Development of Steel-Wood Hybrid Systems for Buildings under Dynamic Loads

Siegfried Stiemer
University of British Columbia, Vancouver, B.C., Canada

Solomon Tesfamariam
University of British Columbia, Kelowna, B.C., Canada

Erol Karacabeyli & Marjan Popovski
FPInnovations, Vancouver, B.C., Canada

ABSTRACT: A steel-wood hybrid system furnishes not only aesthetically pleasing and sustainable hybrid structures but is superior in seismic applications due to the light weight, high resistance, and adjustable ductility. Such hybrid structural systems are not covered by any material and structural design standards that hinder the general implementation. For light structures, a builder's guide to hybrid wood and steel connection details already exists in North America. Despite the obvious advantages, however, today's applications of steel-wood hybrid structures have been limited. Rare hybrid buildings with a concentrically braced frame used for lateral load resistance with a glulam timber floor slab have been built as prototypes. The use of glulam floor slab led to a substantially reduced self-weight, compared with the reinforced concrete slab option. The lighter structure behaves superior in seismic events and has made wind loads the governing design case. The next generation steel-wood hybrid structures should optimally utilize each material. This paper describes a research program of the next generation wood-steel hybrid structures should optimally utilize each material. In detail the following development issues will be addressed: innovative hybrid steel-wood building systems, technical tools to predict structural responses of hybrid systems, design principles underpinning the definition of key code provisions related to strength and serviceability performance of hybrid buildings. It will be highlighted that potential structural problems at the design stage result from material incompatibilities. The constitutive properties of each material, hybrid-material, and joint properties reported in the literature will be used, or supplemented by findings from experimental work.

1 INTRODUCTION

Hybrid structures are commonly used around the world. Steel and concrete hybrid members and systems are the most common. These include concrete on metal deck supported on steel beams as a floor system. Steel frame buildings commonly use concrete elevator shafts and/or stair wells for lateral resistance.

Timber has started to be used more in hybrid structures throughout the world. Quebec contains many steel-timber hybrid bridges for example (Khorasani, 2010). Regardless, there are no codes or standards for the design of timber-steel hybrid construction.

2 MATERIALS AND THEIR PROPERTIES

In order to create methods of combining different materials to work with their respective advantages and compensate for their respective disadvantages, a full understanding of the material properties is required.

2.1 *Steel*

Steel has the highest strength and the highest ductility of all the typical building materials. Its yield strength if generally taken as 350 MPa, but in tension and compression (where buckling is ignored) it can have an ultimate capacity ranging from 400 to 1000 MPa. Steel is also an isotropic material, meaning that its characteristics are constant throughout the material. It also has the largest unit mass of any of the typical construction materials. Because of its extremely high strength, the challenge with steel is to use the least amount of material possible, resulting in buckling governing design of columns and compression zones in beams. As a result, steel is the ideally used in tension.

2.2 *Timber*

Unlike concrete and steel, wood is an anisotropic material. This means that the material has different strengths depending on the orientation and location where the load is applied. There are multiple things that affect the strength of wood. Generally the species and specific characteristics of the tree the timber was harvested from changes the strength of the material. Additionally, wood has different strengths in its three axes. Wood is strongest parallel to the grain, and less strong when perpendicular. Further, it has different characteristics when load is applied perpendicular and parallel to the ring growth of the tree. These can be referred to as the longitudinal, radial, and tangential directions respectively (Figure 1)

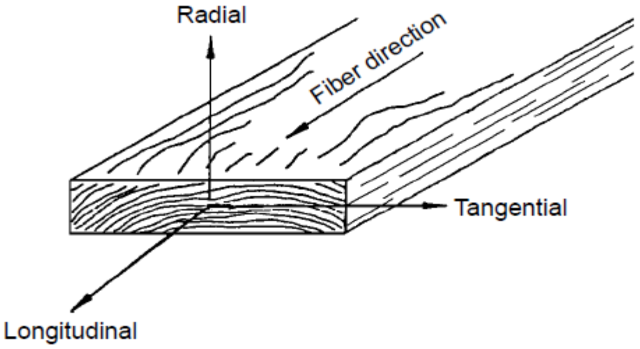


Figure 1: Orientation and directions of timber grain (Khorasani, 2010)

Wood is also much stronger in compression that in tension, although with a much higher ratio of tension/compression than available in concrete. Timber is also the most flexible of the traditional building materials. Although it does not have a defined yield point, like steel, it has a much less brittle failure mode than concrete depending on the orientation of loading. Timber also has a high strength-mass ratio in the direction of compression parallel to the grain.

Aside for directional properties, wood is also susceptible to rolling shear. Rolling shear is the shear that leads to strain perpendicular to the grain direction. If wood is thought of a series a parallel fibers held together with fairly week bonds between them, then rolling shear results in the fracture of this type of bond and the fibers “rolling” next to one another.

It tends to cause detachment of one layer of wood grain from another. Wood is much weaker in this type of shear than in “radial” or “longitudinal” shear.

2.3 *Engineered Wood*

Over the years, engineers have worked to create wood members with more reliable material qualities and more orthotropic properties than sawn lumber. They are called composites and are defined as “material that have the commonality of being glued or bonded together” (Maloney, T., 1996)

The most commonly used type of engineered wood product in structural engineering is Oriented Strand Board (OSB). This material is made by pressing together wood strands to make a mat. The strands are bonded together with resin such as phenol-formaldehyde. The mats are then layered together with the strands running in opposite directions to create a panel. The alternate layering is done so as to provide strength in both axes.

Laminated veneer lumber (LVL) is another type of engineered timber that is gaining popularity in the residential and commercial construction practices. This material is made by creating thin veneers. The veneers are layered and bonded together with a unidirectional grain orientation.

Glued laminated timber, often called Glue-lam is another type of engineered timber that is used frequently in large scales timber beams and columns

Conversely, Cross laminated timber (CLT or Cross-lam) is frequently used for floors and walls and has been used in midrise buildings in various parts of the world. CLT panels are made in a similar fashion to glue-lam members.

3 HYBRIDIZATION

A hybrid is a building or structure that uses both or more materials in combination to get the most out of each material.

This paper will focus most closely on steel and timber. Technically all timber buildings are hybrid systems as the connections are almost always made of steel as nails.

Two types of hybridization are having the attention within this paper. The first is component level hybridization and the second is system level hybridization

3.1 *Component Level Hybridization*

Component level hybridization involved more than one material type within a member.

Flitch beam (Figure 2) is an example for a timber-steel hybrid component. This is a type of beam that generally has a steel plate 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 a steel plate will generally have significant issues with lateral torsional buckling and fire resistance. The timber members provide lateral restraint for the steel preventing lateral torsional buckling or local buckling as well as enough fire protection. Bolts are used to transfer shear between the metal plate and the wooden members (Khorasani, 2010).

A derivative of a flitch beam can be achieved by using an H-shape or steel bar as a core and surrounding timber members (Figure 2). The loading carried by the hybrid member is shared depending on the ration of flexural rigidity of timber and steel.

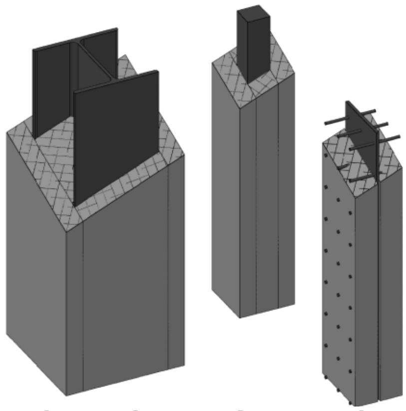


Figure 2: H-shape and rectangular core with timber surrounding and flitch beam with shear bolts

3.2 *System Level Hybridization*

This type of hybridization involves members that are timber and members that are steel. One example of this would be a wood and steel truss. One could use a timber top chord for the truss and steel bottom chord. This would work to each material's advantages as timber has high compression strength and steel is best used in tension.

A vertical mixed system is another way for hybridization. The lower levels are made in steel or concrete and the upper levels are built entirely in wood. In levels with the maximum vertical load concrete or steel is contributing to its high elasticity modulus while wood in the upper levels is reducing the total weight of the building (Khorasani, 2010).

Steel frame, timber floor joist and plywood diaphragm is a different way of hybridization. Hybeams with plywood flooring act as diaphragm and transfer the lateral and gravity load to the steel moment frames (Figure 3). Hybeam is an optimized wooden I-beam with solid timber flanges and a plywood web. The application of hybeams and plywood flooring reduces the over all weight enormously.

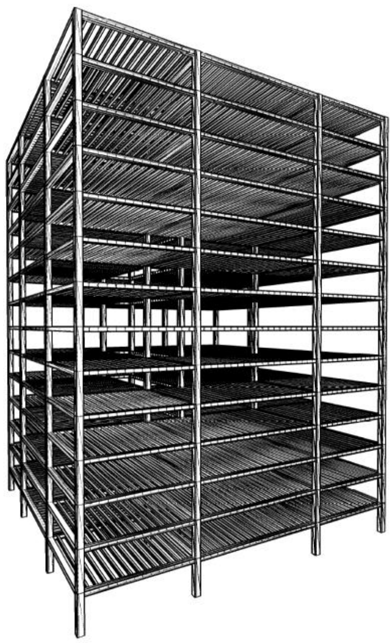


Figure 3: Fourteen story hybrid timber steel building with hybeam joists and plywood flooring

The combination of steel columns and timber beams are called hybrid frame. Compared to an all timber frame, steel columns increase the load bearing capacity while keeping the slenderness of the columns.

4 STRUCTURAL PROBLEMS AND MATERIAL INCOMPADIBILITIES

It is important to note that timber is a material with less reliable strength characteristics than steel and concrete because it is a natural grown material. The strength, stiffness, and density of wood vary greatly between species and even within a species depending on the growing conditions and local imperfections (Table 1). The difference in the modulus of elasticity is shown in Figure 4 (Keenan, 1986).

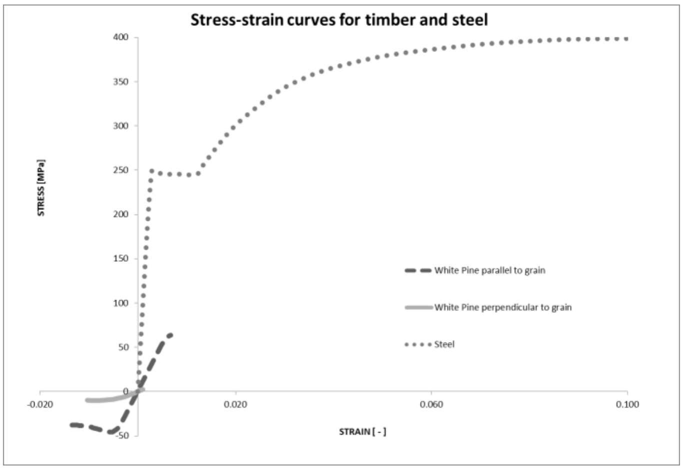


Figure 4: Stress-strain comparison of steel A36 and white pine

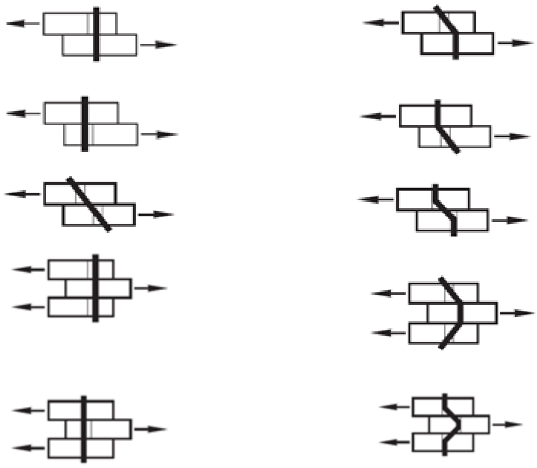
Wood is a hygroscopic material. It gains and loses moisture which causes swelling and shrinkage. Steel has a high thermal expansion coefficient. These behaviors have to be considered in a hybrid building design.

Table 1. Approximate Material Properties for Steel, Wood, and Concrete (Kohrasani, 2010)

Material	Steel	Concrete	Timber
Yield Strength [MPa]	350	N/A	N/A
Density [kg/m³]	7800	2300	400-600
Mod. Elasticity [MPa]	200000	20000	8000-11000
Comp. Strength [MPa]	400-1000	20-40	parallel 30 perp. 8
Tensile Strength [MPa]	400-1000	2.0 – 5.0	parallel 6 perp. 1

Special attention should be paid to the different ways of connecting the two materials. Steel members can be welded or bolted together. Structural timber members are connected in general with only pinned connectors like bolts, screws or nails or additional steel plates. To connect steel and timber in one supporting structure the connection must be easy, durable and accessible since different companies are responsible for timber and steel members. To avoid big internal stresses through the unique properties of each material expansion joints or flexible connections should be provided.

Careful design of these connections is important; Wood splitting should be avoided as it will result in brittle failure. Wood splitting at the connections can be avoided with proper connection design. Steel connectors should be designed to yield prior to failure of the wood. The plastic deformation of the connectors will then result in local crushing of the wood. A comparison of yielding forms for dowel type connectors (nails, bolts) is shown in Figure 5.
An ideal connection involves the largest amount of yielding possible, with the minimum amount of wood crushing (CSA-O86, 2001)



a) wood failure b) bolt yielding, then wood crushing

Figure 5: Wood Connection yielding (CSA-O86, 2001)

5 STEEL FRAME WITH WOOD INFILL SHEAR WALLS

Based on the study done by Yousuf (2009) we will look at the contribution from introduced wood shear walls in a steel frame. He compared a bare steel frame to that will infill walls. The infill walls incorporated in his study were meant to account for the effects of nonstructural elements within the building.

5.1 *Midply Shear Walls*

Wood shear walls are typically made from sheathing and studs nailed together. The sheathing is generally plywood or OSB and the studs are generally sawn lumber. An alternative called midply has been developed using the same materials as a standard shear wall (Varoglu, Ni, Popovski, Karacabeyli, & Stiemer, 2006). Midply shear walls will be used for the infill walls in place of typical wood shear walls.
There are several advantages to using midply shear walls instead of standard shear walls. Midply shear wall use nails in double shear instead of single shear as shown in Figure 6. Also, the nail heads are not flush with the sheathing and can thus avoid pulling through the sheathing. Finally, the midply shear wall configuration also increases the parallel end distance for the nails.

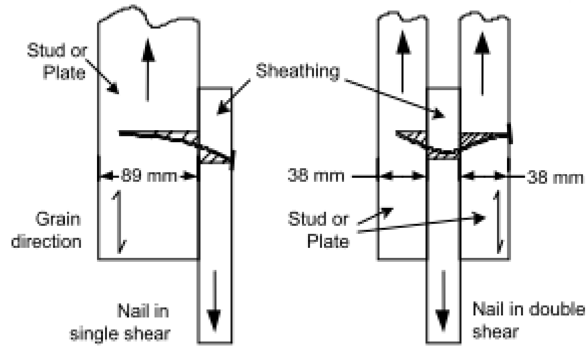


Figure 6: Typical Shear wall nail deflection (left); midply shear wall nail deflection (right) (Varoglu, Ni, Popovski, Karacabeyli, & Stiemer, 2006)

Due to the different nailing configuration, midply shear walls show more than two times the strength and stiffness as well as increased energy dissipation when compared with a standard wood shear wall

Table 2: Shear wall average test results (Varoglu, Ni, Popovski, Karacabeyli, & Stiemer, 2006)

Wall type	Stud Member	Max Shear	Ult. def'l	Shear wall Stiffness
Standard	38x89	8.9 kN/m	88.9mm	0.6kN/m/mm
Midply	38x89	30.0 kN/m	103mm	14kN/m/mm
Midply/Standard Ratio		3.4	1.2	2.3

When failure did occur it involved a combination of yielding of the nails and local crushing of the wood, as well as eventual buckling of the studs. Ni suggested that a ductility factor (Rd) of 3.0 was appropriate for midply wood shear walls, similar to typical shear walls (Varoglu, Karacabeyli, Stiemer, Ni, Buitlaar, & Lungu, 2007)

5.1.1 *Original Model*

The building to be modeled (with SAP2000) was based on the building modeled in Yousuf and Bagchi's study (Yousuf, 2009). This model involved steel moment frames with type D ductility steel moment frames (CSA-S16, 2009). The members used were all CSA G40.21 grade 350W sections as follows:

Table 3: Steel frame members

Columns	Beam Top Storey	Beam Other Storeys
W310x253	W310x79	W310x86

The ground floor storey height was taken as 4.5m and all other storeys taken as 3.65m. The dead loads in Yousuf's study were assumed to be 3.4Kpa for the roof and 4.05KPa for the floor with a tributary width of 6m.

5.1.2 *Model with Infill Wood Shear Walls*

The bare steel frame was then modeled with infill shear walls in the central bay of the frame.

Colin Clarke (2009) modeled a midply shear wall in SAP2000. Testing found that the model matched experimental results well. The sheathing was modeled after plywood as an isotropic shell element in SAP2000. The studs were simplified to be orthotropic. The two are connected together with non-linear elastic links modeled after the behavior of nails as per the behavior found by FP innovations.

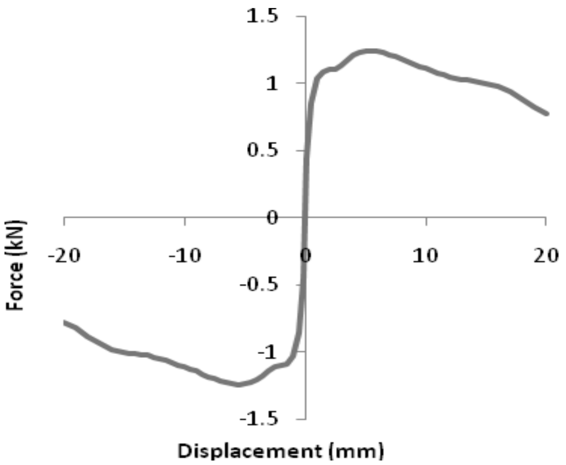


Figure 7: Non-linear spring characterizations of midply nails (Clarke, 2009)

The connections between the shear wall and the moment frame were modeled after bracket connections for CLT shear walls (Schneider, 2009).

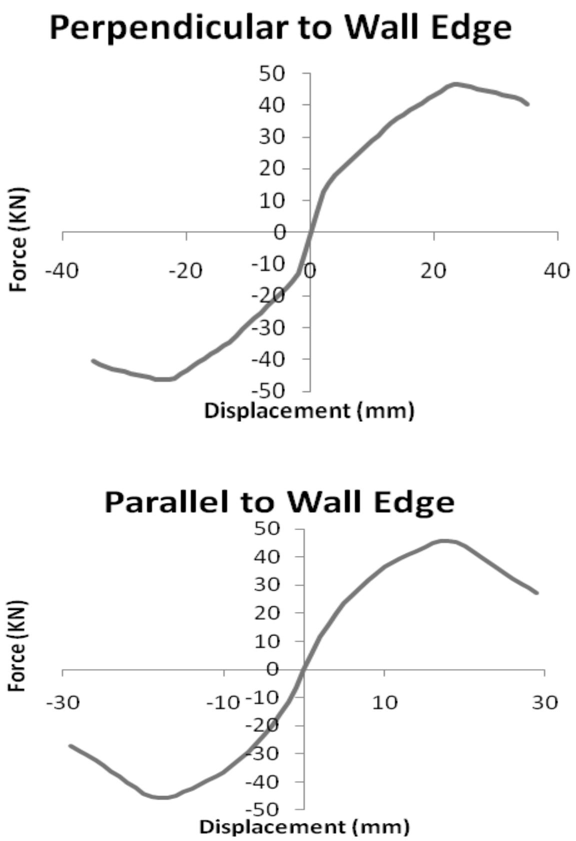


Figure 8: Force-displacement envelopes for both directions of connector between frame and wall (Schneider, 2009)

Note that in addition to these connectors, the model contained "gap" link elements; providing confinement from the frame around the wall. An initial gap of 6mm was provided at the top and at the sides of the frame wall. No gap was provided as the base of the wall as the wall will site directly on the frame.

5.2 *Seismic Design*

The NBCC defines allows for the use of an equivalent static procedure when building specific building irregularities are avoided. This procedure is defined by equation 1

$$V_{base} = \frac{M_v I_e}{R_d R_o} S(T_a) W \tag{1}$$

Where:

V_{base} = seismic base shear

$S(T_a)$ = spectral-acceleration with soil conditions

M_v = higher mode effect factor

I_e = Building importance factor

R_d = Ductility Factor

R_o = Over strength Factor

T_a = NBCC 2005 defined building period

The NBCC 2005 states that the period can be determined by with a model, but limit it to be not more than 1.5x the NBCC calculated building period for steel moment frames or 2.0x the NBCC calculated building period for shear walls.

$$T_{a.FRAME} = 0.085 h_n^{0.75}; \; T_{a.WALL} = 0.05 h_n^{0.75} \tag{2}$$

Where: h_n = structure height

The addition of structural infill walls has resulted in a decrease in the design period and the design ductility of the building resulting in an increased design yield demand.

Table 4: NBCC Equivalent Static Force

	Bare frame	Frame with infill wall	
Rd	5.0	3.0	
Ro	1.5	1.7	
Ta from model	1.32sec	1.16sec	
Ta design	1.16sec	0.96sec	
Base Shear	114.6 kN	214.2 kN	(+87%)

If we apply the increased loads as required from the NBCC 2005, we find that the total deflection is larger than found in the plain frame at its design load. Comparatively, the loads in the steel frame remain smaller than those found in the bare steel moment frame

Table 5: Frame Deflection and Stresses

	Bare frame	Frame and infill wall	increase/ decrease [% - (+/-)]
Roof Deflection	29.3 mm	42.8 mm	+18.8%
Moment* Col	148 KN-m	189 KN-m	+29.1%
Beam	232 KN-m	232 KN-m	-5.6%
Col Compression**	1076 KN	892 KN	-9.3%
Int Beam Moment	160 KN-m	81 KN-m	-49.4%

*Max forces occur in ground floor columns and beams
*Note: the NBCC design forces from Table are applied to the respective models

It is important to note that the change in moment is not consistent between the beams and columns. The moment observed increased in the columns and decreased in the beams.

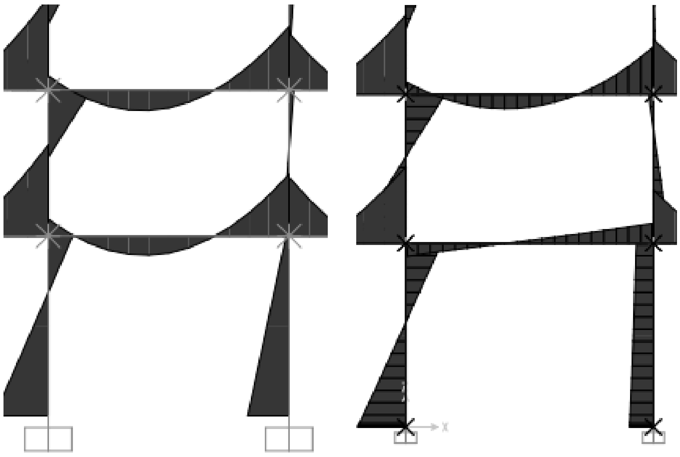


Figure 9: Moment distribution for bare steel frame (left); Moment distribution for steel frame and infill wall (right)

6 CONCLUSIONS

Hybridized structures offer many opportunities to use different materials to their best advantage. To achieve the desired performance, the steel and wood components need to be designed and combined to maximize their strengths. Steel should be used minimally in regions where extremely high stresses are located as well as for all the connections. Ductility is incorporated into the system with the deformation of steel connectors between wood elements and between wood and steel elements.

A case study involving a five storey steel frame was analyzed and compared with the same frame containing a structural wood infill shear wall. The infill shear wall was designed as a midply shear wall. The steel frame was modeled as a type D ductility moment frame and the shear wall was modeled in detail including the individual studs, the isotropic sheathing; the non linear behavior of the wall was incorporated with non-linear links representing the nails in the wall. The connections between the shear wall and the steel frame were modeled to represent the non linear behavior found in steel brackets developed for wood shear walls.

The NBCC 2005 equivalent static force procedure called for an increase of 87% to the applied load on the frame due the addition of infill shear walls. A non-linear analysis using the NBCC 2005 equivalent static loads resulted in a decrease in the design yield moment for the beams and columns at the locations where infill walls had been applied, despite the significant increase in the applied forces.

Further study may help determine if the increase in design loads required by the code is reasonable. The reduction in ductility and the cap for the buildings period to not account for the hybridization in this model; some combination of the ductility and caps for the 2 systems may be warranted. Various combinations as hybrid frames offer a different type of hybridization which might be also an approach for further studies.

Timber steel hybridization offers a wide field of variation where more research needs to be done.

7 REFERENCES

National Research Council of Canada. (2005). *National Building Code of Canada.* Ottawa: NRC.

Clarke. (2009). *Midply Shear Wlls Use in Non-residential Buildings.* University of British Columbia: Thesis (MASc).

CSA-O86. (2001). *Engineering Design in wood.* Ottawa: Canadian Standards Association.

CSA-S16. (2009). *Design of Steel Structures.* Ottawa: Canadian Standards Association.

Keenan. (1986). *Limit States Design of Wood Structures.* Toronto, ON: Canada Morrison Hershfield Ltd.

Khorasani. (2010). *Feasibility Study of Hybrid Wood Steel Structures.* University of British Columbia: Thesis (MASc).

Maloney. (1996). The Family of wood Composite Materials. *Forest Porducts Journal, Vol. 46, Issue 2* , 19-26.

Schneider. (2009). *Connections in Cross-Laminated-Timber Shear Walls Considering the Behaviour under Monotonis and Cyclic Lateral Loading.* University of Stutgart: Thesis (MASc).

Varoglu, Karacabeyli, Stiemer, Ni, Buitelaar, & Lungu. (2007). Midply Shear wall System: Performance in Dynamic testing. *ASCE Journal of Structural Engineering, Vol. 133, Issue 7* , 1035-1042.

Varoglu, Ni, Popovski, Karacabeyli, & Stiemer. (2006). Midply Wood Shear Wall System: Concept and Performance. *Journal of Structural engineering, Vol 132, Issue 9* , 1417-1425.

Yousuf, B. (2009). Seismic Design and Performance Evaluation of Steel-frame buuildings designed using the 2005 National Building Code of Canada. *Canadian Journal of Civil Engineering, Vol. 36, Issue 2* , 280-294.