curves in Figure 10 and Figure 11, respectively, show increased stiffness for each additional shear wall added for the six and nine storeys buildings. The systems with infill CLT walls in two or three bays do not exceed the deflection limits. The points of first yield in the steel frame are also shown on the curves.

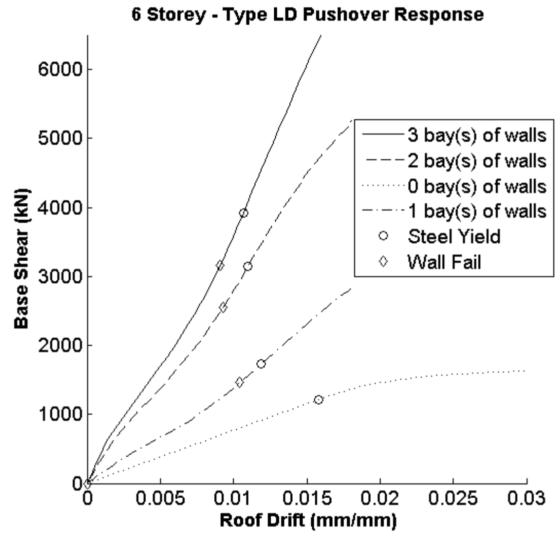

Figure 10: Static modal pushover curves for 6 storey systems with infill Walls

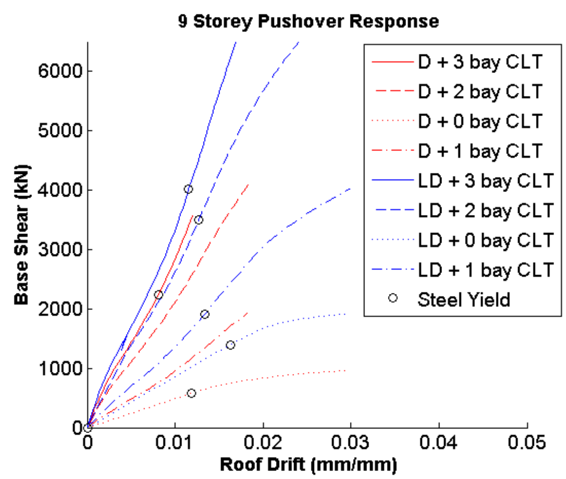

Figure 11: Static modal pushover curves for 9 storey systems with infill walls

The ductility and overstrength factor, as shown in Equations 1 and 2 are also computed, and summarized in Table 3. The type D frame at 6 stories shows a reduction of 50%, 58%, and 74% in ductility compared to the design Rd value of 5.0 for 1, 2, and 3 bays of infill CLT shear walls respectively; at 9 storeys, the reductions in ductility are 58%, 56%, ad 70% compared to the same design value and the infill placement. Comparatively, for both 6 and 9 storeys, the analytical ductility values for type LD frames never show a greater reduction in Rd than 25% and in some cases are actually above it. Further, the range of ductility values for all frame types and all building heights is quite narrow, from 1.5 to 2.2;

the design values for plain CLT SFRS appear to be appropriate for the hybrid system as well.
The analytical overstrength value is consistently higher than the design value ranging from 50% to a full 100% higher than the 1.5 design Ro for type D frames. Comparatively the overstrength values for type LD frames are much closer with values ranging from 5% to 25% higher than design overstrength (Ro) values.

Table 3: Ductility Results for steel frames with infill CLT walls

			Plain Frame	1 Bay	2 Bays	3 Bays
6 Storey Frame						
D	Model	μ_T	6.9	2.5	2.1	1.3
		Ω	2.5	3.1	2.8	1.5
	NBCC	R_d	5.0	2.0	2.0	5.0
		R_o	1.5	1.5	1.5	1.5
LD	Model	μ_T	2.9	1.5	1.6	1.7
		Ω	1.5	1.6	1.7	1.9
	NBCC	R_d	2.0	2.0	2.0	2.0
		R_o	1.7	1.5	1.5	1.5
9 Storey Frame						
D	Model	μ_T	4.8	2.1	2.2	1.5
		Ω	1.5	2.4	2.3	1.6
	NBCC	R_d	5.0	2.0	2.0	5.0
		R_o	1.5	1.5	1.5	1.5
LD	Model	μ_T	2.2	2.2	1.9	1.6
		Ω	1.4	2.1	1.9	1.8
	NBCC	R_d	2.0	2.0	2.0	2.0
		R_o	1.7	1.5	1.5	1.5

The addition of CLT infill walls has minimal effect on the ductility of a LD moment frame. Additionally we can note that the ductility of the steel frame alone appears to have little impact on the ductility of the system after the infill walls have been added. The hinging pattern also seems to be most beneficial for the system with a narrow steel bay between two hybrid shear walls as shown in Figure 12; this system is similar to a coupled wall system in concrete shear walls. Further study would help develop this further. Further optimization of this beam could allow for further energy dissipation through hinging of these beams.

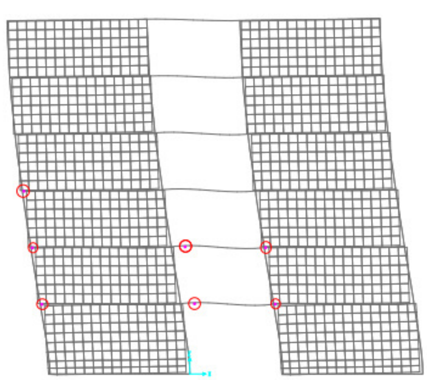

Figure 12: Coupled wall hinging system

It is also important to note that the connectors play very little role in the capacity of the system at the ultimate limit states seismic design (rate of return of 2% in 50 years). The connectors yield significantly before the rest of the system. The form of the pushover curves also shows an initial zone of elasticity which quickly yields followed by a second zone of elastic behaviour before the frame begins to yield. A closer view is provided for the systems in Figures 13 to 16. These show the two zones of elasticity clearly.

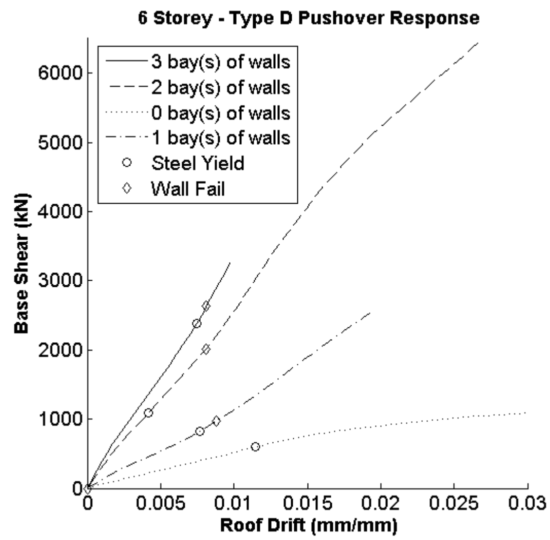

Figure 13: Pushover curves for 6 storey hybrid systems with type D moment frames

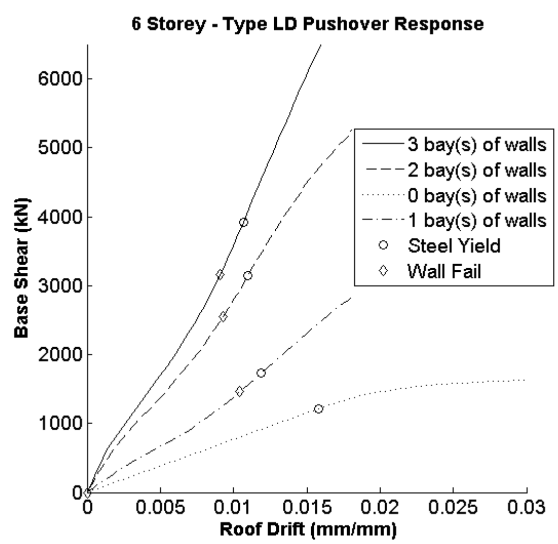

Figure 14: Pushover curves for 6 storey hybrid systems with type LD moment frames

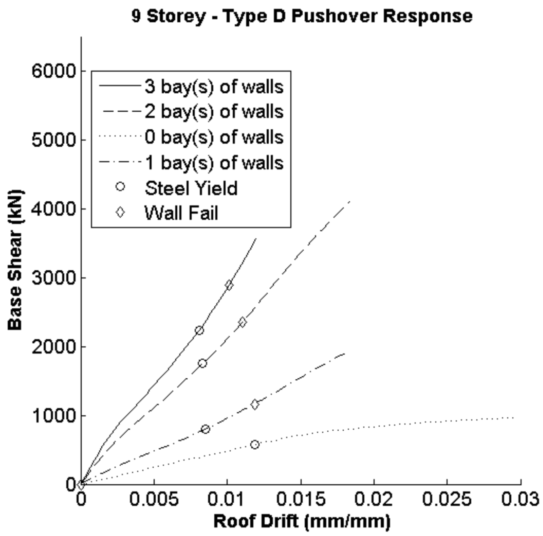

Figure 15: Pushover curves for 9 storey hybrid systems with type D moment frames

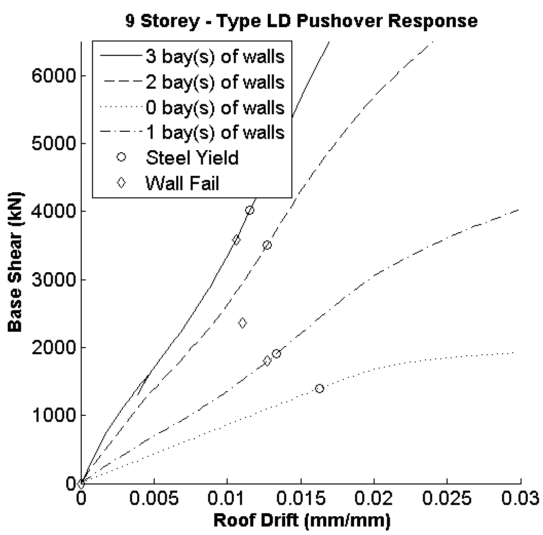

Figure 16: Pushover curves for 9 Storey hybrid Systems with type LD moment frames

This would allow properly designed connectors to work for multiple performance levels, such as 50% in 50 years or 10% in 50 year earthquake occurrence. This system is well suited to performance based design; in a low intensity earthquake, the damaged wood connectors could be easily replaced, with the steel frame remaining undamaged; the low levels of deflection resulting from the walls allows the otherwise ductile system to minimize damage on non-structural components that might have been damaged in a simple steel moment frame.

It is also important to note that the strength of the wood is being reached near the point of hinge creation in the steel. The high stresses are present only at the corners of the panels as shown in Figures 17 and 18. The areas shown in green in the corners of the panels shown in Figure 18 are the only location that experience stresses above the strength limit of the CLT panels.

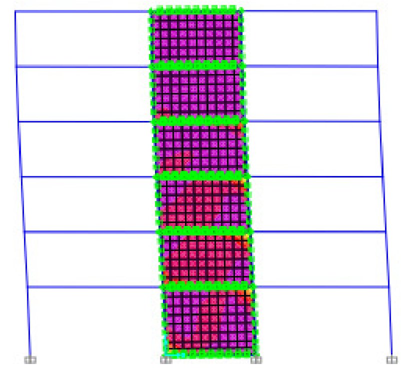

Figure 17: Deflected shape CLT panel stress distribution

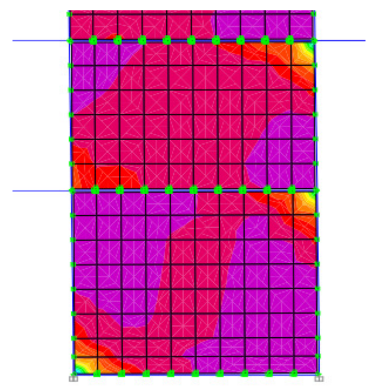

Figure 18: Stress distribution in CLT panel

The NBCC also provides guidelines on period limits, but they do not provide period limits of hybrid infill shear wall systems; Table 4 compares the period limits from NBCC 2005 to the model values. The period is least affected with the addition of wood shear walls in the LD frame; the period is reduced by 20% for the LD moment frame where as the D and MD frames are reduced by 25% and 26%, respectively. Furthermore, as more walls are added to the system, it is clear that the frame type has minimal contribution to the period of the system.

Table 4: Building periods and modal coefficients

System		Analytical LD	Analytical D	NBCC
6 storey	Plain frame	1.20	1.46	0.89
	1 Bay	0.96	1.08	0.52
	2 Bays	0.66	0.75	
	3 Bays	0.61	0.67	
9 storey	Plain frame	1.62	2.23	1.19
	1 Bay	1.35	1.57	0.70
	2 Bays	0.96	1.09	
	3 Bays	0.89	0.98	

6 CONCLUSION

Steel and Timber hybrid buildings show significant promise as a reliable seismic force resisting system. Around the world, steel and wood hybridization is being used more and more frequently. Despite this, Canada is just beginning research on this type of system. Other parts of the world including Japan and New Zealand show many different types of wood hybridized systems, some examples are given with wood diaphragm and wood hybrid braced frames. The study within this paper focuses on the preliminary evaluation of one type of vertical seismic force resisting system, a steel frame with CLT infill shear walls.

The initial results from the static analytical study completed herein show promise for the steel frame with CLT infill wall type system. The addition of wood walls significant increased the strength and stiffness of the system. The increase in stiffness is particularly beneficial for steel moment frames as they are commonly governed by deflection. The addition of walls did result in a decrease in ductility from the plain frame system. The reduction in ductility was the least severe when applied to the low ductility frames. Further, no benefit was found using a ductile moment frame over a low ductility moment frame after 1 or more bays of walls were added.

The hinging pattern appears advantageous for a system with bays of hybrid infill walls surround narrow open moment frame bays. Further parametric study can help determine how to optimize the member sizes to get the most ductility in to the system from the central beam.

Additionally, the addition of connectors can provide the system with two different and separate performance levels. Smaller earthquakes damage only the connectors which can then be easily replaced, while large earthquakes damage the structure significantly, but maintain life safety.

Further research is required to effectively determine appropriate seismic force reduction factors for this type of system including a full incremental dynamic analysis. Finally, to effectively implement any hybrid steel and wood systems, significant additional research is required for the connections, likely including experimental testing.

7 ACKNOWLEDGEMENT

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