FEASIBILITY STUDY OF TALL CONCRETE-TIMBER HYBRID SYSTEM

by

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Abstract

Although wood is widely used as a construction material, it is mostly limited to low and mid-rise residential construction, partially due to fire code restrictions. This limitation can be overcome by considering hybrid systems which combine wood with non-combustible materials.

This research presents an innovative wood-concrete hybrid system, suitable for tall buildings, where a concrete frame with slabs at every third story provides fire separation as well as stiffness and strength to resist gravity and lateral loads. The intermediate stories including their floors are constructed using light-frame wood modules. This approach reduces the environmental footprint of the building, reduces the building weight and therefore the seismic demand on connections and foundation, and speeds up the construction process.

For a novel system, numerical modeling is crucial to predicting its structural response to static and dynamic loading. This thesis studies the structural feasibility of the system by developing finite element models and assessing the structural behavior at the component and system levels when subjected to earthquake and wind loads. Nonlinear analyses are performed considering material and geometric nonlinearity using multiple ground motions to estimate the structure’s inter-story drift and base shear. The results demonstrate the feasibility of the proposed wood-concrete hybrid system for tall buildings in high seismic zones.
Lay Summary

Wood is an advantageous construction material because of its low weight and small carbon footprint. But tall wood buildings are not (yet) feasible, owing to fire regulations, earthquake and wind loads. Hence, an innovative solution combining concrete with wood is proposed where concrete frame resist earthquake loads while the timber modules create the livable space. With this approach, the weight of the structure is significantly reduced and sustainability is intact. This thesis evaluates if such a system is structurally feasible when subjected to earthquake loads.
Preface

This thesis presents the author’s contribution towards the collaborative project on “Innovative Solution for Hybrid Wood-Concrete Tall Buildings” in partnership with Tongji University, Shanghai and the University of British Columbia, Vancouver. All numerical models, analyses and results of this thesis are the original work of the author, executed under direct supervision of Dr. Thomas Tannert and Dr. Carlos E. Ventura.

The experimental results used as an input for validating the numerical study were conducted by Ouyang Lu of Tongji University and were used with permission by the principal investigator from Tongji University, Dr. Haebei Xiong.

The methodology and results presented in Chapter 3 of this thesis were published at the Structures Congress by American Society of Civil Engineers (Denver, CO, 2017): “Kaushik K., Tannert T., 2017. Feasibility Study of a Novel Tall Concrete-Wood Hybrid System. Proc. of the Structures Congress, ASCE, Denver, CO, 2017”.

The procedure and results presented in Chapter 4 and Chapter 5 were submitted as abstract to the World Conference on Timber Engineering (Seoul, South Korea, 2018). The manuscript was developed by the author; Dr. Thomas Tannert provided guidance and feedback.
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Last, but not the least, I would like to thank my parents and my brother for all their love and support throughout my life and continue believing in me.
Dedication

To my grandparents and my parents...
Chapter 1: Introduction

1.1 Background
To accommodate the growing world population, sustainable and affordable housing solutions are required (World Economic and Social Survey, 2013). Tall structures made of steel and concrete create a large carbon footprint. Hence, future structures should be more sustainable. Using timber as a structural material plays a pivotal role in satisfying this condition.

However, for tall buildings, the use of timber alone has limitations owing to the fire regulations in building codes. Also, there are shortcomings with respect to the ductility needed to provide seismic resistance in tall timber structures (Fairhurst, 2014). Most of these challenges can be overcome with the use of hybrid structures (Green & Karsh, 2012) which combine two or more materials and take advantage of both on component or system level (Taranath, 2005).

To assess the feasibility and predict the structural behavior of a novel hybrid system, finite element analyses (FEA) must be supported with experimental validation. Hence, research should include component level experimental and numerical studies followed by system level analyses. The system level study should be done considering material and geometrical nonlinearities to predict the structural and dynamic behavior under gravity and lateral loads.

1.2 Project Overview
The thesis provides a feasibility study of a novel concrete-timber hybrid system, which is a joint initiative between The University of British Columbia (UBC), Vancouver and Tongji University, Shanghai. The project investigates the possibility of developing a Hybrid-Wood-Concrete (HWC) structural system for tall buildings that use a combination of concrete and light-frame wood
modules as the main structural materials, reducing the carbon footprint of the building when compared to a pure concrete building. Preliminary laboratory tests were conducted at Tongji University. The author performed numerical analyses to determine the feasibility of the HWC system and to assess its structural response subjected to dynamic loads. Based on the results, in association with the International Joint Research Laboratory of Earthquake Engineering (ILEE), shake table tests will be performed on various configurations of the HWC system to determine the most suitable configuration. The final outcome of this project will be guidelines for the design of tall buildings using the HWC system. The contribution of the author in this project lies in the numerical modeling of the system’s structural response in both linear and nonlinear domains.

1.3 Objectives

The three main objectives of the research are:

1. To check the feasibility of the proposed HWC system subjected to gravity and lateral loads in high seismic zones in accordance with National Building Code of Canada and to compare the structural response of the hybrid system with a full concrete system to evaluate its advantages in terms of structural demand and sustainability.

2. To conduct a nonlinear parametric study at the component level (hybrid portal frame - HPF) of the HWC system, to understand the failure mechanism and the load sharing between timber and concrete frame and to assess the effect of aspect ratio, the spacing of nails and bolts, timber sheathings, timber shear wall openings on the HPF lateral capacity.

3. To conduct nonlinear dynamic analyses of the hybrid portal frame to assess the demand on the system and assign a performance criterion for the novel system and to perform
nonlinear dynamic analyses on multi-story numerical models considering different configurations and heights of the hybrid system.

1.4 Scope and Outline

In this thesis, the author’s contribution is limited to two-dimensional (2D) numerical modeling and analyses of the hybrid system. Hence, the impact of torsion is beyond the scope of this thesis. Also, the effect of time history analysis of wind load on the structure is not analyzed.

The thesis comprises six chapters. Chapter 2 reviews the literature including the necessity of hybrid structures, briefly discusses already existing hybrid systems, and summarizes the numerical and experimental studies which are relevant to this research. Chapter 3 explains the proposed HWC system, conducts a feasibility study as per NBCC, and compares the structural response of the hybrid system to a full concrete. Chapter 4 consists of a parametric study to investigate the failure and lateral load sharing mechanism between the concrete frame and the light wood frame shear wall. It assesses various parameters effecting capacity of the component level system. Chapter 5 presents a nonlinear dynamic analysis on full-scale level models, to check the response of the hybrid systems when subjected to gravity and lateral loads. Chapter 6 concludes the thesis with a summary and an outlook to further research.
Chapter 2: Literature Review

2.1 Motivation of the Project
The infrastructure development has seen a rapid boom throughout the world due to exponential population increase (Population Reference Bureau, 2016). The need for sustainable and affordable housing has become a world-wide priority. While conventional construction techniques prevail, owing to the huge carbon footprint, new technologies and materials need to be incorporated into construction practices. This is where timber comes into play. The following sections give a description of the motivation for the project.

2.1.1 Rise in Population
The 21st century has seen a major transformation in world population. With the current world population touching the 7.6 billion mark, the United Nations projected this number to reach 8.6 billion in 2030 and 9.8 billion in 2050. The current growth rate of population is around 1.1% per year, yielding an additional 83 million people annually. China and Canada, who are the participatory countries in this project, have an estimated population of 1.4 billion and 36 million in 2017 (Worldometers, 2017), respectively. Most importantly, the urban population of both the countries is increasing. As per UN-Habitat, currently 50% of the world’s population lives in urban environments and this number is likely to increase to 70% by 2040. This means 3 billion people in the world today, over the next 20 years will need a new home, (TED Talks, 2013) resulting in massive investment in infrastructure and affordable high-density housing. China has an urban population of approx. 60%, whereas 84% of Canada’s population lives in urban centers (Worldometers, 2017). Due to the large growth of urban population, the need for sustainable and affordable housing has become a priority for the countries.
2.1.2 Infrastructural Development

Investment in infrastructure leads to the growth of the country’s economy and the well-being of the people (Infrastructure Intelligence, 2017). It has been estimated that by 2030, the world will spend 57 billion dollars per year on infrastructure to realize global economic growth ambitions (McKinsey Infrastructure Practice, 2013). Infrastructure development in China has taken a huge leap forward in the last decade. The country has invested billions of dollars in massive infrastructure projects like roads, railways, housing, airports etc. China is spending almost 8.6% of the world’s GDP on infrastructures, which is the highest compared to all the developing and developed economies (Coy, 2016). Over the next 10 years, China has planned to move 250 million people into country’s rapidly growing megacities (Weller, 2017). To accommodate the population, China will need affordable and tall structures which can be easily constructed, also in seismic zones. Building these houses with traditional materials like steel and concrete would result in an enormous carbon footprint (Section 2.1.3). The need to use environmentally friendly and sustainable materials is hence clear for the construction industry in China.

2.1.3 Carbon Footprint of Infrastructure

Carbon footprint is defined as the total greenhouse gases emitted by an individual, event, organization or product (Carbon Trust, 2009) expressed in terms of equivalent tons of carbon dioxide (CO₂). Greenhouse gases like CO₂, methane (CH₄) and nitrous oxide (N₂O) absorb and emit radiation to keep the heat in earth’s atmosphere. But since the industrial revolution, there has been a 40% increase of greenhouse gases from 280 ppm in 1750 to 406 ppm in early 2017 (Butler & Montzka, 2017). The United Nations IPCC division estimates that, if the emission of greenhouse gases continue in this current trend, by the year 2036, the earth could pass a threshold of two degree Celsius global warming, affecting the ecosystem, biodiversity, and livelihoods of people.
The increase in carbon footprint or emission of greenhouse gases at an alarming rate is primarily due to human activities like combustion of fossil fuels, transportation, construction, and food etc. As per United States statistics, about 47% of the greenhouse gases are emitted from buildings alone, followed by transportation being 33% (TED Talks, 2013) which has a direct and severe impact on climate change. In the construction industry, just concrete and steel accounts for 8% of the total release of greenhouses. China, being the most populous developing country in the world, is posing a huge threat in increasing the carbon footprint in terms of rapid urbanization. About a quarter of the world’s carbon by consumption of energy is being emitted by China (US Energy Administration Information, 2017).

2.2 Sustainable Construction Material Timber

2.2.1 Relevant Properties of Wood

With the rapid rise in population, urbanization, and an increase in carbon footprints, the need for using sustainable materials in construction is more urgent in populous countries like China. Using conventional materials like concrete and steel requires 57% and 26% more energy, emits 81% and 34% more greenhouse gases, and releases 47% and 24% more pollutants (Canadian Wood Council, 2004) than using timber. Wood, being one of the oldest sustainable structural materials, meet the demand for construction of sustainable housing for the growing urban population throughout the world. Timber can store greenhouse gases, it is renewable and a low-carbon alternative which can be used to build structures.

Wood is an organic, hygroscopic and anisotropic material. The thermal, sustainable, mechanical and aesthetic properties of wood make it suitable to use as a sustainable construction material. It can change in size with variation in humidity, which can reduce their strength and therefore a
disadvantage during the fire. Wood as a light material is not perfect for sound isolation, but ideal for sound absorption. Wood prevents echo and noise by absorbing sound, which is also a reason to use wood as an acoustic material in concert halls. Wood is strongest in parallel to the grain while it shows weaker behavior perpendicular to the grain. In general, tensile strengths in tangential and radial directions are low compared to longitudinal strength, about 3% and 5% respectively (Slavid, 2005). Wood, while having a favorable weight to strength ratio has lower stiffness compared to other structural materials. Table 1 shows some mechanical properties of wood and compares them to conventional concrete and steel.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength (MPa)</th>
<th>Density (kg/m³)</th>
<th>Poisson Ratio</th>
<th>Modulus of Elasticity (MPa)</th>
<th>Compressive Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>350</td>
<td>7800</td>
<td>0.30</td>
<td>200000</td>
<td>400-1000</td>
<td>400-1000</td>
</tr>
<tr>
<td>Concrete</td>
<td>N/A</td>
<td>2300</td>
<td>0.21</td>
<td>20000</td>
<td>20-40</td>
<td>2.0-5.0</td>
</tr>
<tr>
<td>Timber</td>
<td>N/A</td>
<td>400-600</td>
<td>–</td>
<td>8000-11000</td>
<td>Parallel 30 Perpendicular 8</td>
<td>Parallel 6 Perpendicular 1</td>
</tr>
</tbody>
</table>

Figure 1 (a and b) shows the stress-strain plot for wood compressed parallel and perpendicular to grain respectively. It can be seen (Figure 1a) that for compression perpendicular to the grain, there is no true ultimate strength value. The failure criterion of the sample is not reaching an ultimate load but acceptance on the basis of the degree of distortion (Stalnaker & Harris, 1989). On the other hand, for compression parallel to grain, shown in Figure 1(b), point A refers to the highest value of stress reached which is the ultimate stress. After point A, until point B, the stress level is no longer proportional to strain, and the sample exhibits failure after point B.
Wood is renewable and stores carbon, rather than emitting since trees use carbon from the air to grow. The process of storing carbon dioxide by wood during its life cycle is called sequestration and it has been estimated that one cubic meter of wood can store one ton of CO₂ (Michael Green, 2013). Figure 2 shows the benefits of using wood as a construction material in terms of environmental impact during the life cycle of a typical North American home. It is obvious that all environmental impacts are significantly lower when compared to steel and concrete.
2.2.2 Shortcomings of Wood as Construction Material

Even though wood has been significantly used, there are shortcomings. While wood has low density and high strength to weight ratio, which offers lightweight structural solutions, resulting in lower seismic forces, it can be very flexible. But wood is brittle in tension and shear (Khorasani, 2011). Moreover, wood being hygroscopic material, gains or loses moisture, which effects dimensional stability and strength. Biotic deterioration of wood from decay, mold fungi, bacteria, and insects play significant roles. Another main disadvantage of wood is that it is a combustible material. Temperatures must be greater than 330 degree Celsius in order to burn wood spontaneously, otherwise, it will stop burning once the flame is exhausted. Also, thick wood members burn very slowly under a controlled rate, which is also the reason for using larger members for construction of tall wood buildings.

2.2.3 History of Timber Construction

Timber as a construction material has been prevailing since the post-Neolithic Construction Age after 5000 BC. It is estimated that the first bridges made by humans were probably wooden logs placed across streams. By 650 BC, the ancient Greek temples were built of wood, after the Greeks invented the truss for longer spans (Klein, 1998). During the middle ages until 1000 AD, timber construction was popular in Northern Europe and most buildings were constructed using timber. China and Japan have a tradition of tall wood buildings in the form of Pagodas. The Yingxian pagoda in China, which was built in 1056 AD is 67m tall is the world’s oldest existing multi-story timber building. The building is constructed as post and beam structure, where exterior and interior circular arrangement of timber columns support the five visible stories and four additional floors hidden within. Horyu-Ji pagoda is another five-story wood pagoda worth mentioning because of its unique technique as a seismic resistant structure. This 32.5m high structure, built in 711 AD,
does not have central load-bearing beams and the individual floors are not solidly connected but piled atop one another with loose fitting timber brackets. This allows the floors to sway in a slithering manner, with the floors moving independently, minimizing the shift of gravity and preventing the structure to collapse. (Tarantola, 2011)

With timber construction being common throughout the world, mid-rise timber buildings were being built in Canada from mid-19\textsuperscript{th} century to mid-20\textsuperscript{th} century using a brick and beam (B&B) structural system. These buildings were up to nine stories tall, with heights up to 30m. Around 191 buildings were constructed using the B&B technique in Toronto while in Vancouver 74 such structures were constructed from 1850 to 1941.

In Vancouver, BC, the brick and beam buildings are located mainly in Gastown and Yaletown. Typical examples of B&B buildings are ‘The Landing’ (Figure 3) and ‘The Leckie’. The Landing, constructed in 1905, is a nine-story B&B building with a total floor area of 16,258m\textsuperscript{2}. This building, which was originally constructed as the warehouse, was renovated to meet modern building codes in 1987. The Leckie is a factory/warehouse building which was completed in 1913. The structure was seismically upgraded in the year 1991 (Koo, 2013).

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{landing_building.png}
\caption{The Landing building in Gastown, Vancouver (Koo, 2013).}
\end{figure}
2.2.4 Resurgence of Timber Construction

The 20th century was an age of concrete and steel, and owing to fire regulations, timber construction was mostly limited to residential houses. But the 21st century has seen a comeback in timber construction. With the development of cross-laminated timber (CLT) and other engineered products, timber buildings more than six stories have been built. These buildings have excellent energy efficiency, reduced carbon footprint, and fast construction speed.

The wooden Bridport House, located in Hackney, London (Figure 4) is an eight-story residential house constructed entirely from CLT. The structure has been designed in a way that the load bearing CLT walls are placed in a variety of positions across the floor to resist the lateral as well as gravity loads. Apart from the fast construction time of just 12 weeks, the Bridport House stores 2113 tons of carbon which is equivalent to 29 years of operational energy of the building (Timber in Construction, 2015). The H8 building in Bad Aibling, Germany, is an eight-story building of 25m height for commercial/residential purpose. The load bearing and the non-load bearing walls of the building are constructed of solid wood fully encapsulated in fiber boards. The ceilings are constructed with CLT encapsulated with plasterboards (Binderholz, 2017).

*Figure 4: Bridport house in Hackney, London (Timber in Construction, 2015).*
In the province of British Columbia (BC), Canada, the current tallest timber building is the Wood Innovation and Design Centre (WIDC) in Prince George (Figure 5). The eight-story, 29.5m tall structure is an innovative combination of glulam post and beam frames connected by aluminum connectors, with a custom designed CLT floor system. The building is balloon framed, which means the columns are superimposed one above the other, with end grain to end grain bearing, which minimizes the vertical shrinkage. The lateral load resistance is provided by the CLT elevator and stair cores (WoodWORKS, 2015). The shear walls are anchored to the foundations using a combination of shear brackets and hold down anchors. WIDC, constructed in 2014, is a showcase for local wood products and BC’s expertise in the construction of tall wood buildings.

*Figure 5: Construction sequence of the WIDC building (WoodWORKS, 2015).*
2.2.5 Hybridization of Timber

Most of the structural challenges, that pure timber buildings face, can be overcome with the use of hybrid structures (Green & Karsh, 2012). Hybrid structures refer to those structures which combine two or more materials and take advantage of these at component or system level (Taranath, 2005).

Component level hybridization is achieved when two or more materials are employed in the structural element, similar to hybrid beams/columns, diaphragms etc. Figure 6 shows some examples in the component level hybridization.

![Hybrid Connection Diagrams](image)

*Figure 6: a) Glulam column with inserted steel section b) Hybrid jointed ductile connection. Adapted from: Khorasani (2011)*

System level hybridization refers to structures which are constructed using timber combined with steel or concrete or both. Loads are been shared between the materials which makes it possible to construct tall and irregular structures subjected to dynamic loading. Examples of system-level hybridization are hybrid frames, steel moment resisting frames with timber floor joist and diaphragm, hybrid roof trusses etc. With the advent of the mass timber and engineered wood products such as CLT, hybrid structures can be constructed with majority of the members of timber, while concrete can be used to provide additional stiffness and steel can be used to provide ductility.
One major structural advantage of hybrid wood structures for seismic design is the lower weight, as the base shear of the structure is directly proportional to the weight of the structure, hence wood hybrid structures will attract lower seismic forces compared to a concrete or a steel structure. Also, the lateral resistance of the hybrid structure is increased significantly. A study conducted by He et al. (2014) shows that the lateral load carrying capacity of a steel moment frame increased by 64% with the inclusion of infill timber shear walls. Hence, hybrid structures can go tall, owing to the lightweight timber and high force retaining material like concrete and steel.

As discussed in Section 2.2, timber is renewable and has a low carbon foot-print which makes hybrid systems more sustainable and environmentally. One other advantage is that wood systems can be easily prefabricated, reducing costs and time of construction. For example, the installation of the timber elements of the Brock Commons building at UBC just took over 9 weeks, which is much faster than traditional concrete casting and construction. Hybrid construction has one more advantage with respect to fire safety. In Canada, there is a six stories restriction for timber structure, but wood, when combined with other materials, has no limitation in height.

### 2.2.6 Proposed and Existing Hybrid Structural Systems

The 21st Century has seen an uprise in hybrid structures around the world. By utilizing the strengths and unique properties of different materials, it has been possible to construct tall hybrid structures. Most tall hybrid structures utilize CLT panels to support vertical and horizontal loads. Others use glulam post and beam with braces in combination with reinforced concrete core as the lateral force resisting systems or ductile steel beams as energy dissipation units. Figure 7 shows an overview of hybrid structures being constructed or proposed along with story and materials being used.
At fourteen stories tall, the Treet building (Figure 8) in Bergan, Norway, completed in 2015, is on the threshold for being considered a tall hybrid structure with reference to existing guidelines (Council on Tall Buildings and Urban Habitat, 2015). The building consists of partially connected braced glue-laminated timber (glulam) frames which act as the primary vertical and lateral load resisting system. The system acts as a ‘cabinet rack filled with drawers’ (Malo et al., 2016), the shelves of the so-called cabinet rack being transfer stories composed of glulam trusses with a 200mm reinforced concrete topping slab and the so-called drawers being prefabricated timber modules (Foster et al., 2016). The stacked modules act as the vertical support only at the transfer stories and the bearing onto the glulam frame via the RCC topping slab. The building has thus been classified here as a mixed timber/concrete structure or a single material timber building above the concrete first-floor podium.
The ‘Brock Commons’ (Figure 9), constructed on the UBC Vancouver campus, is an 18-story concrete-timber hybrid structure, and is with 53m currently the tallest mass-timber structure in the world. The structure has a hybrid structural system with a one-story concrete podium, two concrete cores and 17 stories of mass timber topped with a prefabricated steel beam roof. The concrete cores provide the necessary stiffness to account for the lateral force on the system, while the mass timber acts as a vertical load supporting system. Glulam columns with steel connectors provide a direct load transfer between the columns and support CLT panels on a 2.85m x 4.0m grid that acts as a two-way slab diaphragm (Acton Ostry Architects Inc, 2015). Vancouver, being a high seismic zone, the design of the structure has been done as per NBCC 2015 (NRC, 2015)
The Standparken and Limnologen buildings in Sweden, combine mass timber, concrete, and steel as construction materials. While concrete is used only in the first floor which acts as a podium for the structures, steel rods run the full height of the structure through CLT shear walls, acting as continuous ties against uplifts (Foster et al., 2016). These ties provide the primary tension force path of the lateral load resisting system. Another hybrid structure worth mentioning is the Framework building which is under construction in Portland, Oregon. Framework, a 12-story hybrid structure uses steel ties to externally post tension CLT shear walls, creating a rocking wall mechanism to accommodate the seismic loads.

Finding the Forest through the Trees (FFTT) system is a timber steel hybrid system concept proposed for high rises up to thirty stories in height in high seismic zones like Vancouver, BC. The system (shown in Figure 10) utilizes glulam beams and columns and CLT shear walls for resisting gravity and lateral forces with interconnecting steel members to provide ductility. FFTT system is a strong column-weak-beam concept which reaps the benefit of the lightweight, strength, stiffness, and environmental benefits of engineered timber, and exploits the ability of steel to dissipate energy and provide a ductile failure mechanism (Green & Karsh, 2012).

![Figure 10: Structural elements of the FFTT system (Green & Karsh, 2012)](image-url)
2.3 The Concept of the HWC System

2.3.1 Introduction

Tongji University and UBC proposed a novel hybrid wood concrete (HWC) box system that uses a combination of mass timber and concrete for tall buildings to reduce the carbon footprint. The concept took into consideration the fire code of China (GB 50016-2014, 2014), which limits timber buildings to three stories. This system follows a ‘main structure + substructure’ (Ying, 2016) type of construction which is a common structural concept for buildings in China. The HWC system consists of a concrete core and concrete slabs at every third story in combination with timber shear wall to provide the necessary stiffness and strength to resist the gravity and lateral loads. The intermediate floors as substructure are constructed using light wood frame modular boxes which creates the livable space and the shear walls take part in resisting the lateral force. The modular boxes are connected with the concrete moment frames and the concrete core shear wall by a series of bolts (M14, M16). The conceptual idea of the HWC system is shown in Figure 11.

Figure 11: Concept of tall hybrid timber-concrete box system (units in mm).
The key advantages of the HWC concept are:

1. Taking advantage of concrete in terms of its strength and stiffness and timber in terms of sustainability, low weight, and lateral force resistance.

2. Low weight has a significant advantage in attracting less seismic base shear, resulting in lower demand than a pure concrete structure.

3. The prefabricated timber modules are fast in installation and are replaceable.

4. The HWC system meets the fire design requirements as per Chinese and Canadian codes.

2.3.2 Components of the HWC System

2.3.2.1 Main Structure

The main structure of the HWC system consists of the concrete core, concrete moment frames and concrete slabs at every third story. Reinforced concrete core shear walls as the lateral force resisting system are coupled together by flat slabs and perimeter gravity columns to carry gravity loads. The slabs and the moment frame (consisting of beams and columns) are designed to resist gravity loads and not the lateral force, hence these elements are expected to behave linearly during earthquakes. The shear walls, on the other hand during a rare seismic event, are expected to undergo nonlinear deformation, where the reinforcement yields and plastic hinges are formed. Figure 12 shows an ideal yield mechanism of the shear wall. In the HWC system, the concrete frame is the second level of defense during a seismic event, which means, it should not yield during a low hazard seismic event (probability of occurrence 10% in 50 years), instead only the substructure (timber modules) should yield in such events.
2.3.2.2 The Substructure

The substructure consists of the wood modules, which create the livable space in the intermediate stories and consist of light-frame shear walls and diaphragms. Light-frame construction is made from an assembly of dimensional lumber frames and wood-based sheathing panels connected with nails or screws to the frames (Giovanni et al., 2013). A wood diaphragm is a flat element which acts like a deep, thin beam, where the panels resist shear force, while the diaphragm edge members, called as chords which may be joists, trusses, studs or top plates resist bending stress. Shear walls, on the other hand, acts as a cantilevered diaphragm, where the lateral load collected through the chords of the diaphragm at the top is transmitted to the foundation owing to its ability to behave in a highly ductile manner during dynamic loading (Loo & Chouw, 2012). The primary elements in the timber shear wall providing the ductility are the nails. When a small lateral load is applied initially, the shear wall experiences linear elastic deformation. With an increase in load, the nail connections on the edge of the sheathings behave nonlinearly. The lateral load carrying capacity of the shear wall decreases with a decrease in lateral stiffness. The shear wall dissipates the energy by undergoing plastic deformation in nails as shown in Figure 13.
In the HWC system, the timber shear walls are the first line of defense during a seismic event. The timber shear walls are expected to yield during a medium hazard earthquake (probability of occurrence 5% in 50 years) and to reach their ultimate strength in a high hazard earthquake (probability of occurrence 2% in 50 years).

2.3.2.3 Connections between Main and Substructure

The connections between the main and the substructure are the critical units, as they hold the system together and transfer shear and axial forces between the concrete and the timber elements. Bolts (M14, M16, etc.) can effectively connect both systems and transmit the lateral force effectively, behaving as a rigid element during the moderate level earthquake (Ying, 2016). In case of large displacement, the bolts start yielding and behave in a flexible manner, transmitting only a part of the lateral force to the substructure. The yielding resistance of the bolts is defined by CSA 086-14 design provisions (Canadian Standards Association, 2014). In the HWC system, the connections are the third line of defense in an event of an earthquake. The bolts are designed to not yield before the shear walls and the concrete members but keep the two systems fully connected.
Summarizing the yield mechanisms of the HWC system:

1) Low hazard level earthquake: Elastic deformation of timber shear wall, with no damage in other elements.

2) Medium hazard level earthquake: Yielding of timber shear wall.

3) High hazard level earthquake: Ultimate strength of timber shear wall is reached, formation of hinges in the concrete elements followed by yielding of the bolts as the intensity of high hazard level earthquake increased.

2.4 Experimental Research on the HWC Concept

The HWC system is a novel system, which means, the numerical model must be supported with experimental data. Component level testing has already been performed at Tongji University in 2016 and was reported by Lu (2016). The mechanical performance of single bolted connections between concrete and timber was evaluated with the following objectives:

1. To understand the load slip curve, failure model, stiffness, capacities of connections joining concrete timber system.

2. To understand the influence of bolt diameter, bolt grade, the wood thickness on shear capacity of the hybrid component.

2.4.1 Methodology

Timber-concrete specimens (TCS) were prepared, connected by bolts ranging from 12mm to 18mm with timber depth of 76mm and 114mm, with and without steel plate washers (see Table 2 and Figure 14). The timber used in this experiment was spruce-pine-fir (SPF). Monotonic and
Cyclic tests were performed to evaluate the shear capacity following the ASTMD 5652 (2015) and ASTM E2126, method B (2009) loading protocols, respectively.

Figure 14: Schematic of a TCS connected with single bolt and plate (Lu, 2016)

Table 2: TCS with variation in parameters.

<table>
<thead>
<tr>
<th>Series</th>
<th>Bolt diameter, ( d ) (mm)</th>
<th>Wood thickness, ( h ) (mm)</th>
<th>Bolt grade</th>
<th>Yield load (kN)</th>
<th>Yield displacement (mm)</th>
<th>Ultimate load (kN)</th>
<th>Ultimate displacement (mm)</th>
<th>( h/d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>76</td>
<td>4.8</td>
<td>11.8</td>
<td>6.1</td>
<td>18.9</td>
<td>64.6</td>
<td>6.3</td>
</tr>
<tr>
<td>2</td>
<td>14</td>
<td>76</td>
<td>4.8</td>
<td>11.8</td>
<td>4.8</td>
<td>26.3</td>
<td>49.1</td>
<td>5.4</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>76</td>
<td>4.8</td>
<td>13.4</td>
<td>5.1</td>
<td>26.0</td>
<td>41.3</td>
<td>4.7</td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>76</td>
<td>4.8</td>
<td>14.1</td>
<td>4.2</td>
<td>26.8</td>
<td>46.9</td>
<td>4.2</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>114</td>
<td>4.8</td>
<td>15.7</td>
<td>14.7</td>
<td>36.5</td>
<td>45.9</td>
<td>9.5</td>
</tr>
<tr>
<td>6</td>
<td>14</td>
<td>114</td>
<td>4.8</td>
<td>22.4</td>
<td>11.4</td>
<td>54.2</td>
<td>64.0</td>
<td>8.1</td>
</tr>
<tr>
<td>7</td>
<td>16</td>
<td>114</td>
<td>4.8</td>
<td>24.9</td>
<td>16.2</td>
<td>60.3</td>
<td>91.5</td>
<td>7.1</td>
</tr>
<tr>
<td>8</td>
<td>18</td>
<td>114</td>
<td>4.8</td>
<td>24.4</td>
<td>9.0</td>
<td>58.9</td>
<td>86.4</td>
<td>6.3</td>
</tr>
<tr>
<td>9</td>
<td>12</td>
<td>114</td>
<td>8.8</td>
<td>25.2</td>
<td>25.6</td>
<td>60.9</td>
<td>96.0</td>
<td>9.5</td>
</tr>
<tr>
<td>10</td>
<td>18</td>
<td>76</td>
<td>8.8</td>
<td>17.2</td>
<td>6.7</td>
<td>32.8</td>
<td>34.3</td>
<td>4.2</td>
</tr>
</tbody>
</table>

2.4.2 Results and Discussion

The failure modes observed in the specimens were – depending on parameter combination – splitting of timber parallel to the grain, local compression failure of wood under the plate and shear
failure of the bolts (single and double plastic hinges), see Figure 15. For the monotonic loading, it was observed that the failure mode was local compression of the wood under the washer and in combination with timber splitting with shear failure in bolts. For the reversed cyclic loading, the failure mode was always shear failure of bolts because of the weak behavior of bending of bolts.

![Figure 15: a) Splitting of timber parallel to grain b) Local compression failure of wood under the plate c) Shear failure of bolts forming single/double plastic hinges (Lu, 2016)](image)

The monotonic test results of the TCS are shown in Figure 16, Figure 17 and Figure 18. The cyclic loading results are incorporated in Appendix A. Figure 16 shows the shear capacity of the specimens with a 76mm thickness of the wood and no plate attached. In Figure 17, a steel plate of 60x60x5mm is added to the specimens to assess the shear capacity. Figure 18 shows the shear capacity with a timber thickness fixed at 114mm. It was observed that the initial stiffness of the specimens increased with increase in bolt diameter. The presence of a washer increased the yield and ultimate strength. From Figure 16 and Figure 17, the ultimate capacity with 18mm bolt almost doubled with the incorporation of the steel plate. The bolt grade can also enhance both the yield and ultimate capacity of the specimens. The preliminary experimental results, gave an overview of the connection behavior between the concrete and timber sample and the shear capacities. The results from these initial studies were taken to calibrate the parametric study and multistory study in Chapter 4 and Chapter 5 respectively.
Figure 16: Capacity of TCS with 12-18mm bolts, 76mm SPF thickness, without steel plate (Lu, 2016)

Figure 17: Capacity of TCS with 12-18mm bolts, 76mm SPF thickness, with steel plate (Lu, 2016)

Figure 18: Capacity of TCS with 12-18mm bolts, 114mm SPF thickness, without steel plate (Lu, 2016)
Chapter 3: Linear Feasibility Study of Tall Concrete-Timber Hybrid System

3.1 Overview

This chapter evaluated the feasibility of the HWC system as per the NBCC 2010 (NRC, 2010) provisions in a linear analysis without considering material or geometric non-linearity. The HWC system was compared to a conventional concrete model to assess the structural response differences based on total lateral deformation, inter-story drift and base shear. This chapter included gravity load and equivalent lateral static force method (ESFM) analysis followed by a linear time history analysis. All models were developed in SAP 2000, V-18 (CSI., 2016) a commercial finite element software.

3.2 Numerical Investigation of the HWC system

3.2.1 Building Layout and Concept

The 30-story buildings were assumed to be in Vancouver, BC, Canada, with site class C. The first option was the HWC system and the second option was a conventional concrete structure. The height of both the buildings were 94m with a floor area of 784 m² (28m x 28m). In the HWC structure, concrete slabs occurred at every 9.4m height, with timber modules connected to the intermediate floors, while the concrete structure had slabs at every 2.9m. Sketches of the layout and elevation of the hybrid and concrete model for three-story including the members and elements are shown in Figure 19 and Figure 20, respectively.
3.2.2 Numerical Model Development

To model the proposed system, seven main elements need to be considered:

1. Concrete beams
2. Concrete columns
3. Concrete core
4. Concrete slabs
5. Light wood frame shear wall (LWFS)
6. Timber diaphragms
7. Connections
3.2.2.1 Concrete Elements

Elastic frame objects are used to model beams, columns, braces, truss etc. in a planar and three-dimensional system which are straight lines used to connect two nodes and accounts for biaxial bending, torsion, axial deformation and biaxial shear (Bathe & Wilson, 1976). Material non-linearity can also be modeled using frame hinges. In the concrete and the HWC model, the beams and columns were modeled as moment frame objects, considering rigid beam-column joint. The columns in the HWC system were slender compared to the pure concrete structure as concrete slabs occurred at every third story in the HWC system. Minimum flexural steel reinforcement is required in the bending and compressive members as per CSA A23.3-14 (2014). For the concrete core and slabs, thin shell elements were used. A shell is a three or four-node area object used to the model membrane and plate bending behavior (shown in Figure 21). Shell element may be homogeneous or layered through the thickness. In the models, four nodes, homogeneous shell element had been used. One layer of reinforcement had been used and edge constraints were used for the concrete core for equal displacement behavior with the rest of the structure. In case of the slabs, diaphragm constraints were used to link joints within a plane such that, they move together as a planar diaphragm against in-plane deformation but susceptible to out-of-plane deformation.

Figure 21: Four node homogenous shell element. Adapted from: CSI (2012)
CSA A23.3-14 (2014) suggests the concept of proportioning the concrete elements to account for the inelastic behavior during a seismic event, reducing the section properties of the elements to determine the forces and deflection in them. The gross moment of inertia ($I_g$) and gross sectional area ($A_g$) is multiplied by factors as shown in Table 3, to evaluate the reduced section properties. Where $I_e$ is the effective moment of inertia, $A_{xe}$ is the effective sectional area, $\alpha_c$ is $0.5 + 0.6 \frac{P_s}{f_c A_g}$ where $P_s$ is the axial load, $f_c$ is compressive strength of concrete, and $\alpha_w$ is $1.0 - 0.35(R_d - 1)$, where, $R_d$ is the ductility factor.

### Table 3: Section properties for analysis.

<table>
<thead>
<tr>
<th>Element type</th>
<th>Effective property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>$I_e = 0.4I_g$</td>
</tr>
<tr>
<td>Column</td>
<td>$I_e = \alpha I_g$</td>
</tr>
<tr>
<td>Slab frame element</td>
<td>$I_e = 0.2I_g$</td>
</tr>
<tr>
<td>Wall</td>
<td>$A_{xe} = \alpha_w A_g$; $I_e = \alpha_w I_g$</td>
</tr>
</tbody>
</table>

#### 3.2.2.2 Timber Elements

The timber elements in the HWC system are the shear walls and the diaphragms of the intermediate modular boxes composed of studs, nails, and sheathings. These elements can be numerically modeled as linear link elements incorporating the uncoupled/coupled stiffness. A link object joins two points $i$ and $j$ separated by a length $L$, where linear, nonlinear or frequency dependent properties can be assigned in six degrees of freedom. It can be modeled to represent axial, shear, torsion or pure bending behavior (shown in Figure 22).
In the HWC system, the stiffness of the timber shear wall and diaphragm was calculated according to with the Technical Specification of Wood Frame Construction of China (DG/TJ08-2059-2009, 2009) using Equations 1 and 2:

\[ K = \sum_{i=1}^{3} \gamma_1 \gamma_2 \gamma_3 k_d L_w \]  

\[ k_d = \frac{1}{\frac{2h_w^3}{3EAL_w} + \frac{h_w}{1000G_s} + \frac{h_w d_n}{L_w f_{vd}}} \]  

where \( \gamma_1 \), \( \gamma_2 \), and \( \gamma_3 \) are environmental adjustment factor, shear wall aspect ratio adjustment factor and stiffness adjustment factor, respectively, \( h, A, L \) and \( d \) are the height, sectional area, length and depth of the shear wall, \( E \) is the modulus of Elasticity, \( G \) is the structural panel’s factor equivalent shear stiffness and \( f_{vd} \) is the bearing capacity design value. The stiffness values of the shear walls and diaphragms are summarized in Table 4 and were assigned in a single degree of freedom in the structure, mainly to account for the lateral force.
Table 4: Stiffness values of the light wood frame shear wall and wood floor

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimension</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light wood Frame Wall</td>
<td>12-meter shear wall</td>
<td>25.8</td>
</tr>
<tr>
<td></td>
<td>8-meter shear wall</td>
<td>12.3</td>
</tr>
<tr>
<td>Wood Floor</td>
<td>12x8 meter floor</td>
<td>20.9</td>
</tr>
<tr>
<td></td>
<td>8x8 meter floor</td>
<td>10.5</td>
</tr>
</tbody>
</table>

3.2.2.3 Connections

The timber modules in between every three floors in the HWC need to be attached to the concrete mainframe using M14 (14mm diameter) bolts. These connections were modeled as linear links in where two degrees of freedom were defined to account for the shear as shown in Figure 23.

Figure 23: Link element representing shear behavior in connections. Adapted from: ILEE (2015)

The connections linked the concrete frame and the timber link elements through a rigid frame, with a spacing of 400mm. The rigid frames are sections with a high modulus of elasticity ($E$) where, the moment is being released at both the ends ($M_2, M_3$), which cannot take any lateral force. The reason for using rigid elements was to link the timber elements via the connection to the concrete frame, otherwise, the timber elements would have to be hinged to the concrete frame, which was not realistic in actual practice. The linear stiffness of the M14 bolts was calculated as per ASTM E2126-09 (2009) which is equal to secant stiffness between origin and $0.4 \, P_a$ (where
$P_a$ is the ultimate load-bearing capacity of the connection, as shown in Figure 24. The stiffness values parallel and perpendicular to the grain of the lumber are shown in Table 5.

![Load-displacement curve of M14 bolted connection](image)

*Figure 24: Load-displacement curve of M14 bolted connection. Adapted from: ASTM E2126-09 (2009)*

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimension</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connections</td>
<td>Parallel to grain</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>Perpendicular to grain</td>
<td>0.4</td>
</tr>
</tbody>
</table>

*Table 5: Stiffness value of the M14 bolts*

3.2.3 Materials and Structural Member Specifications

A concrete grade of compressive strength 30MPa was used. The stiffness of the links representing timber members and connections were assigned as per sections 3.2.2.2 and 3.2.2.3, respectively. The structural member's specifications were initially proposed by Ying (2016) at Tongji University, China as per the Chinese building code. The member sizes (shown in Table 6) were taken for the initial modeling and analysis of the two numerical models subjected to gravity loading and lateral loading based on the equivalent static force method. All load combinations and gravity loading were applied as per NBCC 2010, but the lateral force calculation was based on the 2015 Uniform Hazard Spectrum (UHS) of Vancouver (shown in Figure 26). Figure 25 shows the FEA model of the HWC and the concrete structure respectively.
Table 6: Structural member specifications for numerical models

<table>
<thead>
<tr>
<th>Member description</th>
<th>Dimension (mm)</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete columns</td>
<td></td>
<td></td>
</tr>
<tr>
<td>600 x 600</td>
<td>10th Floor</td>
<td></td>
</tr>
<tr>
<td>800 x 800</td>
<td>6-9th Floor</td>
<td></td>
</tr>
<tr>
<td>1000 x 1000</td>
<td>2-5th Floor</td>
<td></td>
</tr>
<tr>
<td>1200 x 1200</td>
<td>1st Floor</td>
<td></td>
</tr>
<tr>
<td>Concrete beams</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300 x 700</td>
<td>Girder</td>
<td></td>
</tr>
<tr>
<td>250 x 500</td>
<td>Secondary beam</td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>100</td>
<td>Thickness</td>
</tr>
<tr>
<td>Core concrete tube</td>
<td>300</td>
<td>Outer thickness</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>Inner thickness</td>
</tr>
<tr>
<td>Stud (SPF)</td>
<td>40 x 140</td>
<td></td>
</tr>
<tr>
<td>Sheathing (OSB)</td>
<td>12</td>
<td>Thickness</td>
</tr>
<tr>
<td>Joist (SPF)</td>
<td>40 x 235</td>
<td></td>
</tr>
</tbody>
</table>

Figure 25: a) Model of the HWC system; b) Model of the conventional concrete system
3.2.4 Loads and Load Combination

3.2.4.1 Gravity Load

Gravity loads (dead load, live load, snow load and rain load) were applied to the numerical models as per NBCC 2010. The specified dead load for the structural members are the self-weight of the members, partition load or a load of any permanent structure. The live load depends on the use and occupancy of the floor, and the value was selected from NBCC considering the building to be for a residential purpose. Also for the HWC system, since the intermediate floors are composed of link elements representing timber floors, the gravity loads had been applied on the concrete floor every third story. Also, the weight of the timber shear walls was applied on the girders as uniformly distributed loads. The snow load was calculated and applied considering Vancouver as the location. The gravity loads applied to the floors and roof have been summarized in Table 7.

Table 7: Gravity loading as per NBCC, 2010

<table>
<thead>
<tr>
<th>Type of load</th>
<th>Floors (kPa)</th>
<th>Roof (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load</td>
<td>4.0 + partition wall weight</td>
<td>3.0</td>
</tr>
<tr>
<td>Live load</td>
<td>1.9</td>
<td>1.9</td>
</tr>
<tr>
<td>Snow load</td>
<td>-</td>
<td>1.7</td>
</tr>
</tbody>
</table>

3.2.4.2 Lateral Load

The lateral loads acting on a structure are earthquake and wind loads. This section focused on seismic load only. Earthquake load is calculated based on the equivalent static force procedure which substitutes the effect of dynamic loading of an expected earthquake by a static force (base shear) distributed on a structure. Article 4.1.8.11 in NBCC 2010 describes the procedure to calculate the base shear on the structure based on the UHS, as per Equation 3 (NBCC, 2010):
\[ V = \frac{S(T_a)M_vI_eW}{R_dR_o} \]  

(3)

Where \( S(T_a) \) is the spectral acceleration value from the UHS at the fundamental period of the building, \( M_v \) is the factor accounting higher mode effects, \( I_e \) is the importance factor of the structure, \( W \) is the weight of the building (dead load+25% snow load), \( R_d \) is the ductility factor of the seismic force resisting system (SFRS) to account for inelastic behavior and \( R_o \) is the overstrength factor of the SFRS. The base shear \( V \) is then distributed at any level, \( h_i \) of the structure:

\[ F_i = \frac{W_i h_i}{\sum_{j=1}^{n} W_j h_j} (V - F_t) \]  

(4)

Where \( F_i \) is the lateral force applied at level \( i \), \( F_t \) is the additional force to be applied at the top of the structure to account for higher modes, \( W_i \) and \( W_j \) are the lumped story mass at level \( i \) and \( j \) respectively, \( h_i \) and \( h_j \) are the heights of the masses \( W_i \), \( W_j \) above the level of application of the seismic action and \( n \) is the number of lumped masses along the height of the structure.

For the analysis of the models in this section, the UHS of Vancouver, BC, site class C and based on the probability of exceedance 2% in 50 years (National Research Council of Canada., 2015) with 5% damping was used (shown in Figure 26). The peak ground acceleration (PGA) and peak ground acceleration (PGV) of the UHS were 0.375g and 0.548 (m/sec) respectively. The importance factor (\( I_e \)) was taken as 1 for a normal category building and higher mode factor (\( M_v \)) as 1 considering the SFRS as moment resisting frame type. The ductility and overstrength factors of the SFRS must be taken as the lowest of the values if the structural system has more than one SFRS (NBCC, 2010). Since the concrete and the HWC models have more than one SFRS, namely, the moment resisting frame and shear walls (concrete and timber), the values were taken as 1.5 as \( R_d \) and 1.3 as \( R_o \) for the analysis.
Figure 26: Uniform Hazard Spectrum of Vancouver, site class C, 2% in 50 years (NRC, 2015)

3.2.4.3 Load Combinations

NBCC 2010 suggests different load combinations for the ultimate limit state (ULS) for the structure to be analyzed as the gravity and the lateral loads act simultaneously on the structure. For the buildings, to have sufficient strength and stability, the loads are amplified by applying factors as shown in Table 8. In the numerical analysis of the concrete and HWC system, 13 different load combinations were applied and analyzed to assess the structural response. Where $D$, $L$, $W$, $S$, $E$ represents dead, live, wind, snow, and earthquake load respectively. Also, $W$ and $E$ loads are applied in both the directions.

Table 8: Load combinations for ultimate limit states (NBCC, 2010)

<table>
<thead>
<tr>
<th>Combinations</th>
<th>Primary load</th>
<th>Companion load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combination 1</td>
<td>$1.4D$</td>
<td></td>
</tr>
<tr>
<td>Combination 2</td>
<td>$1.25D + 1.5L$</td>
<td>$0.5S$ or $0.4W$</td>
</tr>
<tr>
<td>Combination 3</td>
<td>$1.25D + 1.5S$</td>
<td>$0.5L$ or $0.4W$</td>
</tr>
<tr>
<td>Combination 4</td>
<td>$1.25D + 1.4W$</td>
<td>$0.5L$ or $0.5S$</td>
</tr>
<tr>
<td>Combination 5</td>
<td>$1.0D + 1.0E$</td>
<td>$0.5L + 0.25S$</td>
</tr>
</tbody>
</table>
3.2.5 Results

This section presents the results of the linear analysis performed on a thirty story HWC and a conventional concrete numerical model and compares the structural response parameters namely, modal response, lateral story drift, inter-story drift and base shear.

3.2.5.1 Modal Results

Modal analysis is a linear dynamic procedure which evaluates and superimposes free vibration mode shapes to characterize displacement patterns. This is critical for the preliminary understanding of the structural response to a dynamic loading and assessing the frequency/time period of the structure. The period and the translational direction of both the buildings for the first three modes are shown in Table 9. The rationale behind studying three modes of vibration is because of its high mass participation factor in the first three modes. Mass participation factor is a measure of the energy contained within each mode which gives a representation of how much of the part of the structure is contributing during seismic excitation. The typical requirement of mass participation factor in the modal analysis is greater than 60% in the first mode and at least 90% for all the modes combined (RISA Technologies, 2017). As the first three modes had a higher time period and mass participation factor greater than 60%, these structures were most likely to show the translation behavior during seismic excitation as shown in Table 9.

Table 9: Modal response of the HWC and concrete numerical models.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>HWC building</th>
<th>Concrete building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modes</td>
<td>1 2 3</td>
<td>1 2 3</td>
</tr>
<tr>
<td>Period (seconds)</td>
<td>2.67 2.63 2.25</td>
<td>2.75 2.69 2.30</td>
</tr>
<tr>
<td>Direction</td>
<td>Y-axis X-Axis Torsional</td>
<td>Y-axis X-Axis Torsional</td>
</tr>
</tbody>
</table>
Since the buildings were symmetrical along the X and Y directions, a linear translation along Y-axis was observed in both the buildings in the first mode, followed by X-axis translational movement. The third mode of the structure depicted a torsional mode because of the presence of the symmetrical core walls at the center of the buildings. The torsional mode is crucial for any building as this mode can induce higher displacement causing failure of the structure. Thus, torsion must be accounted for, in the fundamental mode by including symmetry in the structure or by providing more stiffness in the torsional mode. As period is an inverse function of stiffness, it can also be observed that the HWC system was more flexible than a conventional concrete structure which made it more effective during dynamic loading.

3.2.5.2 Lateral Deformation and Inter-Story Drift

The lateral deformation of a structure is the relative drift of the stories with respect to the base, while inter-story drift refers to the relative drift of the stories with respect to its consecutive story. Article 4.1.8.13 of NBCC 2010 has set the maximum deflection limits of buildings with normal importance category as 0.025 $h_s$, where $h_s$ refers to the story height. The maximum allowable lateral deformation for the 30-story buildings is 2350mm whereas the maximum allowable per-story deformation is 72.5mm. Figure 27 and Figure 28 show the lateral deformations of both the buildings in X and Y directions, respectively, while Figure 29 and Figure 30 show the inter-story drift of both buildings in X and Y directions, respectively. The analysis was done with the most critical load combination including seismic motion. It can be observed that since the structure was almost symmetrical with the additional shear wall in the X-direction, the lateral deformations and the inter-story drift was less in X direction compared to Y-direction. Also, both the HWC and the conventional concrete structure were well within the prescribed limits of drift as per NBCC. The
HWC structure, however, was more flexible than the concrete structure and experienced higher drifts in all the directions.

Figure 27: Lateral deformation envelope of the HWC and concrete models in Y-direction.

Figure 28: Lateral deformation envelope of the HWC and concrete models in X-direction.
3.2.5.3 Base Shear

Base shear is an estimate of the maximum expected lateral force that occurs due to seismic motion at the base of the building which is directly proportional to the mass and stiffness of the structure. Base shear was calculated for both the numerical models as per equivalent static force method. The results, shown in Figure 31 and Figure 32, are for the most critical load combination involving earthquake load in both Y and X direction. As base shear is a function of mass and stiffness, the
concrete structure showed higher base shear compared to the HWC system. The HWC system had an approximate 22% and 15% less base shear compared to the concrete model in Y and X direction respectively, owing to lighter weight. The base shear in the Y direction was higher than in X direction, because of the presence of the additional shear wall in the Y direction, making it stiffer. The models were designed based on the maximum base shear experienced by both the models in two orthogonal directions and was used as the reference value for subsequent linear time history analysis. Figure 31 and Figure 32 shows the advantage of using HWC because of its light weight and flexibility. As less seismic force was attracted to the HWC system, the demand on the structural members was comparatively smaller compared to the concrete structure, making it more economical.

![Base shear comparison](image)

**Figure 31: Base shear of the HWC and concrete models in Y-direction.**

![Base shear comparison](image)

**Figure 32: Base shear of the HWC and concrete models in X-direction.**
3.2.6 Linear Seismic Time History Analysis

Article 4.1.8.7 of NBCC 2010 states that: “if the structure is more than 60m high and time period greater than two seconds in either of the two orthogonal directions, the dynamic analysis must be carried out to assess the response corresponding to dynamic loading”. Dynamic analysis can be done using either response spectrum analyses or time history analyses (linear or nonlinear). In the response spectrum method, based on the design spectrum of the structure’s location, the peak values of modal contribution are calculated, and the structure’s response is evaluated based on the combination of all the modal responses (CSI, 2012). Linear time history analysis calculates the solution to the dynamic equilibrium equation (shown in Equation 5) for the structural behavior at an arbitrary time using the dynamic properties of structure (Chopra, 1995):

\[ M \ddot{u}(t) + C \dot{u}(t) + Ku(t) = R(t) \]  

(5)

Where \( M, C, K \) are the mass, damping and stiffness matrices, respectively, and \( R(t) \) is the external load vector.

Earthquake loading (time history) is taken into consideration as a natural or artificially generated ground motion on the structural model. Then quantities of interest (displacement, base shear etc.) are recorded as response parameters as the ground motion propagates along time throughout the structure. Figure 33 shows a typical time history function of an unscaled natural ground motion. In this analysis, linear seismic time history analyses were conducted on the HWC system, to get a better structural estimate in terms of inter-story drift and base shear subjected to natural ground motions with a time interval of 0.01.
3.2.6.1 Ground Motion Selection

For time history analyses, either natural or synthetic ground motions needed to be selected. As per ASCE 07 (2013), a minimum of three ground motions must be selected for both orthogonal directions. But NBCC 2010 recommends a minimum of seven ground motions. Herein, ten suites of ground motions were selected as per FEMA P695 (2009) in two orthogonal directions to give a broader range of results. FEMA P695 provides a set of guidelines for ground motions selection. The motions selected were far-field ground motions from sites located greater than ten km from fault rupture. Also, the ground motions must have a magnitude greater than 6.5, PGA greater than 0.2 g and PGV greater than 15 cm/sec. Based on all criteria and their agreement with the Vancouver UHS, 2% in 50 years, 5% damped (shown in Figure 26), ten ground motions were selected from the PEER strong motion database (Chiou et al., 2008), as summarized in Table 10.
Table 10: Ground motion general information.

<table>
<thead>
<tr>
<th>#</th>
<th>Magnitude</th>
<th>Name</th>
<th>Distance (km)</th>
<th>Type</th>
<th>PGA (g)</th>
<th>$V_s$ 30m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.6</td>
<td>Guerrero</td>
<td>54</td>
<td>Reverse</td>
<td>0.20</td>
<td>425</td>
</tr>
<tr>
<td>2</td>
<td>7.2</td>
<td>Miyagi-Oki</td>
<td>86</td>
<td>Thrust</td>
<td>0.23</td>
<td>730</td>
</tr>
<tr>
<td>3</td>
<td>6.8</td>
<td>Nisqually</td>
<td>20</td>
<td>Strike-slip</td>
<td>0.21</td>
<td>416</td>
</tr>
<tr>
<td>4</td>
<td>8.0</td>
<td>Hokkaido</td>
<td>146</td>
<td>Thrust</td>
<td>0.20</td>
<td>457</td>
</tr>
<tr>
<td>5</td>
<td>8.8</td>
<td>Maule</td>
<td>168</td>
<td>Thrust</td>
<td>0.41</td>
<td>537</td>
</tr>
<tr>
<td>6</td>
<td>9.0</td>
<td>Tohuki</td>
<td>196</td>
<td>Thrust</td>
<td>0.35</td>
<td>580</td>
</tr>
<tr>
<td>7</td>
<td>6.6</td>
<td>San Fernando</td>
<td>19</td>
<td>Reverse</td>
<td>0.25</td>
<td>450</td>
</tr>
<tr>
<td>8</td>
<td>6.5</td>
<td>Fruili</td>
<td>49</td>
<td>Reverse</td>
<td>0.28</td>
<td>496</td>
</tr>
<tr>
<td>9</td>
<td>6.5</td>
<td>Imperial Valley</td>
<td>15</td>
<td>Strike-slip</td>
<td>0.35</td>
<td>471</td>
</tr>
<tr>
<td>10</td>
<td>6.9</td>
<td>Loma Prierta</td>
<td>42</td>
<td>Strike-slip</td>
<td>0.46</td>
<td>391</td>
</tr>
</tbody>
</table>

3.2.6.2 Ground Motion Scaling

After selecting the ground motions in accordance with the Vancouver UHS, the ground motions had to be scaled to match the target spectrum, to appropriately reflect the hazard level. There are several methods for scaling ground motions, primarily ‘linear scaling’ and ‘spectral matching’ are used. In the linear scaling method, the ground motions are normalized by their respective PGV and the normalized motions are collectively scaled to a specific ground motion intensity such that the median spectral acceleration of the records matches the fundamental period of the structure (FEMA, 2009). Spectral matching is a more sophisticated method, where the frequency content of the selected ground motions is modified in either time or frequency domain to match the target spectrum. The scaling method used herein for time history analysis was spectral matching, conducted in Seismomatch (Seismosoft, 2016). Figure 34 shows the spectral matching of ten ground motions matched to Vancouver UHS.
3.2.6.3 Linear Time History Analyses

After the ground motions were scaled to Vancouver UHS, the time history functions were analyzed using SAP 2000, V18. The ground motions were applied in both the X and Y directions of the numerical model with a time interval of 0.01. The solution algorithm used in the analysis is Direct Integration type in which the equilibrium equations of motion are fully integrated as the structure is subjected to dynamic loading. Also, Hilber-Hughes-Taylor time integration parameters were used for stability conditions of the equations of motion. Damping, which is a primary criterion for time history analysis can be defined as the effect which reduces the oscillation of the system and related to energy absorption of the system because of elastic deformation of the elements. Damping ratios can vary from 1% up to 10% in the seismic resistant system. Xiong et al. (2008) studied the effect of damping ratios on a concrete-timber hybrid system and suggested 5% damping; hence in this analysis, 5% damping ratio had been considered. Based on these criteria, time history analyses were performed and the structural responses were recorded with respect to lateral deformation, inter-story drift and base shear in both the orthogonal directions.
3.2.6.4 Lateral Deformation and Inter-Story Drift

As described in Section 3.2.5.2, the model was designed for maximum allowable lateral deformation and per story deformation of 2350mm and 72.5mm, respectively. The response of the time history analyses in terms of lateral deformation and inter-story drift for both directions are shown in Figure 35, Figure 36, Figure 37 and Figure 38, respectively.

The short duration ground motions (San Fernando, Fruili, Imperial Valley and Loma Prierta) showed considerably less lateral and inter-story drift, well within 380mm (lateral deformation) and 45mm (per story deformation), which was within building code provisions. The medium duration ground motions (Guerrero, Miagi Oki, and Nisquali) showed higher drift. In fact, the per story deformation for the Nisquali motion was above the prescribed limit of 72.5mm in both X and Y direction. Long duration earthquake (Tohuku, Maule, and Hokkaido) recorded almost similar drift closer to 600mm (lateral deformation) and 70mm (per story deformation). The Tohuku motion exceeded the prescribed limit of 72.5mm per story deformation in both the orthogonal directions. From these results, it can be observed that though eight out of ten ground motions passed the code provisions, the HWC system was susceptible to long duration ground motions.

![Figure 35: Lateral deformation envelope of the HWC models in X-direction.](image)
Figure 36: Lateral deformation envelope of the HWC model in Y-direction.

Figure 37: Inter-story drift envelope of the HWC model in Y-direction.

Figure 38: Inter-story drift envelope of the HWC model in X-direction.
3.2.6.5 Base Shear

The maximum base shear expected for the HWC was 18,500kN as calculated by the Equivalent static force method in Y-direction; the HWC was designed based on that demand. Figure 39 shows the base shear recorded by the suites of ground motion during time history analyses. It was observed that two of the ten ground motions, namely, Nisquali and Guerrero, which are medium duration, exceeded the design limit of base shear. The short and the long duration ground motions were less sensitive to the base shear in this analysis and did not exceed the design value.

![Base Shear of the hybrid structure in X and Y direction](image)

*Figure 39: Base shear of the hybrid structure in X and Y direction.*

3.3 Discussion

Based on the equivalent static force method analysis and linear time history analysis the structural feasibility of the HWC system was validated according to NBCC 2010. The HWC system was flexible compared to a conventional concrete building and attracted less seismic forces reducing the demand on the structural members. Also, it conformed to the NBCC inter-story drift limit of 2.5%; however, the drift was larger than in the concrete model. Out of ten ground motions, eight
resulted in recording the drift and base shear to be within the design limit, which according to the building code was acceptable considering the wide spectrum of ground motions used. Also, it can be deduced that the HWC system was more sensitive to long duration ground motions and hence needed to be designed accordingly, where subduction zones are prevalent.

3.4 Further Investigation

Though linear analyses allow evaluating the feasibility of the HWC system, they do not include the structure’s behavior in the nonlinear domain. In other words, the post yielding behavior of the structural elements is unknown. Moreover, since the hybrid system is a combination of more than one material, the load sharing mechanism between concrete frame and timber shear walls was not investigated, which resulted in conservative modeling approach for such systems. Hence these issues need to be further investigated in a nonlinear parametric study in Chapter 4 of this thesis.
4.1 Overview

The preceding linear analyses evaluated the structural feasibility of the HWC system without considering material or geometric nonlinearity. For the structural assessment of a novel system, the linear analysis must be followed by nonlinear static and dynamic analysis in both the component and system level, to assess demand over capacity and post yielding behavior of the system. In this chapter, a component level study was conducted considering a single-bay single-story concrete moment frame with infill timber shear wall allowing for material nonlinearity to investigate:

1) The capacity of the concrete moment frame with and without the timber shear wall.

2) Failure mechanism of the elements when subjected to monotonic loading.

3) The lateral load sharing mechanism between concrete moment frame and timber shear wall.

4) Effect of parameters like aspect ratio, the spacing of nails and bolts, timber sheathings, timber shear wall openings on the lateral capacity of the portal frame and its ductility.

5) The dynamic analysis of the hybrid portal frame (HPF) to assess the demand on the system and assign a performance criterion.

Based on the results of the component level investigation, two-dimensional, system numerical models were developed, considering material and geometric nonlinearity. The models were evaluated with variation in height and infill wall configuration and subjected to monotonic and dynamic loading in terms of inter-story drift and base shear. The analysis gave a preliminary understanding of designing HWC system and its feasibility considering static and dynamic loads.
4.2 Concept of the HPF

The lateral performance of steel moment frame with the light wood shear wall has been studied by Li et al. (2015) but the performance of concrete moment frames with wood infill walls has not been evaluated. Since the building proposed in this thesis is a concrete-timber hybrid system, the component level study of concrete moment frame with infill timber walls is crucial for preliminary investigation and subsequent numerical efficiency. The HPF system studied in this section comprised of concrete elements (beam and columns) and infill timber shear walls connected using M14 bolts. The height and width of the portal frame were 2.44m x 2.44m respectively. The HPF is a two-dimensional model without accounting for any out-of-plane behavior. All the elements were modeled considering material nonlinearity and calibrated with experimental results in SAP 2000. A typical concept of HPF system is illustrated in Figure 40.

![Figure 40: A hybrid portal frame with concrete moment frame and infill wood shear wall](image)
4.3 Numerical Model Development

4.3.1 Timber Shear Wall

Timber shear walls are a part of the subsystem of the HWC system, which takes part in resisting the lateral force. These walls are the first line of defense in a seismic event and are designed to yield. The main components of the timber shear wall are the rigid sheathings, elastic studs, and ductile nail connections. The nonlinearity in the timber shear walls arises from the nails which connect the studs with the sheathings and provide ductility to the system (Loo & Chouw, 2012).

In this study, the elements of the timber shear wall were developed as an extension of the model developed by Giovanni et al. (2013), accounting for stiffness and strength degradation, post-peak softening and pinching force of the shear walls. The methodology involved detailed modeling of the nail connections based on the experiments conducted by Dinehart et al. (2006), followed by elastic models of frames and sheathings. The model was then calibrated with the hysteretic results based on the experiment conducted on a 2.44m x 2.44m timber shear wall (Dolan, 1989).

Links which can capture the nonlinear behavior of the nails were used to represent the frame to sheathing connections. Multi-linear plastic links in SAP 2000 were used for modeling the links to capture the load-slip behavior of the nails in two mutually perpendicular directions, including the stiffness degradation and the hysteretic behavior. Multi-linear plastic links were used to connect two nodes which consisted of six springs, where three of them $U_1$, $U_2$ and $U_3$, represented translations and other three $R_1$, $R_2$ and $R_3$ represented rotation. For the nail connection, only the $U_2$ and $U_3$ directions were considered as they were related to shear. The force-deformation or moment-rotation relationship was used as input in the multi-linear plastic links. To replicate and calibrate the nonlinear hysteretic behavior of the nails, pivot hysteretic models were adopted. Pivot model was adopted from the library of SAP 2000, because of its ability to capture the pinching behavior.
of timber and stiffness degradation. The pivot model directs both the unloading and reloading towards predefined pivot points. The pivot points control the hysteresis behavior as shown in Figure 41. The parameters which define the hysteretic behavior are $\alpha_1$ and $\alpha_2$. It defines the pivot points for the unloading from the positive part of the backbone curve and the negative part respectively and $\beta_1$ and $\beta_2$, which defines the pivot points for the reloading towards a positive force and a negative force respectively.

![Diagram of pivot hysteresis model](image)

*Figure 41: Pivot hysteresis model. Adapted from: Dowell et al. (1998)*

The pivot point $P_1$ is located at the intersection of the projected line of the first linear portion of the force-displacement relationship and the horizontal line through $\alpha F_y$ and $P_2$ is located at the intersection of a horizontal line through $\beta F_y$ and the elastic portion of the force-displacement curve as shown in Figure 42. For the nail connections, the load-displacement relationship that is the same for both the positive and negative displacement, the absolute values of both $P_1$ and $P_2$, for both positive and negative force is the same. The parameters $\alpha$ and $\beta$ can be calculated (as per Equations 6 and 7) if an empirically determined stiffness $K_3$ is used for unloading at the ultimate force-displacement values as shown in Figure 42.
Figure 42: Assessing hysteretic parameters $\alpha$ and $\beta$. Adapted from: Loo & Chouw (2012).

\[
\alpha = \frac{K_3 \delta_{ult} - F_{ult}}{F_y - K_3 \delta_y} \quad (6)
\]

\[
\beta = \frac{F_1}{F_y}, \ 0 < F_1 < F_y \quad (7)
\]

Where $K_3$ is an empirically determined stiffness, $F_y$ and $F_{ult}$ are the yield force and ultimate force, respectively, $\delta_y$ and $\delta_{ult}$ are the yield and ultimate displacement, respectively, and $F_1$ is the force within the elastic limit less than yield force. The hysteresis parameters were calculated based on the force-displacement relationship of the experiments performed by Dinehart et al. (2006) (shown in Figure 43) with 3mm nails attaching 11mm thick OSB sheathing to SPF framing. The values of $\alpha$ and $\beta$ were calculated to be 64.8 and 0.91 based on Equation 6 and 7. For directions $U_2$ and $U_3$, the force-displacement curve shown in Figure 43 was entered to simulate the behavior of the nails in the light wood frame shear walls.
Nonlinear modeling of the nails was followed by numerical modeling of the 2.44m x 2.44m shear wall. The components of the shear wall (sheathing and frame elements) were assumed to be elastic. Beam elements in SAP 2000, which account for biaxial bending, torsion, and axial deformation were used to model the timber studs and the top and bottom plate of the shear wall. The frames were modeled as pin connections. Sheathings were modeled as shell elements, considering only membrane action. The FEA model is shown in Figure 44; it should be noted that the numerical model did not account for the out-of-plane behavior. This model was calibrated with the experimental results conducted on a 2.44m x 2.44m shear wall by (Dolan, 1989). The model was subjected to cyclic loading to record the hysteretic properties of the timber shear wall, and calibrated with experiments as shown in Figure 45. The calibrated results of the timber shear wall (force-deformation) were used as input parameter for the HPF. The parameters were assigned in the multi-linear-plastic link elements as single-degree-of-freedom elements. The links were connected to the concrete moment frame via connections as shown in Figure 49 to simulate the nonlinear behavior of the walls.

Figure 43: Experimental results of force-deformation relationship for 3mm nails. Adapted from: Dinehart et al. (2006).
4.3.2 Concrete Moment Frame

The concrete moment frame in the portal frame is composed of the beam and columns. This main structure in the HWC system is the second line of defense during a seismic event and hence
nonlinear modeling of the beams and columns is required. Frame models to represent nonlinear behavior in concrete sections can either be represented as concentrated/lumped plasticity model or distributed plasticity model as shown in Figure 46. In the concentrated plasticity model, the nonlinear behavior is assumed in the extremities of the element as either hinge or springs, while the body is modeled as an elastic element (Taucer et al., 1991). The primary advantage of the concentrated plasticity model is that it reduces computational time, but precise results cannot be obtained because of its inability to capture nonlinear characteristics throughout the frame element. On the other hand, a numerical model based on distributed plasticity exhibits nonlinearity at any element section which makes it a more accurate modeling compared to concentrated plasticity model approach (Rahai & Nafari, 2013).

![Figure 46: Types of models of frame element (Deierlein et al., 2010)](image)

In the numerical model of the HPF, lumped plasticity model was used to define the hinges of the beam and the column to reduce the computation time. The hinges at the extremities of the beam and column were modeled using ASCE 41-13 (Structural Engineering Institute, 2014). Generalized force-deformation relationships of building components and acceptance criteria for
deformation-controlled actions in any of the materials are defined in ASCE 41-13, see Figure 47. A linear response is depicted between point A and an effective yield point B. The slope from B to C is typically a small percentage (0-10%) of the elastic slope and is included to represent strain hardening. C represents the ultimate force and deformation of the material after which significant degradation occurs (CD). Beyond point D, the component responds with the substantially reduced strength to point E. After point E, the strength of the component is 0. ASCE 41-13 (Structural Engineering Institute, 2014) also defines the acceptance criteria for deformation or deformation ratios for the structural elements, corresponding to building performance levels of Collapse Prevention (CP), Life Safety (LS) and Immediate Occupancy (IO). The curve parameters a, b and c for various elements are defined in ASCE 41-13. These values are calculated based on the cross-section of the elements, axial force, and reinforcement ratio.

Figure 47: Generalized force-deformation relation for concrete elements. Adapted from: ASCE 41-13 (Structural Engineering Institute, 2014)

SAP 2000 allows assigning auto hinges to the frame elements which incorporate the values based on the element sections. In this study, hinges were assigned at both the ends of the frame element. In case of the beam, moment \( M_3 \) was assigned while in columns, axial force (\( P \)) and moment (\( M_2 \) and \( M_3 \)) were assigned to the hinges. Table 11 shows the curve parameter (moment-rotation) used to define the hinges of the beam and columns of the HPF system.
Table 11: Moment-rotation values for concrete element hinges

<table>
<thead>
<tr>
<th>Element</th>
<th>Moment (kNmm)</th>
<th>Rotation (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Beam</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Column</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

4.3.3 Connections

The methodology involved in modeling the connections followed the steps described in Section 3.2.2.3 and incorporated material nonlinearity. Component level tests of M14 bolts were conducted at Tongji University and the results are summarized in Section 2.4. From the experiments, the configurations for M14 bolt available and type of failure observed are shown in Table 12. It is noted that connections were designed to keep the main structure (concrete) and substructure (timber) connected, hence shear failure of the bolts during a seismic event was not desired.

Table 12: Failure mechanism of the M14 bolt.

<table>
<thead>
<tr>
<th>No.</th>
<th>SPF thickness (mm)</th>
<th>Steel plate</th>
<th>Failure type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>76</td>
<td>Yes</td>
<td>Hinge formation in bolt</td>
</tr>
<tr>
<td>2</td>
<td>76</td>
<td>No</td>
<td>Shear failure of bolt</td>
</tr>
<tr>
<td>3</td>
<td>114</td>
<td>Yes</td>
<td>Shear failure of bolt</td>
</tr>
<tr>
<td>4</td>
<td>114</td>
<td>No</td>
<td>Shear failure of bolt</td>
</tr>
</tbody>
</table>

From Table 12, it can be inferred that the configuration failing by hinge formation of the bolts was the M14 bolt connected to 76mm SPF with the plate. Hence this configuration was used to model the connection for the HPF system. The force-deformation curve of the configuration was idealized as shown in Figure 48. The connections were modeled as nonlinear link elements in SAP 2000 and the force-deformation parameters from Figure 48 were used as input for the multi-linear-link
elements in two-degrees-of-freedom accounting shear. The links connected the concrete moment frame and light wood frame shear wall, with 400mm spacing. The FEA model of the HPF system with all the nonlinear elements is illustrated in Figure 49.

![Figure 48: Force-deformation curve of the M14 bolt.](image)

![Figure 49: Numerical model of the HPF system](image)

### 4.3.4 Material Properties and Frame Sections

A concrete grade of 4000psi (27MPa) was used for the concrete frame members. For the infill timber shear walls, kiln-dried No. 2 Grade Spruce-Pine-Fir (SPF) 38mm x 140mm dimension
lumber was used. The sheathing material was OSB panels, 19/32 (APA panel grade), 1220mm x 2440mm in-plane size and 11mm in thickness. The panels were attached to the frame with 3mm diameter nails spaced at 150mm. The bolts used to connect the concrete moment frame with the timber shear walls were 4.8 graded M14 bolts.

4.4 Results of Nonlinear Static Analysis

To analyze the capacity of a structure in the inelastic zone, nonlinear static analysis (pushover analysis) is an appropriate and powerful tool. The structure is subjected to monotonic displacement-controlled lateral load which continuously increases until an ultimate condition is reached. The lateral load represents the range of base shear-induced during an earthquake loading and the output generates a plot of strength against deflection of the structure. Nonlinear static analysis gives an insight into the ductile capacity of the structural system, and indicates where the failure mechanism of the system occurs.

To assess the HPF system, pushover analysis was performed, considering geometric nonlinearity ($P-\Delta$ effect). The numerical model was analyzed using displacement-based criteria, where the frame is monotonically loaded to 4% drift, which is 97.6mm. A pushover load of 1kN was applied on the top left node of the frame and the strength-displacement parameters were recorded as base shear in X-direction and top node displacement.

4.4.1 Capacity and Yielding Mechanism of Hybrid Frame

Pushover analysis was performed on the HPF and a bare frame without infill shear wall. Figure 50 shows the pushover curve comparing both the models. The yield strength and the ultimate strength of the HPF system showed increases of 22% and 36.7%, respectively, compared to the bare
concrete moment frame. The elastic stiffness of the frames did not vary significantly, but the HPF showed an increase in stiffness from yield to ultimate strength limit.

![Graph showing capacities of concrete moment frame and HPF system.](image)

*Figure 50: Capacities of concrete moment frame and HPF system.*

Figure 51 shows the yielding mechanism of the HPF and the first yield of various elements based on the material model. The nails of the timber shear walls yielded first, at 0.35% drift, resulting in a slight decrease in system stiffness. Then, the concrete beam yielded followed by the column. The stiffness of the structure decreased significantly with the beam yielding, and the HPF reached its yielding point when the column yielded. Hinges in the connections were developed at 1.85% drift. The bolts were designed to keep the substructure and main structure intact, hence the yield of the bolts was at higher drift.
4.4.2 Load Sharing Mechanism between Concrete Frame and Timber Shear Wall

In the HPF system, the lateral force is resisted simultaneously by the concrete moment frame and the timber shear wall. The lateral force resisted by each system by monotonically loading the HPF and the percentage of lateral force on each of the system is shown in Figure 52.

![Diagram showing yield of the elements in HPF system](image)

Figure 51: Yield of the elements in HPF system.

![Diagram showing percentage of lateral force shared between concrete moment frame and timber shear wall](image)

Figure 52: Percentage of lateral force shared between concrete moment frame and timber shear wall

The infill timber shear walls resisted most of the lateral force during the initial loading up to 0.2% drift. As the loading progressed, the nails of the timber shear walls yielded, with more damage
observed in the walls, resulting in a decrease in lateral load capacity of the walls. By 1.23% drift, the percentage of lateral capacity of the timber shear walls decreased to 40% and ultimately below 10% as the HPF reached its ultimate strength. The concrete moment frame, on the other hand, got activated with a decrease in stiffness of the timber shear walls. About 50% of the lateral force was resisted by the concrete moment frame at 0.33% drift, the value increased to 80% by 1.65% drift. The concrete moment frame continued to resist lateral force even after the timber shear wall reached its ultimate strength. It is noted that the post-yield lateral resistance of the HPF system mainly came from the deformation of the concrete moment frame.

4.5 Parametric Study

On the one-story one-bay HPF system, the effect of the aspect ratio, nail spacing and number of sheathing in the timber shear walls on the lateral capacity was studied. To assess the effect of the variables, pushover analysis was conducted on 13 different configurations. The backbone curve (Appendix B) of the analysis was used as an input parameter for multi-linear-links in SAP 2000, representing timber shear walls. The links connected the concrete moment frame, idealizing the HPF system stiffness, and then peak load and ductility were calculated. The results are summarized in Table 13. Where \( SH \) represents the number of sheathings, \( K_{bf} \) and \( K_{hybrid} \) are the secant stiffness between 0% to 40% of the ultimate lateral load resisted by the bare frame and HPF respectively, in kN/mm. \( P_{peak} \) and \( P_y \) are the ultimate lateral load and yield load resisted by HPF, in kN. \( \Delta_y \) is the yield displacement of the HPF which is \( P_y/K_{hybrid} \) in mm. \( \Delta_u \) is the ultimate displacement of HPF at 2.5% drift, in mm and \( \mu \) is the ductility of the system, \( \Delta_u/\Delta_y \).
Table 13: Summary of the HPF characteristics

<table>
<thead>
<tr>
<th>No</th>
<th>$H$ (mm)</th>
<th>$L$ (mm)</th>
<th>SH</th>
<th>Spacing (mm)</th>
<th>$K_{bf}$</th>
<th>$K_{hybrid}$</th>
<th>$P_{peak}$</th>
<th>$P_y$</th>
<th>$\Delta_y$</th>
<th>$\Delta_u$</th>
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<td>7200</td>
<td>1</td>
<td>150</td>
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<td>79</td>
<td>25.5</td>
<td>105</td>
<td>4.1</td>
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4.5.1 Effect of Lateral Capacity of HPF with Variation in Aspect Ratio

The aspect ratio of the walls affects the lateral capacity of the HPF as seen in Table 13. Figure 53 shows the lateral capacity of the HPF system with variation in its aspect ratio, from 0.375 to 1.5. Walls having a lower aspect ratio had a higher lateral load capacity compared to slender walls. Walls having an aspect ratio of 1, were approximately 60% higher in ultimate load capacities compared to the walls having an aspect ratio 1.5. A significant increase in capacity was observed between the walls having an aspect ratio of 0.75 and 1. Walls having an aspect ratio of 0.375 resulted about 300% increase in capacity compared to walls with 1.5 aspect ratio. For higher walls, same lateral load induced higher uplifting forces and overturning moment compared to lower walls and hence this trend was observed. The lateral capacity increased with increase in length, resulting in a higher capacity of long walls compared to slender walls.
4.5.2 Effect of Stiffness Degradation in HPF with Variation in Sheathings

Peak load and ductility of the HPF system increased with an increase in the number of sheathings of the timber shear walls. It is noted that keeping the aspect ratio constant, the ultimate load resisted by the walls increased up to 51% when double sheathing was used. Figure 54 shows the stiffness degradation of the HWC system using single and double sheathings. At the start of the monotonic loading, within 0.32% drift, the double sheathed shear wall showed 14.3% more stiffness than the single sheathed shear wall. As the loading progressed, at 1% drift, the stiffness of both the systems coincided, suggesting after a drift, both the systems behaves the same way, as substantial degradation happens in both the systems which reduces the lateral load capacity of the walls. Hence the number of sheathing is effective during initial loading conditions.

*Figure 53: Capacity variation in HPF with a change in aspect ratio.*
4.5.3 Effect of Lateral Capacity of HPF with Variation in Nail Spacing

Nails in the timber shear walls are the source of its ductility, yielding under lateral loading, dissipating energy. The spacing of the nails is crucial to attaining the desired post yielding strength and energy dissipation in the HPF system. Timber shear wall configuration 5 was analyzed with nail spacing of 150mm, 300mm, and 450mm. Pushover analysis was conducted to assess the lateral capacity considering the spacing variation, see Figure 55. The yield point and the post-yield behavior of the system were proportional to the increase in spacing of nails. A decrease of 19.5% and 20% strength was observed when the spacing was increased from 150mm to 300mm and 300mm to 450mm, respectively. The stiffness, on the other hand, was higher with 150mm spacing in the elastic limit compared to 450mm spacing but showed proportional behavior in the post yielding zone.

Figure 54: Stiffness degradation in HPF with change in number of sheathings
4.5.4 Effect of Lateral Capacity of HPF with Variation in Bolt Spacing

The bolted connections between the concrete moment frame and the timber shear walls transfer shear forces between the two systems. The spacing of the bolts play an important role in this transfer mechanism. Figure 56 shows the pushover analysis results of configuration 1 with various bolts spacing (125mm, 300mm, 600 mm and 750mm) to investigate the capacity of the HWC system. The shear force resisted by the hybrid walls dropped significantly when the bolt spacing increased. A decrease of 33% was observed when the bolts spacing increased from 300mm to 125mm. A similar trend was noticed when the spacing increased to 600mm. But it is to be noted that, with higher bolt spacing, the lateral capacity of the hybrid walls remained unchanged as seen in Figure 56 for 600mm and 750mm spacing. This is because of the fact that with large spacing, the shear force was not adequately transferred between the two systems. To ensure the effectiveness of the infill walls, the connections must be adequately spaced.

Figure 55: Capacity variation in HPF with change in nail spacing
4.5.5 Effect of Stiffness Degradation in HPF with Variation in Openings

Infill timber shear walls, which act as the initial energy dissipation elements in the HPF system, need to be accurately modeled. In conventional buildings, a significant part of the wall may have openings for functional reasons, such as doors or windows. Openings reduce the stiffness of the structure while concentrating stresses around the corner of the walls. Hence, the shear walls must be modeled considering the openings, otherwise, strength and stiffness are overestimated. FEMA P-807 (ATC, 2012) describes the procedure to account for the openings by adjusting the load-drift curve with a reduction factor. Figure 57 shows a typical shear wall with length $L_w$ and height $H$. 

![Figure 56: Capacity variation in HPF with a change in bolt spacing](image-url)
There are two openings with areas $A_1$ and $A_2$. The lengths of the blocked segments are $L_1$, $L_2$, and $L_3$. The adjustment factor is based on the area of openings relative to the area of full height segments. The modification factor, $Q_{\text{open}}$ can be calculated using Equations 8 and 9:

$$Q_{\text{open}} = 0.92 \alpha - 0.72 \alpha^2 + 0.80 \alpha^3$$

$$\alpha = \frac{1}{1 + \frac{\sum A_i}{H \sum L_i}}$$

Where $\sum A_i$ is the sum of the area of the openings, $H$ is the maximum floor-to-ceiling wall height and $\sum L_i$ is the sum of the length of the full height wall segments.

The force-deformation curve of the timber shear walls was adjusted with the reduction factor and stiffness degradation of the HWC system, considering a variety of openings from 10% to 60%, see Figure 58. Within 0.5% drift (12mm), the difference in stiffness with variation in openings was significant. A 20% decrease in stiffness in the timber shear wall was observed when the opening of the shear was 10%. Similar trend was observed when the opening was increased to 25% and 50% where 20-25% decrease in stiffness was seen. But with increase in opening size to 60%, this
changed where the variation in stiffness led to small decreases in the capacity. As load progressed, between 0.5% to 1% drift, the difference in stiffness degradation with openings showed very low variation. In this region, the shear walls already started degrading, as a result of which, the variation in openings had small effect in capacities. Finally, the shear walls completely degraded at 2% drift, with no stiffness in the shear walls. It is to be noted that after 1% inter-story drift, the variation of openings had no effect in lateral capacities of the timber shear walls.

![Figure 58: Stiffness degradation in HPF with variation in openings.](image)

### 4.6 Discussion

It is noted from Table 13 that infill timber shear walls had a significant effect on the increase of stiffness of the HPF system, varying up to 7 times. Stiffness rose with low aspect ratio (height/length) of the walls rather than slender walls. Also, an increase in sheathings had a proportional increase in stiffness of the HPF as seen in configuration 1 and 2. Ductility of the HPF showed a similar trend with respect to aspect ratio and a number of sheathings. Also, ductility increased with smaller nail spacing. It is important to note that the peak capacity of the timber
shear walls was not proportional with the increase in height or length, which meant the capacities cannot be factored proportionally with shear wall length or height.

The parametric study evaluated various criteria: the aspect ratio of the timber shear walls had a significant impact on the lateral capacity of such system, with higher aspect ratio, capacity decreased. This was due to bigger overturning moments and uplifting forces for slender walls with the same acting lateral load. It was also noted that the capacity of timber shear wall was not directly proportional to the height or length of the wall. Hence, FEA modeling is essential to estimate the capacity of shear walls. The bolt and nail spacing had an inverse relationship with ductility and lateral capacity of the HPF system. The shear force resisted by the timber infill walls dropped as the connection spacing increased. As nails are the source of nonlinearity in the timber shear walls, the spacing played a significant role in ductility of the system. It can be also inferred that, with openings, the stiffness and strength of the timber shear wall decreased, but the value was asymptotic after 50% opening, as the shear wall did not have enough capacity to degrade.
Chapter 5: Nonlinear Dynamic Analysis of Hybrid Building

5.1 Overview

From the static analysis, it was noted that the infill timber shear walls in the HPF system were effective in resisting the lateral loads under monotonic loading. The capacity analysis was followed by a dynamic nonlinear time history analysis to evaluate the demand on the system and the effectiveness of timber shear wall under earthquake load. In this section, nonlinear time history analysis had been performed on the HPF system to assess the structural response when subjected to seismic excitation. In nonlinear dynamic analyses, the numerical model is subjected to ground motion records which are either natural or artificially generated. This process estimates the component deformation for each degree of freedom in the numerical model. In nonlinear dynamic analyses, the nonlinearity of the structure is considered as a part of time-domain analysis, which means the functions considered, vary with time. This method is rigorous but efficient.

Also, for a structure to be considered useful after an earthquake, it has to be serviceable. In case of the HWC system, a performance limit has yet to be defined, owing to its novelty. This section delved into defining a performance criterion in terms of drift limit for the HPF system based on its response to dynamic loading.

An HPF model of 3600mm x 2700mm, single sheathed shear wall, with 150mm nail spacing was considered for dynamic analysis. Three far-field ground motions (Nisquali, Guerrero and Hokkaido as described in Section 3.2.6.1) were considered for the analysis based on short, medium and long duration. All ground motions had a return period of 2475 year with PGA and magnitude greater than 0.2g and 6.5, respectively. The ground motions were scaled based on the UHS of Vancouver and used as input for nonlinear time history analysis.
5.2 Results

5.2.1 Inter-Story Drift

Figure 59 (a, b and c) shows the response of the analysis under Nisquali, Guerrero and Hokkaido ground motions, respectively. The peak drift at the top node of HPF under short duration ground motion was 3mm, while the bare concrete frame had a peak drift of 7mm. The medium duration ground motion recorded peak drift of 3.5mm and 10mm respectively for the HPF and bare concrete frame respectively. In case of the long duration ground motion, the peak drift for HPF was 10mm, while drift in the bare concrete frame was 21mm. Compared to the bare frame, the infill wall showed a significant contribution to reducing the drift under earthquake load. The reduction was more prominent in the short and medium duration earthquake.

Table 14 summarizes the results of the dynamic analysis. It is to be noted that, with the inclusion of the infill timber shear wall, drift was reduced by 57.1%, 65% and 52.3% for short, medium and long duration ground motions respectively. However, the strength and stiffness degradation of the infill timber walls under large drifts made them less effective as the degradation is higher with greater load and duration. This explains higher drift reduction for short and medium duration ground motion rather than long duration motions.

<table>
<thead>
<tr>
<th>Event</th>
<th>Duration</th>
<th>Peak inter-story drift (mm)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bare frame</td>
</tr>
<tr>
<td>Nisquali</td>
<td>Short</td>
<td>7.0</td>
</tr>
<tr>
<td>Guerrero</td>
<td>Medium</td>
<td>10</td>
</tr>
<tr>
<td>Hokkaido</td>
<td>Long</td>
<td>21</td>
</tr>
</tbody>
</table>

*Table 14: Peak inter-story drift from nonlinear dynamic analysis*
Figure 59: Comparison of drift at the top node in bare concrete frame and hybrid system for (a) Nisquali (b) Guerrero and (c) Hokkaido ground motions.
5.2.2 Seismic Demand on HPF System

Seismic demand is the dynamic load which acts on a structure due to a certain intensity of an earthquake related to the type of soil and the characteristics of the ground motion. Seismic demand is an important criteria to assess the feasibility of a structure. In nonlinear dynamic analyses, seismic demand is always assessed first before the capacity of the structure. Seismic capacity is the overall characteristics of a structure to withstand the seismic demand. A structure must withstand the seismic demand within the elastic and inelastic limit without collapsing.

Nonlinear time history analysis is an important tool in assessing the seismic demand on a structure. In order to evaluate seismic demands, the results from monotonic loading (pushover analysis) used to evaluate the capacity of the structure is compared to the results of the nonlinear time history analysis (seismic demand). A certain target (performance criteria) is previously defined in order to assess the structural feasibility. It is to be noted that, in no condition, the seismic demand is greater than the capacity. In such cases, the structure needs to be redesigned and analyzed.

In this section, the seismic demand in terms of base shear on the HPF model is analyzed subjected to the Nisquali, Guerrero and Hokkaido ground motions. Figure 60 shows the base shear demand on the HPF over capacity for the three ground motions along with drift. The peak demand on the HPF was 0.11%, 0.13% and 0.56% for the short, medium and long duration ground motion, respectively. The yield drift limit of the HPF was not reached during the dynamic analysis. However, among the individual elements of the HPF, the timber shear wall yielded at 0.36% drift under the long duration ground motion (shown in Figure 62). Hence, a performance limit needed to be assigned during dynamic loading, to assess the damage of the structural elements in the HWC system which is described in the subsequent sections.
Figure 60: Demand and capacity comparison in the hybrid frame for (a) Nisquali (b) Guerrero and (c) Hokkaido ground motions.
5.3 Performance Criteria

Performance criteria for the HWC system need to be developed to assess the results. Performance criteria comprise of a set of standards that a structural model must conform in order to be classified under a certain performance level (Fairhurst, 2014). These are the discrete damage states selected that a building could experience because of earthquake response.

FEMA-350 (ATC, 2009) states three different damage states to evaluate the performance level of a structure based on the performance of the structure to a specific earthquake hazard level. They are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) states. IO refers to the state when a structure is fully functional after a seismic event with no plastic deformations in the structural elements. LS criteria allow minor damage to the structure, ensuring the safety of the occupants inside. While CP state is the final state where the structure experiences major structural damage, with the elements showing plastic deformation, but without a complete collapse of the structure. Figure 61 illustrates the different stages of performance criteria.

![Figure 61: Performance levels. (ATC, 2009)](image-url)
After the criteria are defined, the performance measures are evaluated for the desired objective in a structure in terms of demand/capacity ratio, inter-story drifts, etc. FEMA-350 (2009) has set various guidelines to evaluate the performance criteria and measures of structural elements. However, for the HWC system being a novel structural system, the usual criterions may not meet the desired expectations. Hence, a performance criterion value for HWC is defined considering inter-story drift as the measure.

As per NBCC (NRC, 2010), the life safety inter-story drift limits is 2.5% for regular structures. This value holds true for already established systems. But, HWC model, being a novel system, this value may not be appropriate in defining performance criteria. In the HWC system, the substructure consist of the infill timber shear wall which is the first line of defense during a seismic event and is expected to yield. Figure 62 illustrates the demand on the timber shear wall, during time history analysis for the long duration ground motion. It is noted that the walls experienced peak load at 0.5% drift. At this level, the timber shear walls started to yield. In case of the main structure, which comprises of the concrete elements, the first yield occurred in the concrete beams, at 2% drift. Since the HWC system was designed, keeping the main structure as the second line of defense, the concrete elements should not yield in the life safety performance criteria. Performance criteria for life safety in timber shear walls have been assessed at 2% drift limit for transient damage by Filiatrault & Folz (2002). Hence, considering both the main and the substructure, a life safety criterion of 2% drift limit was fixed for multistory analysis. This value prevented the concrete elements to yield but allowed strength degradation in the timber shear walls, which acted as the primary energy dissipation units in the HWC system.
5.4 Multistory HWC System Parameter Study

5.4.1 Overview

To study the timber-concrete hybrid structural system, FEA models of 9, 15, and 30 stories were developed and analyzed (in SAP 2000) based on the knowledge of the parametric study of the one-story HPF model. Material and geometric nonlinearities were considered. Equivalent static force method (ESFM) was used to estimate the lateral load on the models following the NBCC 2010 methodology for site class C in Vancouver. The NBCC base shear was calculated using ductility and overstrength factors of 1.5 and 1.3 respectively. The story shear was distributed according to ESFM and the members were designed as per NBCC requirements.

The nonlinear dynamic analysis evaluated the structures in terms of inter-story drift and base shear. The base shear calculated according to NBCC 2010 was compared to the maximum base shear demand from the nonlinear analyses.
5.4.2 Numerical Modeling

All the models were two-dimensional, without considering diaphragm or torsional effects. The two-dimensional models have three bays with 8m, 12m and 8m respectively and height of each story is 2.9m. The geometry, gravity loading, and member specifications followed the same methodology as described in Chapter 3. The infill timber shear walls were configured as shown in Figure 63 for the three models. For the 9-story model, the timber infill shear walls were assigned in the mid bay idealized as spring elements, whereas for the 15 story model, both outer 8m bays were infilled with the shear walls. In the 30-story model, all bays had timber infill walls to attain the requisite stiffness of the stall structure. The concrete elements, timber shear walls, and the connections were modeled like those described in the single-story-single-bay frame. Single sheathing was used for the timber shear walls. The nail and the bolt spacing for the HWC system were assumed to be 150mm and 300mm respectively. Also, the strength and stiffness of the timber shear walls were adjusted for 20% openings in them. A performance criterion of 2% drift was considered for life safety condition, for the concrete main structure to remain elastic.

![Figure 63: 2-D Finite element model of (a) 9 story (b) 15 story and (c) 30 story HWC structure](image)
5.4.3 Numerical Analysis

The models were loaded monotonically to estimate capacity. Dynamic analyses were performed considering the same three ground motions spectrally matched to Vancouver UHS, used to evaluate the performance of the HPF described in Section 5.1. Direct integration time history analysis was conducted considering 5% damping ratio and 0.01 seconds time step. The results were evaluated and analyzed considering the limit of 2% inter-story drift limit as the performance criterion for concrete elements to remain elastic. Also, seismic demands on the numerical models were analyzed, which was done by comparing the force-deformation curves recorded during nonlinear time history analysis and the pushover (capacity) curve evaluated during monotonic loading on the structure. This is done to estimate the capacity over demand values.

5.4.4 Results of Parameter Study

5.4.4.1 Inter-Story Drift

The inter-story drifts recorded for the three numerical models are calculated for the short, medium and long duration ground motions are shown in Figure 64. The inter-story drift results for all ground motions and all models were below 2.5% inter-story drift as suggested by NBCC 2010. The models were also consistently below 2% inter-story drift which was considered as the life safety performance criteria for the HWC system so that the concrete elements do not yield and are within the elastic limit. While the ground motions showed an almost similar response on the 15 and 30 story models, irrespective of duration, for the 9 story model, the medium and long duration ground motions tended to show higher drifts than the short duration ground motion. It is also noted that, for 30 story structures, the drift tended to decrease compared to the 9 and 15 story structures indicating that tall structures are less affected by the ground motions.
Figure 64: Inter-story drift envelope of (a) 9-story (b) 15-story and (c) 30-story model subjected to ground motions
5.4.4.2 Base Shear Demand

Figure 65, Figure 66 and Figure 67 present the base shear demands on the 9, 15, and 30 story HWC models, subjected to short, medium and long duration ground motions. The capacity of the models assessed by pushover analysis was plotted as back-bone curves. Also, the base shear calculated as per NBCC 2010 is shown in Table 15 for comparison.

In all numerical models, the demand on the structures for all the ground motions surpassed the yield limit of timber shear walls, but stayed within the elastic limit of concrete elements and connections. For the 9 story model, the maximum demand on the model was from short duration ground motion followed by medium and long duration. Base shear from all the ground motions showed a Capacity/Demand (C/D) ratio larger than 1. Also, NBCC base shear prediction was higher for two out of three ground motions suggesting conservative design methodology for NBCC approach. For the 15 story model, medium and long duration ground motions showed prominent demand on the structure, while short duration ground motion recorded the minimum. The C/D ratio was the highest for short duration ground motion while other ground motions showed almost consistent demand. NBCC base shear prediction was low comparing to the recorded nonlinear results, with one out of three demand value less than NBCC base shear. The 30 story structure was less responsive to all the ground motions showing higher C/D ratio and two out of three ground motion’s led to demands less than the predicted NBCC base shear. The short duration ground motion recorded the highest demand for this model. For all the numerical models, demand by all the ground motions was less than maximum capacity. Also, it is noted that short duration ground motions were more responsive for low and high-rise buildings, while long duration ground motions showed the maximum responsive for mid-rise structure.
Figure 65: Demand vs capacity of (a) Nisquali (b) Guerrero (c) Hokkaido motions on the 9-story model
Figure 66: Demand vs capacity of (a) Nisquali (b) Guerrero (c) Hokkaido motions on the 15-story model
Figure 67: Demand vs capacity of (a) Nisquali (b) Guerrero (c) Hokkaido motions on the 30-story model
Table 15: Capacity over demand by seismic ground motions on the numerical models

<table>
<thead>
<tr>
<th>Model</th>
<th>Ground Motions</th>
<th>Demand (kN)</th>
<th>Capacity (kN)</th>
<th>C/D</th>
<th>NBCC Base shear (kN)</th>
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</thead>
<tbody>
<tr>
<td>9-story</td>
<td>Nisquali</td>
<td>850</td>
<td>1250</td>
<td>1.47</td>
<td>780</td>
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<tr>
<td></td>
<td>Guerrero</td>
<td>760</td>
<td></td>
<td>1.64</td>
<td></td>
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<td></td>
<td>Hokkaido</td>
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<td>1.83</td>
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</tr>
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<tr>
<td></td>
<td>Guerrero</td>
<td>965</td>
<td></td>
<td>1.47</td>
<td></td>
</tr>
<tr>
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<td>Hokkaido</td>
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<td>1.43</td>
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</tr>
<tr>
<td>30-story</td>
<td>Nisquali</td>
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<td>3000</td>
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<td>1960</td>
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<tr>
<td></td>
<td>Guerrero</td>
<td>1780</td>
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<td>1.69</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Hokkaido</td>
<td>1600</td>
<td></td>
<td>1.88</td>
<td></td>
</tr>
</tbody>
</table>

5.5 Discussion

The nonlinear dynamic analyses on the HPF system showed a significant decrease in inter-story drift due to the activation of the timber infill wall. The performance criteria assigned to the HPF system was used to check the response of the 9, 15 and 30 story models. It was found that demand on the structure by different duration ground motions did not exceed the assigned drift and capacity. Inter-story drift was less in the 30 story model because tall structures tend to be more flexible and less susceptible to seismic excitations. The numerical models responded differently to the ground motions which was due to variability in mass and stiffness of the structures. The short duration ground motions posed high demand on the 9 and 30 story model, while the 15 story model was dominated by medium duration ground motions. These deductions will be helpful in designing HWC system of different height subjected to a target site and hazard level. NBCC base shear values deviated from the nonlinear analysis results, which is due to considering conservative ductility and overstrength factors. Also, the higher mode factor for the inelastic design of structures undermined analysis results which resulted in lower NBCC base shear results.
Chapter 6: Conclusions

6.1 Summary

This thesis discussed a proposal of an innovative structural system for tall concrete-timber hybrid buildings which was conceived as a collaboration between Tongji University and UBC. This discussion was followed by numerical modeling and analyses of the proposed system to understand its structural response when it is subjected to static and dynamic loads. The main objective of this thesis was to determine if the structure is feasible in high seismic zones. The numerical analyses were conducted in the elastic and inelastic domain to assess the system’s feasibility subjected to gravity and seismic demands. Numerical modeling included a parametric study of a two-dimensional portal hybrid frame followed by multi-story modeling.

In the linear analyses, the inter-story drift and base shear of the novel hybrid system were compared to a conventional 30 story concrete model. It was noted that the drift and the base shear of the hybrid model were lower and that the drifts were within the code limits.

The nonlinear parametric analysis resulted in understanding the impact of design parameters such as shear wall aspect ratio, the spacing of nails and bolts, number of sheathing and openings in the walls. The infill timber shear wall decreased the drift of the system to about 65% when compared to a bare concrete moment frame. A performance criterion of 2% inter-story drift was assigned for the novel hybrid system to assess the failure of different structural elements in the models.

A multi-story nonlinear dynamic analyses helped the understanding of the effect of different duration ground motions. It was observed that the 30 story model was least responsive to ground motions because of its high flexibility. Short duration ground motions led to the highest drifts of the 9 story and 30 story models, whereas long duration ground motions had the highest impact on
the 15 story model. The nonlinear dynamic analysis was compared with NBCC calculated base shear. It was seen that, in almost all the cases, the base shear value predicted by NBCC was conservative compared to the nonlinear results. This was because of the fact that no ductility, over strength and higher mode factors are listed for the novel hybrid system in NBCC.

The research presented in this thesis contributed to a preliminary understanding of the novel system’s response and can contribute to developing design guidance for tall hybrid buildings.

6.2 Recommendations for Future Studies

To assess the feasibility of the hybrid structure, component-level tests need to be conducted. These tests should include monotonic and cyclic loading on a one-story one-bay-concrete frames with an infill shear wall, which will enable to calibrate the numerical models and optimize the design. Tests should also be performed at the system level, including dynamic analysis using hybrid simulation to assess the behavior of the system to different types and intensities of ground motion.

The nonlinear dynamic analysis conducted in this thesis should be extended towards more ground motions in each direction to a broader spectrum of understanding of the structure’s behavior to seismic excitation.

Three-dimensional numerical analyses need to be conducted accounting for the diaphragm and torsional effects. Since structures higher than 60m are responsive to wind, a wind linear time history analysis is highly to assess. The damping ratio was assumed to be 5% for the concrete-timber system based on literature review. But further research on the damping ratio for this novel system needs to be conducted.

Since the NBCC base shear design values were conservative, research needs to evaluate the ductility, over strength and higher mode factors, for better design provisions for hybrid structure.
Bibliography

Acton Ostry Architects Inc. (2015). *Brock Commons Phase 1.*


TED Talks. (2013). Michael Green: Why we should build wooden skyscrapers. USA.


Appendix A - Cyclic Loading Results on Single Bolted SPF-Concrete Connections

This appendix illustrates the cyclic loading results on single bolted SPF-Concrete connections conducted at Tongji University, China, as a part of component level testing on the hybrid system.

Figures A1- A8 show the force-displacement curves of M12 and M14 bolts with variation in thickness of SPF lumber and steel plates.

![Figure A1: M12 bolt connected to 76 mm SPF lumber without steel plate.](image)

![Figure A2: M14 bolt connected to 76 mm SPF lumber without steel plate.](image)
**Figure A3**: M12 bolt connected to 76 mm SPF lumber with 60x60x5 mm steel plate.

**Figure A4**: M14 bolt connected to 76 mm SPF lumber with 60x60x5 mm steel plate.

**Figure A5**: M12 bolt connected to 114 mm SPF lumber without steel plate.
Figure A6: M14 bolt connected to 114 mm SPF lumber without steel plate.

Figure A7: M12 bolt connected to 114 mm SPF lumber with 60x60x5 mm steel plate.

Figure A8: M14 bolt connected to 114 mm SPF lumber with 60x60x5 mm steel plate.
Appendix B - Capacity Curves of Timber Shear Walls.

Appendix B shows the capacity curves of timber shear walls of 13 different configurations used in the parametric study of HPF system. Table B1 describes the geometry and property of all the configuration of timber shear wall used in the parametric study. Figure B1 illustrates the capacity curves of the 13 configurations of timber shear walls which were used as backbone curve for analyzing the HPF system.

Table B1: Summary of the timber shear wall characteristics.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>H (mm)</th>
<th>L (mm)</th>
<th>Sheathing</th>
<th>Nail Spacing (mm)</th>
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<tbody>
<tr>
<td>1</td>
<td>2700</td>
<td>3600</td>
<td>1</td>
<td>150</td>
</tr>
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Figure B1: Capacity curves of 13 configurations of timber shear walls.