EXPERIMENTAL INVESTIGATION OF CONNECTION FOR THE FFTT,
A TIMBER-STEEL HYBRID SYSTEM

by
POOJA BHAT

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Abstract

Hybrid systems have grown in popularity over the past years but the lack of established design guidelines has delayed the construction of the hybrid structures. This thesis fills the existing knowledge gap between detailed design and global behaviour of hybrid systems through an experimental study on an innovative timber-steel hybrid system called “FFTT”. The FFTT system relies on wall panels of mass timber such as CLT for gravity and lateral load resistance and embedded steel sections for ductility under the earthquake loads. An important step towards the practical application of the FFTT system is obtaining the proof that the connections facilitate the desired ductile failure mode.

The experimental investigation was carried out at the facility of FPInnovations, Vancouver. The testing program consisted of quasi-static monotonic and reverse cyclic tests on the timber-steel hybrid system with different configurations. The two beam profiles, wide flange I-sections and hollow rectangular sections were tested. The interaction between the steel beams and CLT panels and the effect of the embedment depth, cross-section reduction and embedment length were closely examined.

The study demonstrated that when using an appropriate steel section, the desired ‘Strong Column –Weak Beam’ failure mechanism was initiated and excessive wood crushing was avoided. While wide-flange I-sections were stiffer and stronger, the hollow sections displayed better post-yield behaviour with higher energy dissipation capacity through several cycles of deformation under cyclic loads. The out-of-plane buckling at the point of yielding was the major setback of the embedment of wide-flange I-sections. This research served as a precursor for providing design guidance for the FFTT system as one option for tall wood buildings in high seismic regions.
Preface

This dissertation represents the original, unpublished and independent work conducted by the author, Pooja Bhat under the supervision of Dr. Thomas Tannert and Marjan Popovski.
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Finally, I dedicate my thesis work to my loving parents who have supported me throughout my years of education.
Chapter 1: INTRODUCTION

1.1 Mid - High Rise Timber Structures

For more than a century, concrete and steel have dominated as construction materials for high-rise buildings around the world. These materials continue to be excellent choices for construction of tall buildings and their enhanced structural properties have stretched buildings to great height. Until recently, there was no reason to challenge steel and concrete as essential structural materials. But now, due to global climate change, there is a need to reduce emission of greenhouse gases. With rapid growth of urbanization and demand of living and working space for billions of people, the use of alternative materials, such as structural wood, that has a smaller climate impact, is gaining new attention.

Tall wood buildings are not a new concept. 19 storey wooden pagodas were built in Japan 1400 years ago and are still standing in one of the highly seismic regions in the world. The Stadthaus project, London (2008) is an example of an innovative system of timber construction. It is a nine storey building constructed entirely with timber, claimed to be the world’s tallest pure timber residential building at the time of completion. Its structural system is made of a Cross-Laminated Timber (CLT) system pioneered by KHL-Austria, offering an effective solution for construction of large-scale tall wood buildings. In North America, however, the use of structural wood in construction of new high-rise buildings is not common. History of losses due to fire has regulated the limitations on the building area and height for timber structures in various structural building codes. The BC Building Code (*BCBC, 2009*) allows the construction of light-frame wood structures to maximum of 6 storeys since 2009, before which the limit was 4 storeys.
Mass timber products such as CLT, on the other hand, provide an excellent solution for large-scale and taller construction. Mass timber construction is an approach of combining mass timber panels to produce a system whose structural behaviour is significantly different from light-frame wood construction. Mass timber products have less variability in mechanical properties and they are produced in larger dimensions when compared to conventional timber products. They tend to have higher strength and stiffness than light wood frame systems and improved performance in terms of dimensional stability and fire. Although wood is a combustible material, mass timber structures have been recognized to perform well in fire according to North-American and International standards (*CLT Handbook – FPInnovations, 2011*). Further tests on the fire performance and the applications of charring methods associated with mass timber products can raise the confidence in the application of mass timber systems in high-rise buildings.

The application of mass timber in the construction of high rise structures in a high seismic region such as coastal BC is perceived to have shortcomings in the performance under earthquake loads due to lack of material ductility. But, a highly ductile material such as steel, when combined with timber, can enhance post-yield behaviour of timber structures. A good engineering design of a hybrid system that combines the merits of the two materials can overcome the limitations of light frame wood construction and revoke the building height restriction placed on timber buildings. During the past few years, extensive research has targeted the construction of hybrid structures in order to increase the performance of timber structures and also, owing to the demands of sustainable construction. One such system is the so-called FFTT system (*Tall Wood Report, 2012*), which is predominantly a mass timber vertical system with embedded steel beam sections that provide ductility in the system. This system is discussed in detail in Chapter 2.
1.2 Need for Research

Mass timber and steel hybrid systems have a great potential to impact the building industry, address issues of climate change and pose a challenge to concrete and steel structures. However, the current building codes provide no guidelines on seismic design and parameters for the construction of hybrid systems. Due to lack of design values and guidelines and the clear understanding of the global behaviour of hybrid systems, the implementation of a large scale timber-steel hybrid system has not yet been possible in Canada. Intensive analytical studies and experimentations that verify the system performance, identify the challenges posed by the system and optimize the connection design for hybrid systems are necessary in order to establish design guidelines and enable implementation. An advanced knowledge of the performance of hybrid systems, such as the FFTT system, will ultimately help in the development of reliable performance data, successful implementation of tall wood buildings and positively influence sustainability of the construction industry.

1.3 Research Objective

The purpose of this study is to investigate the component level behaviour of the FFTT system and propose recommendations that will facilitate successful implementation of this system in mid-rise and high-rise wood-hybrid structures. This objective is met by experimental investigations on the embedment behaviour of steel beams in CLT panels and the performance of different connection configurations through a series of quasi-static monotonic and reversed cyclic tests.
Chapter 2: LITERATURE REVIEW

2.1 Tall Wood Buildings

Numerous mid-rise wood-based structures have been built in Canada in the last century. Two such examples are 8-storey office building in Toronto from the 1920s, a brick-and-beam construction where exterior walls were made of bricks while the heavy timber post and beams were used in the interior and an 8-storey building built in 1905 in Vancouver (Mohammad et al., 2011). However, the history of fire losses in timber buildings resulted in area and height restrictions be placed on the buildings of combustible construction according to Part 3 of the National Building Code of Canada (NBCC).

The building height restriction for wood-frame buildings as regulated by the NBCC is summarized in Table 1. In 2009, the British Columbia Building Code (BCBC, 2009) made significant changes allowing 6-storey light-frame wood construction and specified the maximum building height as 18 meters (measured till the floor level of the top storey) above the grade (Division B, Part 3, Clause 3.2.2.45).

<table>
<thead>
<tr>
<th>YEAR</th>
<th>CONSTRUCTION TYPE</th>
<th>MAXIMUM STOREYS (With Sprinkler Systems)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1941</td>
<td>Heavy Timber</td>
<td>4</td>
</tr>
<tr>
<td>1953</td>
<td>Heavy Timber</td>
<td>3</td>
</tr>
<tr>
<td>1960-1985</td>
<td>¾ hr Fire Separation</td>
<td>3</td>
</tr>
<tr>
<td>1990-2005</td>
<td>1 hr Fire Separation</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 1: NBCC Building Height Restriction (Senez Reed Calder Fire Engineering Inc. 2008)
The recent introduction of innovative wood-based products and systems has revived the interest in extending the application of wood-based products from low-rise to mid-rise construction and potentially high-rise construction. Engineered wood products (EWPs) provide engineers with alternative materials that not only possess efficient structural properties but also better environmental attributes compared to other construction materials. One of the key EWPs that provide this opportunity is Cross-Laminated Timber (CLT), a mass timber product developed in Europe in the early 1990s. Researchers, fire experts and design professionals have recognized that the behaviour of a mass timber product is significantly different from light-frame wood construction. Mass timber systems outperform light-frame wood systems in terms of strength and stiffness. They have better dimensional stability and exhibit better fire ratings due to their solid nature. Extensive research has been conducted at FPInnovations (2011) to study the behaviour of CLT and recommend National Research Council (NRC) to make changes to the current restrictions placed on the wood buildings in Canada (Tall Wood Report, 2012).

Light-frame wood buildings with Plywood or Oriented Strand Board (OSB) sheathing constitute most of the low-rise timber construction in Canada. New materials, systems and design methods which are not defined in the code can also be implemented provided the expected level of structural performance is achieved and the proposed “alternative solution” proves to be equivalent to an “acceptable solution” specified in the Division B of NBCC. The term “acceptable solution” in NBCC 2005 (also in NBCC 2010) has replaced the term “requirements” from NBCC 1995 and the “alternative solution” represents the minimum performance level that satisfies the objectives of the NBCC (Mohammad et al., 2011). The adoption of this objective based approach in Canada can favour large-scale wood construction if equivalent performance...
and fire safety design can be demonstrated. Lack of design guidance and material ductility in the wood based materials has hindered the construction of timber high-rise buildings in seismically active regions of Canada. But, with the combination of ductile materials like steel, the post-yield behaviour of timber structures can be enhanced and thus, the seismic performance of the structure can be drastically improved. Therefore, timber and steel hybrid systems provide a promising solution for the construction of tall wood buildings.

2.2 Hybrid Systems

All timber structures, to some extent, are hybrid structures since connections are mostly made using steel and the foundations are usually concrete. However, true hybridization is the process of combining two or more materials to form a system by making use of the strength of each material and overcome their weaknesses. Hybridization can be classified as (i) Component Level Hybridization and (ii) System Level Hybridization.

Component hybridization exists when two different materials are combined together to act as a single structural unit that increases the overall load carrying capacity of the composite unit. Common examples of this hybridization type are hybrid bridge decks, hybrid slab/diaphragms, hybrid beams and columns. System hybridization combines different materials at the structural level to share the loads acting on them. Common examples for this type of hybridization are mixed vertical systems where the first few storeys are built from a material different from that of the upper storeys, hybrid roof trusses where timber is used for the top chord and steel as bottom chord and hybrid frames where wood and steel share both gravity and lateral loads (Khorasani, 2010).
One of the examples of hybridization at structural level is the 5-storey Kanazawa M Building, a hybrid vertical mixed system of concrete and timber-steel composite members (Figure 1), built in Kanazawa city of Japan in 2005. The first storey is of reinforced concrete while storey 2 to 5 consists of timber-steel hybridization. Performance based design was used for structural framing and static structural experiments were conducted on seismic performance of the shear wall and the buckling stress of the timber-steel hybrid system. The fire tests conducted on the three fireproof elements- column, girder and braces proved that the elements had 1-hour fire resistance rating, as required by the Building Code of Japan (Koshihara et al., 2005).

![Figure 1: Kanazawa M Building, Japan (Koshihara et al. 2005)](image)

The building uses laminated timber with built-in steel materials for beams, columns and braces in order to satisfy the structural and fire resistance requirements of a 5-storey building. The
column consists of square laminated timber with built in square steel bars and the beam is laminated timber with steel plates. The steel frame carries the load while timber acts as a buckling restraint and increases fire resistance of the member. The cross-section of braces is identical to that of a column, which is necessary for fire resistance certification. The floors and roofs are made of reinforced concrete slabs joined together using lag screws and steel plates built into the beams. The lateral walls are non-load bearing while the longitudinal walls are load bearing and made of nailed plywood. The stairs are made of steel frames.

The gravity loads in the Kanazawa M Building are distributed in the timber-steel hybrid system depending on the ratio of flexural rigidity (EI) of timber and steel frame (Table 2).

<table>
<thead>
<tr>
<th></th>
<th>$E$ (MPa)</th>
<th>$I$ (mm$^4$)</th>
<th>$EI$ (N-mm$^2$)</th>
<th>$EI/\sum EI$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber Frame</td>
<td>1.05 x 10$^4$</td>
<td>5.55 x 10$^8$</td>
<td>0.583 x 10$^{13}$</td>
<td>0.366</td>
</tr>
<tr>
<td>Steel Frame</td>
<td>2.05 x 10$^5$</td>
<td>4.95 x 10$^7$</td>
<td>1.01 x 10$^{14}$</td>
<td>0.634</td>
</tr>
</tbody>
</table>

Table 2: Flexural Rigidity Ratio of Timber and Steel Frame (Koshihara et al. 2005)

The timber and steel frame of the beam are joined at the beam edge using drift pins which transmit the load from the timber to the steel frame. The gusset plate from the steel frame of the column is connected to the steel frame of the beam using high tension bolts and the holes in the side of the timber frame are filled with timber after the high tension bolts are clamped (Figure 2). The timber of the column acts as the buckling restraint and hence the timber-based hybrid column does not buckle when steel frame yields in axial compression force.
The steel frame of the timber-based hybrid beam bears axial force while the timber frame, once again, acts as a buckling restraint. The braces are subjected to axial forces under lateral loads but the buckling of braces is not observed. The bearing wall, consisting of 24 mm structural plywood, 8 mm diameter screws and both vertical and horizontal frames of laminated timber, bears the lateral force during an earthquake event. The vertical frame transfers vertical shear force from the plywood bearing wall to the timber of the column. The timber of the column consists of bearing plates at both the ends, which transmits the axial force to the steel frame of the column (Koshihara et al., 2005).
2.3 Timber-Steel Hybridization

2.3.1 Material Properties

The structural response of buildings is characterized by the nature and behaviour of construction materials. It is essential to understand the properties of each material for the design of hybrid structures. Some relevant properties of steel and timber are summarized in Table 3, clearly showing that strength and stiffness of steel exceed that of timber by several magnitudes.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m³)</th>
<th>Elastic Modulus (MPa)</th>
<th>Compressive Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>7800</td>
<td>200000</td>
<td>400-1000</td>
<td>400-1000</td>
</tr>
<tr>
<td>S-P-F</td>
<td>400-450</td>
<td>9500</td>
<td>Parallel 11.2</td>
<td>Parallel 5.5</td>
</tr>
<tr>
<td>No.1/No.2</td>
<td></td>
<td></td>
<td>Perpendicular 5.3</td>
<td>Perpendicular 0.5</td>
</tr>
</tbody>
</table>

Table 3: Basic Material Properties of Steel and Timber (CSA S-16 and CSA 086)

2.3.1.1 Structural Steel

Steel has the largest unit mass of all the common construction materials. It is a homogeneous and isotropic material; that is, the material characteristics are the same in all directions. Steel has high compression and tension capacity with high stiffness and ability to sustain large inelastic deformation without fracture. Steel shows a linear stress-strain relationship up to yielding (Figure 3). This linear region is within the elastic range and the slope of the curve is elastic modulus (E) of the material. Beyond the elastic range, stress increases with the increasing deformation due to strain hardening until ultimate capacity after which the material fractures. Steel exhibits very good post-yield behaviour providing ductility to the system.
2.3.1.2 Timber

Timber is an anisotropic material; that is, mechanical properties vary in the three mutually perpendicular directions: - Longitudinal, Tangential and Radial (Figure 4). The properties in tangential and radial directions are often assumed to be equal for practical design purposes. The strength properties are strong parallel to the grain and weak across the grain. Timber is stronger in compression than in tension, both parallel and perpendicular to the grain. Non-homogeneous properties of wood result in great variability of properties within a structural element and further more variations exist between different structural elements. Wood is a hygroscopic material: loss and gain of moisture greatly affects the dimensional stability and strength. In addition, the physical and mechanical material properties are dependent on the species from which the timber
was harvested. The growing conditions and local imperfections (like knots) have a great impact on the strength properties (Keenan, 1986). Therefore the strength properties of timber products are retrieved by engineers based on the grading specified by CSA O86.

Figure 4: Principal Axes of Wood

Over the years, research has been carried out to make wood properties more reliable. Most common EWPs used as plate elements are plywood and OSB. Glue-laminated timber, also known as Glulam, is another very popular EWP that finds its main application as structural stick elements, such as in the design of beams and columns. Other commonly used EWPs include PSL (Parallel Strand Lumber), LVL (Laminated Veneer Lumber), LSL (Laminated Strand Lumber) and CLT (Figure 5). These mass timber products exhibit improved structural integrity and dimensional stability.
2.3.2 Cross-Laminated Timber – A Mass Timber Product

2.3.2.1 Overview

CLT products were developed in Germany and Austria in the early 1990’s and have gained popularity in residential and non-residential application in Europe and North America. CLT is an EWP where panels are formed by adhesively bonding layers of timber with alternating grain directions. The boards within each layer are placed parallel to each other but each layer is orthogonal to its adjacent layers (Figure 6). The layers are usually arranged symmetrically about the middle layer. The panels are usually made of 3, 5, 7 and 9 layers depending on desired thickness and structural requirement. CLT panels are used for shear wall, flooring and roofing systems.
Cross lamination increases in-plane and out-of-plane strength and stiffness in both directions (longitudinal and transverse) similar to two-way action seen in reinforced concrete slab elements. Different qualities of boards may be used based on the application of the CLT panels. Usually higher grade lumber is used in the outer layers whereas lower grade may be used for inner layers (Chen, 2011). The cross-lamination provides better dimensional stability to the product and makes prefabrication of long and wide panels possible. Different manufacturers produce CLT panels of varying dimensions: the typical dimensions of CLT panels as reported in the CLT Handbook (FPInnovations, 2011) are summarized in Table 4.
2.3.2.2 Mechanical Properties

The estimation of design properties of CLT is not straightforward. These properties not only depend on the species and quality of the wood used, but also the number, orientation, and thickness of the layers. The orientation of the outer layer of CLT is very important since it affects the mechanical behaviour especially for out-of-plane bending (Ashtari, 2009). The orientation of the outer layer of CLT panels used as wall elements is vertical so as to increase the resistance against gravity loads. The outer layer orientation of CLT panels in floor systems is parallel to the spanning direction due to higher bending resistance to out-of-plane loads (CLT Handbook - FPInnovations, 2011). Since CLT panels consist of crosswise layers, the material properties and the global behaviour of the panel is different from that of individual layers. The elastic properties of Spruce-Pine-Fir (SPF) No.1/No.2 which is a commonly used wood species for CLT, are summarized in Table 5.

<table>
<thead>
<tr>
<th>Modulus of Elasticity (MPa)</th>
<th>Shear Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_L$</td>
<td>$E_T$</td>
<td>$E_R$</td>
</tr>
<tr>
<td>$G_{LR}$</td>
<td>$G_{LT}$</td>
<td>$G_{RT}$</td>
</tr>
<tr>
<td>$\gamma_{LR}$</td>
<td>$\gamma_{LT}$</td>
<td>$\gamma_{RT}$</td>
</tr>
<tr>
<td>10660</td>
<td>725</td>
<td>1087</td>
</tr>
<tr>
<td>522</td>
<td>490</td>
<td>53</td>
</tr>
<tr>
<td>0.316</td>
<td>0.347</td>
<td>0.469</td>
</tr>
</tbody>
</table>

Table 5: Properties of Individual CLT Boards (Yawalata et al. 2011, Wood Handbook 2010)
The global properties of CLT are usually derived based on experimentation or from the properties of each layer using different engineering theorems like Gamma Method, Shear Analogy and Composite Theory. The research conducted by Gsell et al. (2007) concluded that the assumption of orthotropic, homogeneous and linear elastic material behaviour for CLT is sufficiently accurate to evaluate the mechanical behaviour of CLT. For simplified design, measured global orthogonal characteristics are easier to deal with than the orthogonal properties derived from each layer. The properties of CLT panels derived from their study are summarized in Table 6. X, Y and Z in subscript refer to three mutually perpendicular directions; $E_0$ and $E_{90}$ refer to stiffness modulus parallel and perpendicular to the majority grain direction.

<table>
<thead>
<tr>
<th>$E_y (E_0)$</th>
<th>8210 GPa</th>
<th>$\gamma_{yx}$</th>
<th>0.090</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_x (E_{90})$</td>
<td>4630 GPa</td>
<td>$\gamma_{zx}$</td>
<td>0.040</td>
</tr>
<tr>
<td>$E_z$</td>
<td>500 GPa</td>
<td>$\gamma_{yz}$</td>
<td>0.364</td>
</tr>
<tr>
<td>$G_{xz}$</td>
<td>949 MPa</td>
<td>$\gamma_{xy}$</td>
<td>0.051</td>
</tr>
<tr>
<td>$G_{xy}$</td>
<td>747 MPa</td>
<td>$\gamma_{xz}$</td>
<td>0.380</td>
</tr>
<tr>
<td>$G_{yz}$</td>
<td>54 MPa</td>
<td>$\gamma_{xy}$</td>
<td>0.022</td>
</tr>
</tbody>
</table>

Table 6: Elastic Properties of CLT (Gsell at al. 2007)

Following an approach of assuming an isotropic behaviour in the plane for “simplified design purposes”, Structurlam, one of the major CLT manufacturers in Canada, specifies the same modulus of elasticity (9600 MPa), shear strength (1.5 MPa) and bending strength (11.8 MPa) in the two in-plane (0° and 90°) directions of CLT Panels (CLT Design Guide –Structurlam. 2012).
2.3.2.3 Connections in CLT

Self-tapping screws are widely used in Europe due to ease of installation and high withdrawal capacity. Nails are used with metal plates and brackets for CLT assembly connections but are not as popularly used as wood screws. Bearing-type connections such as shear plates and split rings are not commonly seen in connecting CLT components. Dowel-type connections, on the other hand, are used in mass timber construction and their application can be extended to joining CLT panels. Proprietary connection systems like Geka Connectors, Knapp Systems and glued-in rods can also be used for CLT connections (CLT Handbook – FPInnovations, 2011).

Johansen’s Yield Model (European Yield Model) can be used for dowel-type connection design. According to the model, dominant failure mode for slender-type fasteners is ductile in nature. Cross-lamination creates a reinforcing effect that improves the splitting resistance of CLT compared to other wood-based material (Uibel and Blass, 2006). However, when fasteners are driven through the end grain of the wood, splitting may occur due to tensile stresses in perpendicular to the grain direction. Therefore, conditions where brittle failure modes may occur need to be established when larger diameter fasteners are used in CLT. Further research on brittle failure modes associated with different fastener types in CLT connections should be conducted.

The Canadian Timber Design Standard CSA O86 (CSA 2009) specifies in Clause 10.4.4.3.3, the embedment formulae for general wood-based products loaded in parallel and perpendicular to the grain directions. The formulae include the density of the wood product and fastener specifications. The embedment strengths (MPa) parallel and perpendicular to the grain for wood-based products, according to CSA O86 are given by Equations [1] and [2].
\[ f_{\text{parallel}} = 50G(1 - 0.01d) \] \hspace{1cm} \text{Equation 1}

\[ f_{\text{perpendicular}} = 22G(1 - 0.01d) \] \hspace{1cm} \text{Equation 2}

where,

\( G \): Relative density of wood-based product (dimensionless)

\( d \): Fastener diameter (mm)

The equation proposed by Uibel and Blass (2006) to establish characteristic embedment strength of dowel connection in CLT is given by Equation [3]

\[ f = \frac{0.035(1-0.015d)\rho^{1.16}}{(1.1(\sin \alpha)^2 + (\cos \alpha)^2)} \] \hspace{1cm} \text{Equation 3}

where,

\( d \): Fastener diameter (mm)

\( \rho \): Average density of main member (kg/m\(^3\))

\( \alpha \): Angle load and grain direction of outer most layer (degree)

A preliminary experimental study on the embedment strength of CLT was conducted using 20 specimens considering two dowel diameters of 12.7 mm (0.5”) and 25.4 mm (1”) at the University of British Columbia (Healey et al., 2013). The study confirmed that the Canadian code equations [1], [2] produce overly conservative results because the additional benefits that CLT offers are not fully accounted for in the equations. This makes the design of dowel-type connection using these equations impractical. The European equation [3] was specifically developed for CLT and shows much better agreement, although not always conservative. Figure 7 shows the comparison between the test results and the equations.
Specific embedment formulae for dowel-type fasteners in CLT, that account for cross-lamination, layers, species, edge gluing and panel specifications need to be developed. Once embedment properties of fasteners in CLT are established, the lateral resistance of the dowel-type connections can be calculated based on CSA O86 guidelines.

2.3.2.4 Fire Resistance of CLT

It is necessary to determine the fire-resistance rating of wood products while designing timber structures in order to ensure their performance meets the requirements of the building code. Massive timber products have the inherent nature to char slowly when exposed to fire and maintain structural capacity for extended duration of time. Char is a residual matter resulting from combustion of certain solids, such as wood, when subjected to high heat. The surface of
thick solid wood member starts charring when subjected to fire until a protective layer is formed, which can partially retain the load carry capacity of the member by insulating the remaining core of wood. Therefore, CLT panels provide better fire resistance compared to lumber and small thickness panels used in light-frame wood construction. The performance under fire is comparable with the typical assemblies of non-combustible construction (CLT Handbook - FPInnovations, 2011). The charring rate of the panel is influenced by the adhesive type used during the manufacturing process (Frangi, 2009). The reduced (effective) cross-section method, specified by CSA O86 can be used to calculate the fire-resistance rating of CLT panels and the implications of fire exposure to the structural design should be considered.

2.3.2.5 Seismic Performance of CLT Shear Walls

In light-frame wood systems, lateral loads are transferred to the foundation by vertical bracing achieved by mainly shear walls with panel sheathing. Plywood and OSB panels are commonly used for horizontal diaphragms and shear walls to brace the building for wind and seismic loads. The wood sheathing behaves linearly and elastically under cyclic loads and failure is brittle in nature with no dissipation of energy. Therefore, the connections are designed to be “semi-rigid” allowing for plastic deformation and energy dissipation, and thus adding ductility to the system.

The force-deformation curve of a typical timber shear wall system (simplified in Figure 8) is initially steep till its elastic limit, and then the curve becomes non-linear and less steep reaching a peak, where the maximum connection capacity may be found as $F_{\text{max}}$. The displacement at the “near collapse” criterion, taken as $0.8F_{\text{max}}$, is defined as the ultimate displacement (Ceccotti et al., 2002).
Various tests have been conducted to study the behaviour of CLT shear walls. One of the most comprehensive studies conducted to evaluate seismic behaviour of CLT was the SOFIE Project, which included in-plane cyclic tests with different layouts of connections and openings (Ceccotti et al., 2006). A total of 14 tests with four different wall configurations were conducted under monotonic loads (EN26891) and reversed cyclic loads (EN12512). This study confirmed the high influence of the design of joints on the overall behaviour of the structure. The CLT panels behaved almost completely rigid and all the dissipating energy resulted from the connections. The hysteresis behaviour of the system showed good ductile behaviour and energy dissipating capabilities. Despite the high stiffness of the CLT panels, the system was proved to be suitable for seismic applications.
A full-scale shake table test was also carried out on a 3-storey structure on a 1D shake table of NEID in Tsukuba, Japan and a 7-storey structure made of CLT slabs and walls at the E-Defense facility in Miki, Japan. The test outcomes indicated that CLT buildings displayed good seismic behaviour under all the severe earthquake motions. The seismic reduction factor ($R = R_d \times R_o$) was approximated to be equal to 3 for the seismic design of CLT shear walls (*Ceccotti et al.*, 2010).

The behaviour of CLT wall assemblies under lateral loading can be broken down into two components: Rocking/Overturning and Shear. Traditionally, metal brackets are used to join perpendicular panels and anchor the CLT wall panels to the foundation. Brackets are also used to transfer the shear load between the diaphragm and the wall. CLT walls perform adequately under seismic loads when nails / slender screws are used with steel brackets to connect the walls to the floors below (*Schneider, 2009*).

A series of quasi-static tests on CLT wall panels with different wall configurations and connection details were conducted by FPInnovations in Vancouver (*Popovski et al., 2010*). CLT wall panels behaved as rigid bodies during the experimentation. Slight shear deformation was observed in the shear walls but most of the panel deflection occurred as a result of deformation of the joints connecting the wall to the foundation. Lateral deformation of CLT wall panels are greatly influenced by the base connectors. The connection between CLT walls and the floors below using diagonally placed screws are not recommended due to lower ductile behaviour of the walls. The CLT wall systems with brackets and hold-downs showed the best ductility values and are recommended for high seismic regions (Figure 9).
Based on the research conducted in Europe, Japan and Canada, it can be concluded that CLT construction shows adequate seismic performance when nails and screws are used with metal brackets and hold-downs. The NBCC specifies $R_d=2.0$ and $R_o=1.5$ for braced frames when designed with moderately ductile connections. In comparison with the test data available on braced frames (Popovski et al., 2008), CLT structures with ductile connections have equivalent seismic behaviour or better. Therefore, conservative estimates of $R_d$ and $R_o$ values for the shear walls of CLT panels with nail and screw connections are stated equal to 2.0 and 1.5 respectively (CLT Handbook - FPInnovations, 2011).

### 2.3.3 Advantages of Timber-Steel Hybridization

Even though steel and timber are two materials with different properties, combining them can take advantage of their inherent properties and diminish their limitations. Steel excels in tension...
and timber behaves well in compression. For instance, during the construction of hybrid steel-timber truss, wood should be placed on the top of the truss (compression) and steel at the bottom chord (tension).

The force-deformation response of structures is dependent on the hysteresis response of the applied structural materials under cyclic loads. The area under the hysteresis loop represents the dissipation of energy. While the stress-strain relationship of timber under compression is non-linear, timber does not exhibit post-yield behaviour in tension and shear which leads to brittle failure. Structural steel on the other hand, dissipates great amounts of energy under cyclic loads and thus, if designed efficiently, steel structures exhibit extreme ductile behaviour during an earthquake event.

Wood has high strength to weight ratio compared to steel and concrete resulting in lighter structures. Since earthquake forces are proportional to the weight of the building, wood frame buildings are subjected to lower earthquake forces than concrete and steel structures. But, the performance of timber structures during a seismic event is highly dependent on the behaviour of connection under cyclic loading. The steel elements (or connections) add ductility to the system required to dissipate energy without collapse during an earthquake event. Therefore, timber structures can exhibit adequate ductility under cyclic loads through incorporation of ductile materials like steel in the system. Seismic performance of timber buildings can be, thus, improved by timber-steel hybridization. Moreover, combining steel and wood results in sustainable construction due to high recycle content in steel and low carbon footprint of wood.
2.3.4 Challenges of Timber-Steel Hybridization

Due to differences in material properties of steel and timber, an efficient connection between the materials is of high priority. Change in temperature causes steel to expand and contract but has little to no effect on wood. On the other hand, change in humidity has little effect on steel but causes wood to shrink or swell. Therefore, great care should be taken while combining steel with wood in order to accommodate dimensional changes that may occur. This challenge can be mitigated by optimizing the connection design of hybrid components. An ideal connection between steel and timber should be yielding of steel components before wood crushes. Splitting of wood is a brittle failure and should be avoided. The lack of established guidelines towards the design of connections in hybrid systems makes the optimization of connection design, one of the biggest challenges of timber-steel hybrid construction.

2.4 Design Principles for Seismic Force Resisting Systems in Timber Structures

Two main principles of design of Seismic Force Resisting System (SFRS) currently in practice are: (i) Force-Based Design and (ii) Displacement-Based Design.

2.4.1 Force-Based Design Approach

The current code based provisions of seismic design of timber structures follows the force-based design approach which provides sufficient lateral strength to the structure under a single seismic hazard level for life safety (Filiatrault et al., 2002). The force-based design procedure uses a spectral acceleration ($S_a(T)$) from the NBCC 2010 Design Spectrum (Figure 10), based on 2% in 50 years probability of exceedence, to determine the lateral strength required by the system to remain elastic. The applied loads are then reduced using seismic reduction factors that account
for the inherent ductility ($R_d$) and over-strength ($R_o$). The reduction of the elastic force by ductility and over-strength factors allows for inelastic deformation in the structure and dissipation of energy during a seismic event.

![NBCC 2010 Design Spectrum](image)

**Figure 10: Design Spectrum of Vancouver – NBCC 2010**

Most commonly used LLRS (Lateral Load Resisting Systems) in steel and timber construction are moment frames and braced frames and light-frame wood or CLT shear walls. Steel frames can be designed to provide different levels of ductility: D (Ductile), MD (Moderate Ductility), LD (Limited Ductility) and CC (Conventional Construction), while timber shear walls are designed to provide ductility through the yielding of nails. The seismic reduction factors for these different systems are summarized in Table 7. No such reduction factors for steel-timber hybrid systems are currently available.
### Table 7: Ductility and Over-Strength Reduction Factors (NBCC 2010)

<table>
<thead>
<tr>
<th>Ductility Type</th>
<th>Steel Moment Frame</th>
<th>Steel Braced Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R_d$</td>
<td>$R_o$</td>
</tr>
<tr>
<td>D</td>
<td>5.0</td>
<td>1.5</td>
</tr>
<tr>
<td>MD</td>
<td>3.5</td>
<td>1.5</td>
</tr>
<tr>
<td>LD</td>
<td>2.0</td>
<td>1.3</td>
</tr>
<tr>
<td>CC</td>
<td>1.5</td>
<td>1.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type</th>
<th>Timber Moment/Braced Frames</th>
<th>Timber Shear Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R_d$</td>
<td>$R_o$</td>
</tr>
<tr>
<td>MD</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>LD</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Nailed (No Gypsum)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Nailed (With Gypsum)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Other</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The shortcomings of the force-based approach lie in the determination of the fundamental period of the system. Empirical formulae for elastic fundamental period available in the NBCC are not tailored for mass timber and hybrid systems. Due to lack of knowledge of global seismic response of mass timber (CLT) and hybrid buildings, reduction factors are primarily based on engineering judgement. Moreover, the reduction of elastic forces using the seismic reduction factors implies that the maximum displacement experienced by the structure if it remained elastic is equal to the inelastic displacement of the structure. This assumption of “equal displacement” rule is, however, not appropriate for short-period wood frame structures (Filiatrault et al., 2002).
In addition, difficulty lies in establishing the yield point and ultimate displacements for timber LLRS.

### 2.4.2 Displacement-Based Approach

The concept of the displacement-based approach was first proposed by Priestley (1998) where the design is based on a specific target displacement at a particular seismic hazard level. Filiatrault and Folz (2002) adopted this approach to light-frame wood buildings. A large portion of structural and non-structural damages to light-frame wood buildings after an earthquake event, designed according to the force-based design approach, have resulted from excessive lateral displacement. In order to overcome the disadvantages of force-based design and limit the damage in wood frame buildings, inter-storey drift should be the key parameter considered at the beginning of seismic design (Filiatrault et al., 2002). The maximum deformation experienced by the structure is evaluated and the system is designed to resist this deformation either elastically or plastically. Plastic design ensures dissipation of energy during a seismic event but results in larger deformation. This method is known as Performance Based Plastic Design (PBPD). While this procedure has been widely implemented on concrete and steel structures, it has been recently explored for wood buildings through implementation on a two storey wood-frame structure (Filiatrault et al., 2010). Extensive research efforts have been carried out as a part of NEES Wood project (van de Lindt et al., 2011) focusing on the performance-based seismic design procedure for mid-rise light-frame wood buildings.

The displacement-based design process converts acceleration spectra into inter-storey drift spectra to evaluate the minimum stiffness required at each floor level to restrain the storey drifts within the given design limits (Pang et al., 2007). The essential parts of the PBPD are defining
performance acceptance criteria for a particular seismic hazard level and determining the allowable damage in the structure without leading to collapse.

Hinges are defined as the point of plastic yielding. Each point on the hinge behaviour model corresponds to different performance levels that define acceptance criteria at each level. Typically, the acceptance criterion is indicated in terms of plastic deformation for hinges in steel structures and rotations or curvatures for hinges in concrete structures. The numerical acceptance criterion for wood-frame buildings is expressed in terms of acceptable deformation ratio at each performance level for various primary and secondary wood components (Table 8-4, FEMA 356 (2000)).

The three performance levels (Table 8) defined by FEMA 356 are: - Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The IO level accommodates minor damage to wood-frame structure or resisting shear walls after a moderate earthquake event where no structural repair is necessary prior to re-occupancy. The objective of the LS level is to minimize life-threatening injuries to the occupants. It allows structural damage and strength degradation but provides a margin of life safety. The CP level corresponds to deformation just before the failure point. The sole focus of this level is prevention of loss of human life resulting from partial or complete collapse of the structure. Wood-frame buildings that meet the CP criteria are expected to remain standing with the maximum drift of 3% but non-life-threatening injury to the occupants and economic losses are acceptable at this level (FEMA 356, 2000).
### Performance Levels for Wood Shear Walls (Table C1-3, FEMA 356 -2000)

<table>
<thead>
<tr>
<th>Performance Levels</th>
<th>Seismic Hazard Level</th>
<th>Drift Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Occupancy (IO)</td>
<td>50 % in 50 years</td>
<td>1 %</td>
</tr>
<tr>
<td>Life Safety (LS)</td>
<td>10 % in 50 years</td>
<td>2 %</td>
</tr>
<tr>
<td>Collapse Prevention (CP)</td>
<td>5 % in 50 years</td>
<td>3 %</td>
</tr>
</tbody>
</table>

Although some limitations of the force-based procedure are alleviated, the displacement-based design approach has its own challenges. The estimation of structural performance involves several uncertainties like the variation in ground motion characteristics and the capacity of the components of the system to resist the imposed demands. This approach requires the knowledge of global nonlinear load-displacement behaviour of the building and its viscous damping at the target displacement (Pang et al., 2007). In addition to sophisticated structural analysis models, system testing is necessary to obtain the required information for the design. With further research and test results on the global behaviour of timber structures, this design procedure can be valuable in controlling damages resulting from seismic events in timber buildings.

### 2.5 Capacity Design Philosophy in Timber Design

A structure should be designed as a chain whose weakest link is ductile. If the weakest link is ductile, the system can undergo larger deformations as opposed to a brittle weak link where the system fails suddenly. The philosophy, known as ‘Capacity Design’ is based on the hypothetical behaviour of the structure where the system is designed so as to trigger a desired failure mechanism during a seismic event and suppress any undesired response. In this approach, the weak links of the system are predetermined and such members are designed to behave as ductile.
elements. These elements are detailed to undergo yielding under cyclic loads, accommodate large deformations, limit the inelastic behaviour of other components and thus, avoiding potential brittle failure in the system (Mitchell et al., 2003).

In steel moment frames, seismic force generated at floor levels are transferred to the foundation through beams and columns. These members are typically designed to yield before the connections. Moreover, beam failure mechanisms are preferred since they provide sufficient ductility without causing undesirable collapse mechanism. If the column is the weak link, the stability of the entire system is compromised (Soft Storey) while the failure of a beam would be a localized effect (Figure 11). Therefore, it is desirable to design columns stronger than beams resulting in the so-called ‘Strong – Column Weak – Beam’ failure mechanism in moment frames (Paulay, 1995).

![Figure 11: Strong – Column Weak – Beam (left) and Soft-Storey Mechanism (right)](image-url)
Unlike steel frame structures, failure of wood in tension or bending in wood-frame structures is not recommended due to brittle nature of the material. The nonlinear behaviour and energy dissipation in the system should occur at the connections. In braced timber frames, wood members are designed to remain elastic while the energy dissipation occurs at the brace-to-frame connections. The braces are designed not to buckle (Popovski et al., 2008). Similarly, the ductility in the shear walls in light-frame wood buildings is induced through the connections. The nails of timber shear wall systems are designed to yield before the wood failure. This ensures a ductile failure mechanism as opposed to brittle crushing of wood in the shear wall.

Hybrid systems, similar to conventional timber structures, have to be designed to resist gravity loads like dead, live and snow loads and lateral loads like wind and earthquake loads. In order to optimize the design of hybrid systems, the strengths of each material should be utilized. Therefore, the capacity design strategy is most appropriate for the design of a hybrid structure in order to avoid complex techniques of determining potential collapse mechanism. This strategy helps in the development of a hierarchy of capacity among the components of the structure.

2.6 Innovative Hybridization in Timber Construction

Over the last decade, several innovative hybrid systems were developed to promote the use of timber in large-scale construction.

2.6.1 Post-Tensioned Timber Construction

The concept of post-tensioned timber construction, an innovative ductile connection, is based on the post-tensioned precast concrete systems by Precast Seismic Structural Systems (PRESSS) program at the University of California in San Diego (Priestley et al., 1999) which was extended
to steel moment frames. This system was subsequently adopted for timber seismic force resisting systems at the University of Canterbury in New-Zealand (Palermo et al., 2005).

The structural elements are connected by a combination of post-tensioned tendons that provide self-centering characteristics that limit residual deformation after a seismic event. The inelastic seismic demand is accommodated at the connection through opening and closing of an existing gap while the structural components are kept within the elastic range and, thus, the structural damage is minimized (Palermo et al., 2006a). The controlled rocking mechanism within the connection and self-centering properties of unbonded tendons result in flag-shaped hysteresis behaviour (Figure 12).

![Figure 12: Beam-Column Assemblies and Idealized Hysteresis Loop (Palermo et al. 2006a)](image)

Experiments on beam-to-column, column-to-foundation and wall-to-foundation were conducted at the University of Canterbury (Palermo et al. 2005, 2006a, 2006b and Smith et al., 2007). The behaviour of the connection was a stable flag-shaped hysteric loop with negligible residual deformation that illustrates the self-centering capacity of the hybrid system (Figure 13). The
yielding point corresponds to yielding of energy dissipation devices and the moment capacity was achieved from the elongation of the tendons. Stiffness degradation and structural damage did not occur and a maximum drift of 4.5 % was achieved. In addition to these tests, coupled and parallel wall tests were conducted where energy dissipation occurred due to relative motion between the walls.

Different methods of energy dissipation were investigated: the U-shaped Flexural Plates (UFP) as shown in Figure 14 was the most effective method (Smith et al., 2008).
Although this hybrid system was applied to LVL frame and wall system, it can be extended to other wood based materials after suitable testing. Due to design flexibility, speed of construction and enhanced seismic performance, post-tensioned timber construction has the potential for future development and implementation around the world.

2.6.2 Infill Wall System

Common infill wall systems include masonry infill walls in steel and concrete moment frames. Infill walls in concrete or steel frames acting as stud partition walls are not accounted for in the structural design but their additional mass is taken into consideration. Previous studies have confirmed that the presence of infill walls increases stiffness and strength of the frame and the resistance to incremental collapse; but they also decrease the ductility of the system (Kodor et al., 1995).
The addition of masonry infill walls in a steel moment frame impacts the seismic performance of the system because of high stiffness of masonry walls and high flexibility of steel frame. Therefore, it is very necessary to detail the structural elements such that the infill panels are isolated from the frame so that they do not participate in the lateral load resistance of the structure or their contributions are accounted in the structural design of the system (Yousuf et al., 2009).

Preliminary analytical studies examined the behaviour of wood (CLT) infill walls in steel moment frames (Dickof et al., 2012). Although the stiffness added by wood infill walls was not as great as by masonry walls, the reduced weight of the system was identified as an advantage. CLT infill panels provided higher strength and stiffness than OSB / plywood shear walls. The addition of infill walls reduced the ductility of the system. The reduction in ductility was least severe for low ductility moment frames and therefore, no evident benefit was found in choosing high ductility over low ductility moment frames. More detailed parametric studies are required to optimize the member sizes in order to get maximum system ductility. Further research and experimental testing on the seismic reduction factors and the connection behaviour need to be carried out for successful implementation of such a hybrid system.

2.6.3 FFTT System for Tall Wood Buildings

‘Finding Forest Through Trees’ abbreviated as FFTT is an unique timber and steel hybrid approach for constructing tall wood buildings (Tall Wood Report, 2012). This innovative system that promotes environmentally friendly high-rise buildings was proposed by Michael C. Green (Principal, Michael Green Architecture) and Eric Karsh (Principal, Equilibrium Consulting Inc). The FFTT system has concrete foundations but no concrete is used above grade. It consists of
mass timber wall panels such as CLT, LVL or LSL as the vertical system and steel beams are partially embedded into the panel faces. Efficient connections between the wood panels and steel are necessary to ensure an adequate seismic response of the structure. The connections should possess enough ductility to deform under loading and prevent brittle failure of the wood panels. The proposed connection for the embedment of the beam section into the face of the panel is a simple bolted connection as shown in Figure 15.

![Figure 15: Wall-to-Beam Connection (Tall Wood Report, 2012 – used under CC License)](image)

The steel beams hold together the wall panels across openings (Figure 16). The force transfer occurs through the bearing of header beams on the solid wood panels. The steel beams act as the ductile weak link of the system and are designed and detailed such that plastic hinging occurs at the beam sections triggering a “Strong – Column Weak – Beam” failure mechanism. The system
is capacity designed so that the hierarchy of capacity follows the path of beam yielding before the connection failure and the fracture of wood panels. This hierarchy of capacity can be achieved by reducing the cross-section of the steel beams in the vicinity of intersection between the wall panel and the beam (*Tall Wood Report, 2012*).

![Figure 16: FFTT System (Tall Wood Report, 2012 – used under CC License)](image)

Four combinations of LLRS are proposed for the timber high-rise buildings based on the number of storeys (extending up to 30 storeys) using the FFTT methodology. These systems are listed in Table 9.

<table>
<thead>
<tr>
<th>System</th>
<th>Lateral Load Resisting Combination</th>
<th>Storeys</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Structural Core Wall – Glulam Perimeter Columns</td>
<td>12</td>
</tr>
<tr>
<td>II</td>
<td>Structural Core Wall – Interior Shear Walls – Glulam Perimeter Columns</td>
<td>20</td>
</tr>
<tr>
<td>III</td>
<td>Structural Core Wall – Perimeter Moment Frame</td>
<td>20</td>
</tr>
<tr>
<td>IV</td>
<td>Structural Core Wall – Interior Walls and Exterior Moment Frame</td>
<td>30</td>
</tr>
</tbody>
</table>

*Table 9: FFTT System Combinations (Tall Wood Report, 2012 – used under CC License)*
The Type I combination that consists of wood core walls and glulam perimeter columns as the supporting structure is proposed for buildings up to 12 storeys. Two combinations of LLRS are suggested for structures extending up to 20 storeys. Type II is a combination of wood core walls, interior wood shear walls and glulam perimeter columns, while Type III system replaces the interior wood shear walls and perimeter glulam columns with perimeter wood walls. The steel beams run from end-to-end across the perimeter wall and support the panels across openings (Figure 17). The Type IV system combines wood core walls, interior and exterior walls pushing the building height up to 30 storeys. The walls are anchored down using ductile hold downs or dampers and rigid (elastic) shear connectors (*Tall Wood Report, 2012*).

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*Figure 17: Type III LLRS of FFTT (Tall Wood Report, 2012 – used under CC License)*
A comparative study conducted in the ‘Tall Wood Report’ (2012) based on the gross area of mass timber products and a typical concrete core wall confirmed that the resistances of solid wood panels are comparable in magnitude with that of reinforced concrete in many aspects such as stiffness, axial, shear and in-plane and out-of-plane flexural capacities. The higher strength to weight ratio of timber structures result in lower gravity and seismic loads on the structure and lead to potential savings in foundation costs especially in poor soil conditions. The biggest structural challenge, however, is the development of structural solutions with sufficient ductility and efficient connection designs. A good engineering design of a hybrid system like FFTT could set the standard for the development of construction technology for safe high-rise timber structures.

Further structural analyses, testing and diligent peer review are necessary before the FFTT system can be successfully implemented in practice. The experimental investigations carried out within the scope of this research (presented in Chapter 3) address the interaction between the timber and steel, embedment strength of CLT, influence of cross-section profiles and placement of the embedded beams on the capacity and energy dissipation under cyclic loads.
Chapter 3: EXPERIMENTAL PROGRAM

This chapter describes the test specimen details, instrumentation and testing protocol adopted to meet the objective of this study. Subsequently, in Chapter 4, the results are presented and discussed. The tests were conducted at two levels: 1) Material level and 2) Component level. Material testing was carried out to evaluate the embedment strength of CLT panels. The compression behaviour under the bearing of circular dowels was compared with square rod profile, the results of which can be extrapolated to understand the bearing of steel beam profiles on CLT wall panels within the FFTT system. Component testing evaluated the behaviour of embedded wide-flange I-sections and hollow rectangular sections with different configurations through quasi-static monotonic and reversed cyclic tests.

3.1 Material Level Testing

The connection of the wall-beam interface is designed to transfer the force through bearing of steel beams on the wall panels. The bearing capacity of CLT panels was measured through a series of material tests conducted according to ASTM D 5764-97a (2002).

3.1.1 Materials

The specimens consisted of CLT made of SPF-No.1/No.2. The specimens were clear of knots, cross-grain and natural or manufacturing defects. The moisture content of each specimen at the time of testing was measured using a moisture meter. The average moisture content was determined to be 9.4 %. The dry density of the samples was also determined by oven drying the
samples at 103°C for 48 hours according to ASTM D2395-07a. The average dry density of the material (0% Moisture Content) was 410 kg / m³.

3.1.2 Specimen Description

The test samples were rectangular members of cross section 150 mm x 150 mm and 120 mm thickness. One of the outer most layers was 32 mm and two inner layers were 35 mm thick. The specimens were cut from symmetrical 7-ply (239 mm thick) CLT panels used for subsequent component level testing. Therefore, the other outer layer of the test specimen was 18 mm thick. Two configurations (Figure 18) were used to investigate the embedment behaviour of CLT:

(i) Embedment behaviour under bearing of a circular steel dowel of 25.4 mm (1”) diameter (according to ASTM D 5764-97a)
(ii) Embedment behaviour under bearing of a square steel rod of 25.4 mm x 25.4 mm (1” x 1”) cross section.

Figure 18: CLT Loading Configuration - Circular Dowel (Left) and Square Rod (Right)
3.1.3 Test Procedure

The test equipment was an Instron Universal Testing Machine at Wood Mechanics Lab of the Department of Wood Science at the University of British Columbia, Vancouver. The loading rate was maintained at 1 mm/min and the tests were terminated when the embedment of one half of the diameter was achieved or if no further increase in load was observed. The compression of wood was assumed equal to the movement of the movable crosshead, measured with a linear variable differential transducer, based on the assumption that the steel dowel or square profile did not deform under the applied load (Figure 19).

Figure 19: Bearing Strength Test Setup
The embedment strength of the specimens was calculated by evaluating the yield load from the load-deformation plot at deformation equal to 5% of the dowel diameter (Figure 20).

![Figure 20: Typical Load-Deformation Curve (ASTM D 5764-97a)](image)

Three series of tests were conducted as summarized in Table 10.

<table>
<thead>
<tr>
<th>Series</th>
<th>No. of Samples</th>
<th>Steel Profile</th>
<th>Grain Orientation with respect to Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Parallel</td>
</tr>
<tr>
<td>1</td>
<td>15</td>
<td>Circular Dowel</td>
<td>1.5 Layers</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>Square Rod</td>
<td>1.5 Layers</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>Square Rod</td>
<td>2 Layers</td>
</tr>
</tbody>
</table>

Table 10: Bearing Strength Test Details
3.2 Component Level Testing

A series of component tests were conducted in the Structural Laboratory of FPInnovations, Vancouver to validate the hypothesis of the ‘Strong-Column Weak-Beam’ failure mechanism of the FFTT system by yielding of the embedded steel beams before crushing of the wood. Furthermore, the energy dissipation under cyclic loads was investigated.

3.2.1 Materials

Two 7-ply CLT panels of grade S-P-F No.1/No.2 of 0.9 m (3’) wide and 4 m (13’) long were used. The outer laminations were 32 mm while the inner laminations were 35 mm thick because the outer surfaces were planed. The overall thickness of the panel was 239 mm. The average apparent density of the specimens, calculated based on weight and volume, was 430 kg/m$^3$. The average moisture content of the specimens measured by moisture meter was 9.4 %. The design material properties listed by the manufacturer of the CLT product (Structurlam) used in the project are summarized in Table 11. The CLT handbook (*FPInnovations, 2011*), however, specifies a more detailed procedure of determining modulus of elasticity values ($E_0$ and $E_{90}$).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength ($f_{c,0}$)</td>
<td>11.5 MPa</td>
</tr>
<tr>
<td>Tensile Strength ($f_{i,0}$)</td>
<td>5.5 MPa</td>
</tr>
<tr>
<td>Bending Strength ($f_{b,0}$ and $f_{b,90}$)</td>
<td>11.8 MPa</td>
</tr>
<tr>
<td>Shear Strength ($f_{v,0}$ and $f_{v,90}$)</td>
<td>1.5 MPa</td>
</tr>
<tr>
<td>Rolling Shear Strength ($f_{rv}$)</td>
<td>0.5 MPa</td>
</tr>
<tr>
<td>Modulus of Elasticity ($E_0$ and $E_{90}$)</td>
<td>9500 MPa</td>
</tr>
</tbody>
</table>

Table 11: CLT Properties (CLT Design Guide – Structurlam 2012)
The yield strength and ultimate strength of the steel specimens were 300 MPa and 410 MPa respectively. The modulus of elasticity was taken to be 210 GPa. The following sections of 1.8 m in length were used for beam embedment into the CLT Panel:

1)  W 150 x 100 (6” x 4” Wide Flange I-Beam)

2)  HSS 100 x 50 x 3.125 (4” x 2” x 0.125” Hollow Rectangular Beam)

### 3.2.2 Specimen Description

The slots, into which the beam sections were embedded, were pre-cut at a uniform spacing of 610 mm (2’) in the CLT panel. The beams embedded into these slots were held in place using two 9.5 mm (3/8”) lag bolts in 12.7 mm (½”) drill holes, at the locations, 152.4 mm (6”) from each side of the panel (Figure 21). The force transfer through bearing of bolts was assumed to be negligible. Complete load transfer occurred through the bearing of steel beams alone.

![Figure 21: Typical Beam Embedment Procedure](image-url)
Five series of tests were conducted with two replicates of monotonic tests and one cyclic test for each series (Table 12).

<table>
<thead>
<tr>
<th>Series</th>
<th>Profile</th>
<th>Embedment</th>
<th>Embedment Length</th>
<th>Reduction in Cross Section</th>
<th>Bolted Connection (Lag Bolts Details)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Wide Flange</td>
<td>57.2 mm</td>
<td>914.4 mm (3’)</td>
<td>None</td>
<td>2 Bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2.25”)</td>
<td></td>
<td></td>
<td>9.5 mm (3/8”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>95.2 mm (3.75”) long</td>
</tr>
<tr>
<td>2</td>
<td>Wide Flange</td>
<td>101.6 mm</td>
<td>914.4 mm (3’)</td>
<td>None</td>
<td>2 Bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4”)</td>
<td></td>
<td></td>
<td>9.5 mm (3/8”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>95.2 mm (3.75”) long</td>
</tr>
<tr>
<td>3</td>
<td>Wide Flange</td>
<td>101.6 mm</td>
<td>914.4 mm (3’)</td>
<td>Yes</td>
<td>2 Bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4”)</td>
<td></td>
<td></td>
<td>12.7 mm (1/2”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>152.4 mm (6”) long</td>
</tr>
<tr>
<td>4</td>
<td>HSS</td>
<td>50.8 mm</td>
<td>914.4 mm (3’)</td>
<td>None</td>
<td>2 Bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2”)</td>
<td></td>
<td></td>
<td>9.5 mm (3/8”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>95.2 mm (3.75”) long</td>
</tr>
<tr>
<td>5a</td>
<td>HSS</td>
<td>50.8 mm</td>
<td>304.8 mm (1’)</td>
<td>None</td>
<td>1 Bolt</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2”)</td>
<td></td>
<td></td>
<td>12.7 mm (1/2”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>152.4 mm (6”) long</td>
</tr>
<tr>
<td>5b</td>
<td>HSS</td>
<td>50.8 mm</td>
<td>609.6 mm (2’)</td>
<td>None</td>
<td>1 Bolt</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2”)</td>
<td></td>
<td></td>
<td>12.7 mm (1/2”) diameter</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>152.4 mm (6”) long</td>
</tr>
</tbody>
</table>

Table 12: Component Test Specimen Description
Series 1 was conducted by embedding the wide-flange I-section 57.2 mm (2.25”) into the outer 1.5 plies of the panel (Figure 22a). Series 2 was conducted by embedding the section 101.6 mm (4”) into the outer three plies in order to study the effect of increase in embedment area on wood crushing and component performance (Figure 22b).

![Partial Embedment – Series 1](image1) ![Full Embedment – Series 2](image2)

**Figure 22: Partial and Full Embedment of Wide-Flange I-section (Series 1 and Series 2)**

Series 3 was conducted on the same wide flange I-section embedded fully into the panel with cross-section area reduced to 84 % of the original area, which leads to reduction in moment of inertia about the strong axis by 25 %. The top and bottom flange of the beam were reduced by 25.4 mm (1”), that is, 12.7 mm (1/2”) on both the sides of the flanges (Figure 23). This reduction was made using a smooth curve that extended from the point of panel - beam intersection up to 101.6 mm (4”) to avoid the creation of stress concentration resulting from a sudden change in cross-section.
Series 4 was conducted using hollow steel sections (HSS) embedded 50.8 mm (2”) into the outmost 1.5 plies to study the effect of beam profile (Figure 24). Series 5 was conducted on a HSS section with similar configuration but with reduced length of beam embedment. The embedded beam extended up to 1/3 of the width of the panel, that is, 304.8 mm (1’). A single lag bolt of 12.7 mm diameter (1/2”) was used at the center of the embedded portion to hold the beam in place (Figure 25).
Figure 24: Full Embedment of HSS (Series 4)

Figure 25: Full Embedment of HSS with Reduced Embedment Length (Series 5a)
3.2.3  Test Procedure

The panels were bolted down to the floor at both ends to restrain them from translation, rotation or uplift movement during the experiments. Six linear variable differential transformers (LVDTs) were attached to the embedded beam at a spacing of 304.8 mm (12”) with the first LVDT placed 152.4 mm (6”) from the edge of the panel (Figure 26). For series 5, LVDT 3 was placed at the center of the embedded portion.

![Figure 26: Typical Setup and Instrumentation](image)

Figure 26: Typical Setup and Instrumentation
The load was applied at the end of the projecting beam through a calibrated MTS - Actuator (225 kN capacity). The loading was maintained constant at a rate of 12.7 mm/min (0.5”/min, equivalent to a rotation of the beam at 0.017 rad/min). The loading for the monotonic tests was continued until the applied load dropped to 70 % of peak load. The displacement at 80 % of the maximum load from monotonic tests was chosen as the target displacement (100 % displacement) for the subsequent cyclic tests.

The CUREE protocol (Krawinkler et al. 2001) was used for the cyclic loading for each test series (Figure 27). The loading was programmed to continue with an increment of 20 % beyond target displacement until 160% of the target displacement. The cyclic tests were conducted at a loading rate of 5 mm/min (0.2”/min, equivalent to a rotation of the beam at 0.007 rad/min).

Figure 27: Cyclic Loading – CUREE Protocol
Chapter 4: RESULTS AND DISCUSSION

4.1 Material Level – Embedment Tests

The purpose of the embedment (bearing) tests was to evaluate how embedment strength of CLT determined according to the standard procedure can provide input data for designing the FFTT system where square cross-sections are bearing on the cross layers of CLT.

In Series 1, the CLT specimens were loaded perpendicular to grain direction of 2 layers and parallel to the grain direction of 1.5 layers, under the bearing of circular dowel. The average circular-dowel bearing strength of the CLT specimens was determined to be 21.6 MPa with coefficient of variation of 10.1 % (Figure 28).

![Figure 28: Series 1-Embedment Behaviour of CLT under the Bearing of Circular Steel Dowel](image)
In Series 2 and 3, tests were conducted with the load applied through square profiles. Series 2 specimens consisted of 2 layers with the grain direction perpendicular and 1.5 layers parallel to the loading direction, while Series 3 specimens had 2 layers with the grain direction parallel and 1.5 layers perpendicular to the loading direction. The average embedment strength of CLT for Series 2 and Series 3 was determined to be 23.1 MPa and 26.8 MPa, respectively. The coefficients of variation for Series 2 and Series 3 were 11.9 % and 11.3 %, respectively. The compression behaviour of CLT under the bearing of square profile is presented in Figure 29.

![Graph showing compression behaviour of CLT](image)

(a) Series 2 - Perpendicular to the Majority Grain Orientation
Figure 29: Compression Behaviour of CLT under the Bearing of Square Steel Rod

The elastic stiffness of CLT, the slope of the load-deformation curves for each series within the range of 10% to 40% of the maximum load, was evaluated. The summary of the embedment test results are summarized in Table 13.

<table>
<thead>
<tr>
<th>Series</th>
<th>Embedment Strength (MPa)</th>
<th>Coefficient of Variation (%)</th>
<th>Elastic Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.6</td>
<td>10.1</td>
<td>35120</td>
</tr>
<tr>
<td>2</td>
<td>23.1</td>
<td>11.9</td>
<td>36980</td>
</tr>
<tr>
<td>3</td>
<td>26.8</td>
<td>11.3</td>
<td>61,450</td>
</tr>
</tbody>
</table>

Table 13: Bearing Strength Test Results - Summary
4.2 Component Level – Monotonic Tests

4.2.1 Series 1- Partial Embedment of I-Section

During Test 1, 3.2 mm (1/8”) diameter lag bolts (152 mm long) in a 12.7 mm (½”) drill hole were used to hold the beam in the panel slot. The outermost bolt withdrew slightly after the system reached the peak load and uplift was seen at the end of the beam. During Test 2, 9.5 mm (3/8”) lag bolts (100 mm long) were used and the actuator was anchored down to restrain the uplift of the beam. Beam yielding occurred at the panel-beam interface at the exposed side of the top flange (Figure 30) at an average load of 40 kN. Maximum loads of 43.8 kN and 45.8 kN were achieved during Test 1 and Test 2, respectively.

![Image of beam yielding](image)

Figure 30: Beam Yielding – Partially Embedded I-Section – Series 1

The post-yield behaviour for each test varied due to difference in testing conditions (different bolts and actuator anchorage). The uplift at the end of the beam during Test 1 reduced the in-plane deformation beyond yielding and displayed a steeper negative slope curve beyond peak
The deformation at the end of the beam at yield and peak load was 20.7 mm and 29.5 mm with displacement of 57.1 mm at 80% of the peak load. On the other hand, uplift restraint during Test 2 allowed for higher in-plane deformation, 25.4 mm at yielding and 44.7 mm at the peak load, respectively. Higher deformation capacity without significant loss of load carrying capacity; that is, a flatter post-yield curve was obtained. Test 2 was, however, terminated at a drop to 90% of peak load due to slippage of the anchor, which led to sudden uplift of the beam and cracking in the panel. LVDT 6 corresponds to the LVDT attached to the far end of the cantilever portion while LVDT 4 corresponds to the LVDT closest to the interface.

![Figure 31: Load-Deformation Curves - Cantilever Portion - Series 1](image)

The in-plane deformation of the embedded portion of the beam was measured at three locations. LVDT 1 corresponds to the LVDT attached at the far end of the embedded beam section. Since
yielding of the beam occurred at the interface, the post-yield deformation rate at LVDT 4, the location closer to the interface on the cantilever side of the beam, was found to be higher than that of LVDT 3, the location closer to the interface on the embedded side of the beam. Therefore, the beam section caused minimal crushing in the wood and most of the deformation capacity of the system was obtained through bending of the cantilever side of the beam. The curves in Figure 32 indicate higher in-plane deformation during Test 2 due to actuator anchorage.

![Figure 32: Load-Deformation Curves - Embedded Portion - Series 1](image)

The data acquired at LVDT 1 during Test 1 was erroneous due to withdrawal of lag bolts during the experiment. However, by comparing the results obtained from LVDTs 2 and 3, it was clear that the beam rotated about a point in between LVDT 2 and LVDT 3. Slot fabrication imperfections (e.g. slight spaces between beam and wood) that existed prior to the testing
resulted in discrepancies between the LVDT 2 readings; but the stiffness of both curves was similar, with deformation at peak load less than 1 mm in both cases. By comparing the deformations at LVDT 1, LVDT 2 and LVDT 3 from Test 2 at peak load (-4.56 mm, -0.01 mm and 5.94 mm respectively), it can be inferred that the point of rotation was slightly to the right of LVDT 2, which is 457.2 mm (18”) from the embedded corner.

The in-plane deformation of the beam caused negligible wood crushing, even at the interface. However, the tendency of the beam to uplift beyond yielding resulted in pull-out of the wood material below the embedded portion of the beam near the wall-beam interface (Figure 33).

Figure 33: Wood Damage below the Partially Embedded Beam Section - Series 1

4.2.2 Series 2 - Full Embedment of I-Section

Series 2 was conducted on fully embedded I-sections. Beam yielding occurred at the panel-beam interface at the exposed side of the top flange at an average load of 41 kN and the average maximum load attained was 45.4 kN (Figure 34). The actuator was not restricted from lifting up and hence significant uplift (94 mm) was seen at the end of the tests.
The behaviour along the cantilever portion of the beam was similar for both tests except for some beam sliding during Test 2 because of slot fabrication imperfections (Figure 35). The average deformation at the end of the beam at yield load and maximum load was 18.7 mm and 26.5 mm respectively.
The in-plane deformations of the embedded portion of the beam measured at three locations are presented in Figure 36. The board to which LVDTs were attached slipped at a load drop equal to 75% of peak load. Hence the data from LVDT 1 and LVDT 2 beyond this point were discarded.

![Figure 36: Load-Deformation Curves - Embedded Portion - Series 2](image)

There exists good agreement in the behaviour displayed at the location closer to the interface (LVDT 3) between both tests, after ignoring the beam sliding during Test 2 due to slot imperfections. The deformation at the location of LVDT 3 is approximately zero at yield and peak load, which indicates that the point of beam rotation was at LVDT 3 which is 152.5 mm (6”) from the interface. The load-relationship data obtained at LVDT 1, was unexpected since beam bending in double-curvature did not occur during the experimentation. The LVDTs were attached to the flange of the beam by a set of magnets, that is, the data obtained represents the movement of the top flange of the beam, and not the center of the beam (web). Therefore, beam twisting and uplift at the flange level was captured by the LVDTs in addition to in-plane movement of the embedded beam and it was impossible to separate these effects afterwards. This explains the discrepancies in the test results obtained at LVDT 1 and LVDT 2.
Since the actuator was not anchored for test Series 2, no significant wood pull-out occurred beneath the embedded beams but a crack at the layer below the embedded beam due to beam twisting and uplift was seen (Figure 37). Lag bolts displayed adequate uplift capacity and no withdrawal of the bolts was observed. Due to high embedment strength of CLT, negligible crushing was observed in the panel and at the interface.

4.2.3 Series 3- Full Embedment of I-Section with Reduced Cross-Section

Series 3 was conducted on fully embedded wide flange I-sections with reduced cross-section near the beam-panel interface. The lag bolts used were of 12.7 mm (1/2”) diameter instead of 9.52 mm (3/8”) lag bolts used during Series 1 and Series 2. The beam yielding occurred at the point of cross-section reduction at the exposed side of the top flange (Figure 38) at an average load of 44.5 kN and the displacement at the end of the cantilever was 25.4 mm.
The behaviour along the cantilever portion of the beam during both tests was in good agreement with each other in the elastic range (Figure 39). The post-yield behaviour of the specimens was different even though the testing conditions were not varied.
During Test 1, the cantilever end of the beam started lifting upwards after the beam had yielded near the interface. The maximum load reached was 44.6 kN and a total uplift of 156 mm was seen at the end of the test. However, the beam did not uplift at the onset of beam yielding during Test 2 resulting in higher in-plane deformation compared to Test 1. The in-plane deformation at the cantilever end at maximum load during Test 1 and Test 2 were 27.3 mm and 66 mm respectively. Beam uplift of 117 mm occurred only after the maximum load of 50.2 kN was attained.

There exists good agreement in the behaviour displayed at the locations of LVDT 1 and LVDT 2 between both tests. The deformation at the location of LVDT 2 was approximately zero at the yield and peak load, which indicates that the point of beam rotation was at LVDT 2. But the data obtained from LVDT 3 (closer to the interface) during Test 1 exhibited inconclusive fluctuations (Figure 40).

![Figure 40: Load-Deformation Curves - Embedded Portion - Series 3](image-url)
No wood pull-out occurred beneath the embedded beams but a crack at the layer below the embedded beam was seen due to the uplift of cantilever end of the beam (Figure 41). The lag bolts displayed adequate uplift capacity and no withdrawal of the bolts was observed. Due to high embedment strength of CLT, wood crushing did not occur in the panel or at the interface.

![Figure 41: The Beam Uplift and Wood Cracking below the Embedded Beam - Series 3](image)

4.2.4 Series 4- Full Embedment of Hollow Section

Series 4 of monotonic tests was conducted on HSS profiles fully embedded in both depth and length, without anchoring down the actuator to resist uplift of the beam (Figure 42a). During Test 1, LVDT 5 and LVDT 6, attached towards the end of the beam slipped (Figure 42b) at a deformation of 76 mm. This slippage, however, occurred well beyond the point of beam yielding. During Test 2, these two LVDTs were replaced by string pots (Figure 42c), capable of measuring deformation up to 760 mm to accurately record the load-deformation relationship along the length of the beam.
Figure 42: Monotonic Testing – Hollow Beam Section Instrumentation – Series 4
The yielding of the beam occurred at the beam-wall interface at an average load of 17 kN. Due to uniform bearing of the hollow section on the CLT panel, the yielding occurred symmetrically on both faces (Figure 43). No uplift, wood crushing and withdrawal of the lag bolts were observed.

Figure 43: Beam Yielding of Fully Embedded HSS – Series 4
Due to slippage of LVDT 5 and LVDT 6, data points beyond 76 mm (3”) deformation are not represented in Figure 44. The results from the two tests are in good agreement with each other. The average maximum load attained was 18.5 kN.

![Figure 44: Load-Deformation Curves - Cantilever Portion - Series 4](image)

The load-displacement relationships at locations on the embedded portion of the beam are presented in Figure 45. A slight shift in the load-deformation curve of LVDT 3 during Test 2 can be traced back to the existing fabrication imperfection at that location (0.7 mm) prior to the testing that led to beam sliding.
The data obtained from the LVDTs in the embedded portion of the beam suggests that the beam rotated about a location between LVDT 2 and LVDT 3. Deformation at LVDT 3 was found to be approximately zero until the point of yielding. This suggests that the point of rotation is closer to the location of LVDT 3 which is 152.5 mm (6”) from the interface.

**4.2.5 Series 5- Full Embedment of Hollow Section with Reduced Embedment Length**

Series 5 was conducted on HSS profiles fully embedded in depth and partially embedded along the length of the beam (width of the panel). The yielding of the beam occurred symmetrically on both faces at the beam-panel interface (Figure 46) at an approximate average load of 17.1 kN.
The data obtained during the two tests from the four LVDTs are in good agreement with each other (Figure 47). An average displacement of 59.4 mm was seen at the end of the beam at an average maximum load of 18.5 kN. Neither beam uplift nor withdrawal of lag bolt was seen.
The beam was found to rotate about the location of the lag bolt (LVDT 3) as seen in Figure 48a. No wood crushing occurred around the embedded beam in the end grain of CLT (Figure 48b). But wood deformation of 10.1 mm was recorded at the interface in the layer with grain orientation perpendicular to the load (Figure 48c, d) while 4.2 mm deformation was recorded in the end grain.

![Image](image1.png)

(a) Rotation about the center of embedded portion

![Image](image2.png)

(b) High Resistance of End grain

![Image](image3.png)

(c) Interface before the test

![Image](image4.png)

(d) Damage at the interface after the test

Figure 48: Wood Damage from HSS with Reduced Length of Embedment - Series 5
4.3 Component Level – Cyclic Tests

4.3.1 Series 1- Partial Embedment of I-Section

Due to the difference in testing conditions between the two monotonic tests and the termination of Test 2 at a drop of 10% of the peak load, the displacement at 80% of peak load was approximated to be equal to 76.2 mm (3”). This was used as the target displacement for the cyclic tests. The actuator was anchored down to resist beam uplift (similar to Test 2 of the monotonic test series). The mode of failure was pull-out of the beam from the embedded portion along with major chunks of wood below (Figure 49a, 49b) instead of yielding at the interface. The lag bolts withdrew and the beam was completely lifted from the panel (Figure 49c). A significant amount of wood damage was incurred due to sudden beam lifting out-of-plane and twisting action in the beam (Figure 49d, 49e). The lag bolts and the beam began withdrawing from the embedded position before the onset of beam yielding at interface at a load approximately equal to 40 kN. The loading was continued only until 70 % of the target displacement, with 34 load cycles in total, at which point complete beam withdrawal from the panel occurred.

(a) Uplift from Embedded End  
(b) Uplift at the Interface
The maximum load was 44.5 kN. The hysteresis behaviour of the beam in the embedded portion is presented in Figure 50. Based on the monotonic and cyclic tests, it can be deduced that the point of rotation of the beam is adjacent to the location of LVDT 2. The deformation at this location was found to be zero, with negligible energy dissipation. Hysteresis plots at the locations of LVDTs 4, 5 and 6 suggest almost all energy under cyclic loading was dissipated through the deformation of the cantilever portion of the beam (Figure 51).
Figure 50: Cyclic Test - Series 1 – LVDT 1 (Top), LVDT 2 (Center) and LVDT 3 (Bottom)
Figure 51: Cyclic Test - Series 1 – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.2 Series 2 - Full Embedment of I-Section

4.3.2.1 Test 1

The cyclic test on the fully embedded I-section was conducted without restricting the beam uplift from the cantilever end. The average displacement at 80% peak load from monotonic tests, 64.7 mm (2.55”), was considered as the target displacement for cyclic loading protocol. The cyclic load was continued until 38 load cycles, thus achieving 120% of target displacement and a maximum load of 48 kN. Since the actuator was not anchored down, the beam yielding caused beam uplift at the cantilever end and then, complete pull-out of the beam from the embedded position occurred about the interface (Figure 52a). Nevertheless, the pull out of the embedded portion of the beam did not occur until the beam had fully yielded (Figure 52b). Minimum amount of wood damage (less than 5 mm) at the interface due to in-plane deformation was incurred; however, wood pieces were pulled out from beneath the embedded portion along with the beam (Figure 52c) and significant wood damage was seen below the embedded beam (Figure 52d).
The major concern with embedment of wide-flange I-section was out-of plane buckling of the beam section from the face of the panel. Figure 53 and Figure 54 illustrate the hysteresis behaviour of the beam section. The behaviour of the embedded portion of the beam captured at the location of LVDT 1 does not truly represent the in-plane deformation of the beam; rather the combination of the in-plane deformation and the twisting action of the flange. Due to this discrepancy in both, monotonic and cyclic tests, a conclusive result on the point of rotation of the beam was difficult. Most of the energy dissipation occurred due to the yielding at the interface and translational deformation of the free end portion of the beam.
Figure 53: Cyclic Test - Series 2a – LVDT 1 (Top), LVDT 2 (Center) and LVDT 3 (Bottom)
Figure 54: Cyclic Test - Series 2a – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.2.2 Test 2

A second cyclic test was conducted by replacing the two 95.2 mm (3.75”) long lag bolts of diameter of 9.5 mm (3/8”) by 152.4 mm (6”) long lag bolts of 12.7 mm (1/2”) diameter in order to resist the beam from pulling out of the panel. The test was conducted without restricting beam uplift at the cantilever end. The target displacement considered for cyclic loading protocol was 64.7 mm (2.55”). The cyclic load was continued until 37 load cycles, achieving 100 % of the target displacement and the maximum load attained was 52.1 kN. A significant uplift of approximately 150 mm was seen at the cantilever end of the beam (Figure 55a ) and the lag bolt closest to the interface withdrew as the beam yielded and lifted from its embedded position (Figure 55b). The embedded end of the beam, however, was held in place with the second lag bolt resisting the complete beam pull-out from the end (Figure 55c). Wood damage of 7.6 mm (0.3”) was incurred at the interface due to in-plane deformation. However, slight wood cracking was seen at the layers beneath the beam as a result of beam lifting off the panel near the interface (Figure 55d,e).

(a) Uplift at the Cantilever End of the Beam  
(b) Lag Bolt Withdrawal
The hysteresis behaviour of the embedded beam was similar to that of Test 1. Since the beam was resisted from significant pull-out and twisting action near the embedded end by means of larger lag bolts, the data obtained from LVDT 1 was a better representation of the embedment behaviour. The point of rotation of the beam, based on this test, can be inferred to be in between LVDT 1 and LVDT 2 (Figure 56). Most of the energy dissipation occurred due to the yielding at the interface and deformation of the free end portion of the beam (Figure 57).
Figure 56: Cyclic Test - Series 2b – LVDT 1 (Top), LVDT 2 (Center) and LVDT 3 (Bottom)
Figure 57: Cyclic Test - Series 2b – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.3 Series 3- Full Embedment of I-Section with Reduced Cross-Section

The cyclic test on the fully embedded beam with reduced cross-section was conducted without restricting beam uplift from the cantilever end. Two lag bolts of 12.7 mm (1/2”) diameter and 152.4 mm (6”) in length were used to keep the embedded beam in place and provide resistance to the embedded beam from pulling-out of the panel (similar to cyclic Test 2 of Series 2). The average displacement at 80 % peak load from monotonic tests, 72.6 mm (2.86”), was considered as the target displacement for cyclic loading protocol. The cyclic load was continued until 36 load cycles, achieving 100 % of target displacement and a maximum load of 43.9 kN. The behaviour was similar to the cyclic Test 2 of Series 2. A significant uplift of approximately 300 mm was observed at the end of the cantilever portion of the beam while the embedded end of the beam was held in place without any beam pull-out (Figure 58a). The lag bolt closer to the interface did not display adequate withdrawal capacity and slight uplift of the beam was seen at the interface (Figure 58b). The beam yielding occurred at the point of cross-section reduction and negligible wood damage (less than 4 mm) was incurred due to in-plane deformation of the beam and beam uplift near the interface (Figure 58c).

(a) Uplift at the Cantilever End of the Beam
The hysteresis behaviour of the embedded beam is presented in Figures 59 and 60. Due to higher accuracy of the data obtained from LVDT 1 during this test, the point of rotation of the beam can be concluded to be in between LVDT 1 and LVDT 2. The monotonic and cyclic results confirm the location to be close to LVDT 2. As previously noted, most of the energy dissipation occurred due to the yielding at the interface and deformation of the free end portion of the beam.

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(b) Withdrawal of the embedded beam and lag bolt near the interface

(c) The beam yielding and wood damage at the interface

Figure 58: Failure of Reduced Wide Flange I-Section under Cyclic Loading - Series 3
Figure 59: Cyclic Test - Series 3 – LVDT 1 (Top), LVDT 2 (Center) and LVDT 3 (Bottom)
Figure 60: Cyclic Test - Series 3 – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.4 Series 4- Full Embedment of Hollow Section

The average displacement at 80% peak load during monotonic test was 126 mm (4.95”), which was taken as the target displacement for the cyclic load (100% displacement). The maximum load achieved was 18.4 kN. The beam yielding occurred at the interface at load approximately equal to 17 kN. The loading continued up to 140% of target displacement with 42 cycles in total, after which complete failure occurred, with the beam tearing into two pieces at the point of yielding (Figure 61).

Figure 61: Failure of Fully Embedded HSS under Cyclic Loading - Series 4
The beam was securely held in place by means of the lag bolts throughout the experiment without any uplift at the end of the beam. The section did not display twisting action due to its symmetry and full embedment in the panel. The failure was purely ductile and higher deformation capacity was displayed by the hollow section than by the wide flange I-section. Crushing of the wood at the interface was as low as 4.2 mm as a result of uniform bearing of the section. No major wood damage was incurred as witnessed in the Series 1 and 2.

The hysteresis curves at the locations of the LVDTs attached to the embedded portion of the beam (Figure 62) illustrate that minimal energy dissipation occurred through the deformation of embedded portion of the beam. The ductility in the system under cyclic loads was contributed by the bending of the beam at the interface (Figure 63). The inference made based on the monotonic tests on the point of rotation of the beam being in between LVDT 2 and LVDT 3 was successfully confirmed by the cyclic test results. The deformation and energy dissipation at the location of LVDT 3 was negligible indicating that the point of rotation is very close to LVDT 3.
Figure 62: Cyclic Test - Series 4 – LVDT 1 (Top), LVDT 2 (Center) and LVDT 3 (Bottom)
Figure 63: Cyclic Test - Series 4 – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.5  Series 5- Full Embedment of Hollow Section with Reduced Embedment Length

4.3.5.1  Series 5a – One - Third Embedment Length

The average displacement at 80 % peak load during the monotonic tests was 174.5 mm (6.87”), which was taken as the target displacement (100 % displacement) for the cyclic load. The loading continued up to 140 % of target displacement with 42 load cycles in total, after which the load dropped to almost zero and the beam tearing was initiated at the point of yielding near the interface (Figure 64).

![Figure 64: Failure of the Fully Embedded HSS under Cyclic Loading - Series 5a](image1)

The beam was held in place using the 152.4 mm (6”) long lag bolt of 12.7 mm (1/2”) diameter. No uplift of the beam near the interface or at the cantilever end of the beam was observed. Slight damage around the embedded portion of the beam was incurred due to in-plane deformation of the beam. The beam yielded uniformly on both faces due to uniform bearing and no twisting action was witnessed. The maximum load attained was 17.5 kN.
Based on the observations and the hysteresis behaviour obtained at LVDT 3, it can be concluded that the beam rotated about the pinned connection and the beam yielding occurred at the interface. The deformation and energy dissipation was negligible at this location (Figure 65).

![Figure 65: Beam Rotation (Left) and Hysteresis Behaviour at LVDT 3 (Right) – Series 5a](image)

Rotation of the beam about the pin connection and yielding at the interface caused non-uniform wood crushing at the end of embedded beam portion and at the interface, respectively (Figure 66a, b). The beam crushed 30.5 mm (1.2”) into the CLT panel at the interface in the layer with grain orientation perpendicular to the applied load (second layer), while 15.2 mm (0.6”) of wood crushing was seen in the end grain (first layer) of the panel (Figure 66c). The measured wood damage at the end of the embedded portion of the beam (Figure 66d) was 8.7 mm (0.34”).
The smooth hysteresis curves obtained at the locations of the LVDTs attached to the cantilever portion of the beam are presented in Figure 67. These curves illustrate that the entire ductility in the system of this configuration was contributed by the yielding and bending of the beam at the interface.
Figure 67: Cyclic Test - Series 5a – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.3.5.2 Series 5b – Two - Third Embedment Length

A second cyclic test of Series 5 with the embedment length of 609.6 mm (2’), that is, twice the embedment length of the first cyclic test (Series 5a), was conducted in order to establish an approximate limit on the embedment length for the beam along the width of the panel that initiates the desired failure mechanism. The beam was bolted into the slot using a 152.4 mm (6”) long lag bolt of 12.7 mm (1/2”) diameter at the same location as Series 5a (location of LVDT 3). Two LVDTs (LVDT 2 and LVDT 3) were attached to monitor the behaviour of the embedded portion of the beam (Figure 68).

![Figure 68: Full Embedment of HSS with Reduced Embedment Length - Series 5b](image)

The target displacement (100 % displacement) considered for the cyclic load was 150.1 mm (5.91”), interpolated between the target displacements of Series 4 and Series 5a. The loading
continued up to 120% of target displacement, with 40 load cycles in total, after which the load dropped to almost zero. Beam tearing was initiated at the point of yielding near the interface (Figure 69).

Figure 69: Failure of the Fully Embedded HSS under Cyclic Loading - Series 5b

No uplift of the beam near the interface or at the cantilever end of the beam was observed. The beam yielded uniformly on both faces due to uniform bearing and no twisting action was witnessed. Minimal wood damage was incurred at the interface due to the in-plane deformation of the beam. Wood crushing of less than 2 mm (0.08”) was seen in the first layer of the panel while 9.7 mm (0.4”) was seen in the second layer. No wood damage at the end of the embedded portion of the beam was observed.

The maximum load attained was 19.95 kN. The hysteresis curves obtained by the LVDTs attached to the embedded portion of the beam (Figure 70), suggest that the beam rotated about a point located in between the LVDT 2 and LVDT 3. Due to lack of monotonic test replicates, no conclusive comments can be made on the location of beam rotation.
The hysteresis curves obtained at the locations of the LVDTs attached to the cantilever portion of the beam are presented in Figure 71. LVDT 4 slipped off its position and it was adjusted back to the position during the progress of the test. This explains a slight jump in the hysteresis curve of LVDT 4.
Figure 71: Cyclic Test - Series 5b – LVDT 4 (Top), LVDT 5 (Center) and LVDT 6 (Bottom)
4.4 Discussion of Results

4.4.1 Material Level

The embedment strength of CLT under two loading cases (round and square profile) and two majority grain orientations were compared to evaluate how accurately the strength values of CLT under the bearing of a dowel can provide input data for designing the FFTT system, where load transfer occurs through the bearing of square beam profiles. The average stress-strain curve for each test series presented in Figure 72 shows that CLT exhibits higher embedment strength under the bearing of a square profile compared to a circular dowel.

![Figure 72: Material Testing – Average Embedment Behaviour of CLT](image-url)
4.4.1.1 Influence of Bearing Profile on Embedment Strength

The difference in the cross-section of the bearing profile would result in different stress distribution pattern in the panel. The applied compression load on the dowel gets distributed into the panel along the circumference of the embedded dowel, causing slight shear stress component in addition to the compression stress. The stress distribution in the case of bearing under square rod, however, occurs predominantly through the compression stress (Figure 73).

![Figure 73: Influence of Bearing Profile on Stress Distribution](image)

This may have resulted in lower dowel-bearing strength of CLT compared to the strength under the bearing of square profile. The average dowel-bearing strength of CLT under loading perpendicular to the majority grain direction was 21.6 MPa, 6% lesser than the strength under the bearing of square profile (23.1 MPa) with same orientation (Figure 74).
ANOVA (Analysis of Variance) was carried out to investigate if the difference between the dowel-bearing (Series 1) and rod-bearing (Series 2) strengths was significant at an alpha level ($\alpha$) of 0.05. The F-Statistic Value ($F$-value) defined as the ratio of variation between the two series to the variation within the series, was 2.9 which was lower than the critical F-Value ($F_{critical}$) of 4.2. The $P$-value was 0.1 (greater than $\alpha$-value). This confirmed that the influence of bearing profile was not significant and hence the results from dowel-bearing test (ASTM D 5764-97a) could be used as the input data for the design of FFTT system with sufficient accuracy.

4.4.1.2 Influence of Grain Orientation on Embedment Strength

The embedment strength evaluated based on the equations [1] and [2], for the given configuration of 25.4 mm (1”) dowel and specific gravity ($G$= 0.41) was 15.3 MPa and 6.7 MPa
in the parallel and perpendicular direction respectively. This indicates a reduction of more than 50% in the embedment strength in the perpendicular direction. According to the characteristic dowel-bearing strength equation [3], the embedment strength of CLT was 23.3 MPa with majority grain oriented parallel to the loading and 21.1 MPa in perpendicular direction for the same configuration. These strength values are in good agreement with the experimental results. The embedment strength of CLT under the bearing of square steel profile with grain orientation of 67 mm parallel (53 mm perpendicular) to the applied load was 26.8 MPa while the embedment strength with majority grain oriented perpendicular to the load was 23.1 MPa (Figure 75).

![Embedment Strength Distribution for Parallel and Perpendicular Grain Orientation](image)

**Figure 75: Embedment Strength Distribution for Parallel and Perpendicular Grain Orientation**
The cross laminations increase the strength of CLT in both directions but the difference in the embedment strength in two grain orientations is still significant. This was concluded based on the ANOVA results which produced $F$-value (12.5) greater than $F_{crit}$ (4.2). The $P$-value was 0.0014, which was lower than $\alpha$-value of 0.05.

### 4.4.2 Component Level

The monotonic and cyclic test results for all series are summarized in Table 14. The ultimate deformation at the end of the beam is defined as the deformation at load equal to 80 % of the peak load (“near-collapse” state). The maximum energy dissipated was calculated based on accumulated energy dissipation from each loading cycle due to the deformation at the end of the beam (cantilever end). The cyclic tests were terminated when the system failure occurred or when the applied load dropped to zero.

<table>
<thead>
<tr>
<th>Series</th>
<th>MONOTONIC TESTS</th>
<th>CYCLIC TESTS</th>
</tr>
</thead>
<tbody>
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<td>Peak Load (kN)</td>
<td>Ultimate Deformation (mm)</td>
</tr>
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</tr>
<tr>
<td>2-1</td>
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<tr>
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<td>-NA-</td>
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<tr>
<td>5 B</td>
<td>-NA-</td>
<td>-NA-</td>
</tr>
</tbody>
</table>

Table 14: Summary of Monotonic and Cyclic Test Results (Series 1 - Series 5)
4.4.2.1 Load-Carrying Capacity

The effect of embedment depth of the beam on the behaviour of the system was analyzed by comparing the capacities of Series 1 and Series 2 (where embedment depth was increased by 2 times). This increase in embedment depth marginally increased the load carrying capacity of the system by 1.5% while the cross-section reduction in Series 3 further increased the capacity by 4.4%. No considerable variation in the capacity of hollow-sections with different embedment lengths (Series 4 vs. Series 5) was observed. The beam profile, however, had significant effect on the load-carrying capacity of the system. The wide flange I-sections were about 2.5 times stronger than the hollow rectangular sections (ratio of moment of inertia of the wide flange I-section to the moment of inertia of the hollow section was 5.7). The increase in the load beyond yield load was less than 12% for the wide flange I-sections while the increase was as low as 8% for hollow sections. The peak loads of the cyclic tests were roughly equal to the peak loads of the monotonic tests. The load carrying capacity of five configurations of steel embedment in the CLT panel under monotonic and cyclic loading is illustrated in Figure 76.

![Graph showing load carrying capacity for Series 1 to Series 5 with yield load, peak monotonic load, and peak cyclic load marked.]
4.4.2.2 Post-Yield Behaviour

The load-displacement relationships obtained from the LVDTs show that the wide flange I-sections were not only stronger but also 3 times stiffer than the hollow sections (the moment of inertia of the wide-flange I-section was approximately 6 times the moment of inertia of the hollow section). The hollow sections, however, possessed higher deformation capacity. They also retained a major proportion of their load-carrying capacity beyond the point of yielding and displayed a flatter post-yield curve (indicating more system ductility). The strength degradation beyond yielding in the wide-flange I-sections was rapid (Figure 77).

![Figure 77: Monotonic Tests – Average Deformation Capacity at the Cantilever End (LVDT 6)](image)

The effect of embedment depth on the deformation capacity and post-yield behaviour was negligible. However, the deformation capacity of Series 1 increased by 82 % during the
monotonic Test 2 compared to Test 1, because of the anchorage of the actuator that restrained the beam uplift. Beam uplift of 32 mm was seen while significant uplift of about 100 mm was seen with unrestricted testing condition. This indicates that the post-yield deformation capacity of wide-flange I-sections can be considerably increased provided the beam is restrained from out-of-plane buckling.

Series 3, where the cross section of the beam was reduced by 16% near the interface, failed to facilitate the desired uniform yielding in the steel member. Yielding initiated simultaneously at the top and bottom flanges of the beam under cyclic loads, but the unexposed flange sections remained unaffected (Figure 78).

![Figure 78: Non-Uniform Yielding of Top and Bottom Flange – Cyclic Loading (Series 3)](image)

The hollow sections, on the other hand, had higher resistance against out-of-plane buckling because of uniform bearing against the panel and uniform yielding at the interface (Figure 79). The decrease in the embedment length of the beam to 1/3 of the width of the panel (decrease by 66.7%) increased the ultimate deformation capacity by approximately 40%. More tests, however, with different embedment lengths and beam placements are necessary to get a clear understanding of the effect of embedment length on the post-yield behaviour of the system.
Due to the nature of non-uniform yielding in the wide flange I-sections, the beams buckled out-of-plane at the onset of yielding at the interface and led to significantly high uplift at the cantilever end (Figure 80). The beam uplift under cyclic loading was as high as 300 mm during Series 3.
4.4.2.3  Point of Beam Rotation

The point of yielding was at the interface of the wall panel and the beam for all configurations; the point of beam rotation (within the CLT panel), however, varied between the test series. The position of beam rotation was concluded based on the load-displacement relationships of the LVDTs attached to the embedded beam section. The location at which the deformation and dissipated energy was closest to zero was called the ‘point of beam rotation’ (Figure 81). The location about which the beam rotated in Series 1 and Series 3 was near the location of LVDT 2 (457.2 mm from the interface) while the point of beam rotation for rest of the configurations (Series 2, 4 and 5) was about the location of LVDT 3 (152.4 mm from the interface). These points are of great interest in the estimation of the stress transfer between steel and timber through the bearing of the embedded beams on the wall panels.

Figure 81: Point of Beam Rotation and Yielding (Series 1 - Series 5)
4.4.2.4 Mode of Failure

Ductile failure through the yielding of the beam at the interface was initiated under monotonic loading in all test series. A “Strong – Column Weak – Beam” failure mechanism was seen in most of the configurations but out-of-plane buckling of the embedded wide-flange I-sections after yielding posed a serious challenge under cyclic loads. While the failure mode of hollow sections was mainly ductile in nature, considerable wood damage was incurred by the pull-out of the wide-flange I-sections from the face of the wall panel.

Partial embedment of sections (Series 1) resulted in high twisting action in the embedded portion of the beam displaying high uplift tendencies. The mode of failure under cyclic loads was brittle in manner. The beam completely pulled out of the panel before the beam yielded (Figure 82a) and the test was terminated before the target displacement was reached. The anchorage of the actuator was also a significant contributing factor towards the brittle failure mode.

Increasing the embedment area (depth) through complete embedment of the beam without anchorage of the actuator at the end (Series 2) increased the resistance against out-of-plane buckling of the beam to a certain extent. The beam eventually pulled out of the panel from the embedded end but this did not occur until the beam was already completely yielded (Figure 82b). Wood crushing due to the in-plane deformation of the beam was negligible at the interface. The out-of-plane buckling of the I-sections caused the majority of the wood damage.

The second cyclic test of Series 2 and the Series 3 had the lag bolts replaced by the ones with higher withdrawal capacity (larger diameter and length), which prevented the complete pull-out of the beam from the embedded end and controlled the wood damage around and beneath the
beam section. The beam buckled out-of-plane, lifting the portion from the panel near the interface while the embedded end was securely held down by the lag bolt (Figure 82c). Therefore, full-embedment of the section is more advantageous than partial embedment, provided the beam pull-out from the panel is restrained using efficient bolted connections.

Figure 82: Out-of-Plane Buckling of Wide-Flange I-Sections (Series 1- Series 3)

The failure mode of the configurations with the embedded hollow sections (Series 4 and 5) was extremely ductile. The beam yielded at the interface without displaying twisting and out-of-plane buckling. The failure mechanism of Series 4 was tearing of the section at the point of yielding
without significant wood crushing. The same ductile failure mechanism was observed in Series 5 where embedment length was decreased. However, the decrease in the embedment length increased the amount of wood deformation at the interface. The summary of the wood crushing caused by the bearing of the embedded hollow beams at the interface (averaged over the top two wood layers) is summarized in Table 15.

<table>
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<tr>
<th>Embedment</th>
<th>1/3rd Embedment</th>
<th>2/3rd Embedment</th>
<th>Full Embedment</th>
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<tr>
<td>22.9 mm</td>
<td>5.9 mm</td>
<td>4.2 mm</td>
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</tr>
</tbody>
</table>

Table 15: Wood Crushing at the Interface – Hollow Sections – Cyclic Loads

The wood damage at the interface was increased by more than 4 times when the embedment length was decreased to 1/3 of the full embedment length. Wood crushing was also observed at the embedded end of the beam section because of in-plane rotation of the beam. The cross-section of the beam was deformed at the embedded end because of high resistance of CLT end grain (Figure 83).
4.4.2.5  Hysteresis Behaviour

The comparison of hysteresis behaviour of the system close to the wood-steel interaction (interface) as well as the monotonic curves between the series is illustrated in Figure 84. The hysteresis behaviour of the embedded wide-flange I-sections of all the series were similar but the number of applied load cycles before the complete strength degradation of the system varied between the series.

Series 1 underwent the least amount of cyclic deformation before the brittle failure; the hysteresis curves stayed within the linear range for most load cycles (Figure 84a). The monotonic curve was a close representation of the envelope curve for the hysteresis curves of LVDT 3 but it was a non-conservative estimation of the envelope curve for the hysteresis at LVDT 4. The out-of-plane buckling of the cantilever side of the interface was more dominant under cyclic loads and thus, the in-plane deformation capacity of the system was greatly reduced.

The eccentricity due to the non-symmetric beam yielding in Series 2 and Series 3 resulted in beam twisting at the interface. This in turn, caused unsymmetrical deformation in the compression and tension load cycles. The beam rotation in Series 2 occurred at the location of LVDT 3, but a deformation up to 25 mm was captured by the LVDT at the tension side while the deformation was approximately zero in the compression side (Figure 84b). Here, the cyclic deformations under the load cycles after significant uplift of the beam were omitted. Furthermore, the accuracy of the deformation data obtained by the LVDTs was compromised because of the twisting and the uplift at the cantilever end of the embedded beam. The out-of-plane buckling of the beam under cyclic loads increased the rate of strength degradation between the load cycles and decreased the in-plane deformation capacity of the system (Figure 84c).
(a) Series 1 - Wide Flange – Partial Embedment – LVDT 3 (left) and LVDT 4 (right)

(b) Series 2 - Wide Flange – Full Embedment – LVDT 3 (left) and LVDT 4 (right)

(c) Series 3 - Wide Flange – Full Embedment, Reduced C/S – LVDT 3 (left) and LVDT 4 (right)
Series 4 and 5 (consisting of the hollow sections) resisted maximum number of load cycles. The hysteresis loops in the cyclic tests were large, implying a connection with adequate ductility for the desired seismic application. The dominant in-plane behaviour of the section made the system capable of undergoing larger cyclic deformations without a significant decrease in load carrying capacity. The data obtained by the LVDTs were assumed to be of sufficient accuracy (Figure 84d). The hysteresis behaviour of the beam in the embedded portion (LVDT 1, 2 and 3) was within the linear range and the nonlinearity in the system was contributed by the yielding at the interface and in-plane deformation of the cantilever portion of the beam (Series 4). The stiffness of the system was reduced when the embedment length was decreased to 1/3 the width of the panel (Series 5a) and decreasing the embedment length allowed the beam to rotate more freely in the CLT panel.
4.4.2.6 Energy Dissipating Capacity

The behaviour under cyclic loads confirmed that the initiation of plastic hinge in the embedded beam section was the most significant contributing factor towards the system ductility. The CLT panels behaved as rigid members and resisted the in-plane wood deformation under the bearing of embedded steel sections, dissipating little energy. Almost all energy was dissipated through the in-plane deformation of the overhanging portion of the steel beams. The total cumulative energy dissipation from each load cycles at the locations of the LVDTs for all the test series is presented in Figure 85. The energy dissipation was calculated based on the area under the hysteresis curves obtained from the cyclic loads.

Figure 85: Cumulative Energy Dissipation – Cyclic Loading (Series 1 - Series 5)
The total energy dissipating capability of the system was dependent on number of cyclic deformation undergone by the embedded beam sections. The beam profile (cross-section) was the most dominant driving factor for the energy dissipation capacity and hysteresis behaviour of the system. The energy dissipating capacity of hollow sections was 10% more than the capacity of wide-flange I-sections, where the in-plane deformation was hindered by the out-of-plane buckling action. The cross-section reduction (Series 3) increased the beam uplift and decreased the in-plane deformation even further. The energy dissipating capacity of Series 3 was one-quarter the capacity of Series 2 with similar embedment depth and bolted connection configuration. Similarly, partial embedment (depth) of the beam section displayed the least energy dissipating capacity due to brittle nature of the failure mode. The comparison of energy dissipating capacity at the end of the cantilever portion based on the beam placement and profile configuration for full length embedment of the sections is summarized in Figure 86a.

(a) Influence of Beam Placement and Profile Configuration
The variation in the embedment length of the beam section influenced the ductility of the system (Figure 86b). The energy dissipating capacity of Series 5b, where the embedment length was reduced to 2/3 of the length in Series 4 (full length embedment), was increased by 29% and a further increase by 3% was achieved by decreasing the embedment length to 1/3 of the full embedment length.

(b) Influence of Embedment Length

It was worth noting that the energy dissipated by the embedded beam in the panel was higher for wide-flange I-sections (Series 1 to Series 3) compared to embedded hollow section (Series 4). The twisting and uplift action of the embedded wide flange I-sections caused higher wood deformation in the panel. CLT offered higher resistance against the embedment of hollow section...
and restrained the rotation of the beam in the embedded portion. However, the decrease in the embedment length (Series 5a and Series 5b) facilitated higher rotational motion of the beam in the embedded and the cantilever portion. The participation of wood in the energy dissipating capacity of the system was increased (Figure 86c).

(c) Energy Dissipation in the CLT Panel near the Interface (LVDT 3)

Figure 86: Summary – Energy Dissipation Capacity under Cyclic Loads (Series 1 - Series 5)
Chapter 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

This research focused on the material level performance of CLT embedment and the component level performance of the proposed hybrid FFTT system under quasi-static monotonic and reversed cyclic loads.

Material level testing consisting of three series of embedment strength tests, examining the behaviour of CLT under the bearing of a dowel and a square rod, was conducted. 15 samples were tested per series and ANOVA analysis was carried out to determine if there was a significant difference in the CLT behaviour under the bearing of two different profiles. The embedment strength of CLT parallel and perpendicular to the majority orientation was also established.

The experimental program at the component level consisted of five test series; two quasi static monotonic tests and one cyclic test per series were conducted. The effects of partial embedment, cross-section reduction, embedment length and the beam profile were investigated. A full-size 7-ply CLT assembly subjected to static and dynamic loads demonstrated high connection strength while maintaining ductile performance. The embedment of steel sections in the CLT wall panels increased the deformation capacity of the structure under lateral loads and significantly increased the ductility of the system. The embedded steel beams initiated the desired ductile ‘Strong Column – Weak Beam’ failure mechanism by beam yielding (plastic hinging) at the intersection of the wood panel and the steel beam. Most of the energy dissipation
occurred as a result of the in-plane deformation of the cantilever portion of the embedded beam while the embedded portion were held in the wall panel by means of simple bolted connections.

Although the systems with embedded wide-flange I-sections were stronger and stiffer than the systems with embedded hollow rectangular sections, the out-of-plane buckling of the beam was the major setback of the embedment of wide-flange I-sections in the CLT wall panels. Unsymmetrical yielding of the top and bottom flange of the section caused eccentricity in the system resulting in the twisting of the beam and increased pull-out tendencies in the embedded section. Due to the nature of symmetry in the profile of the hollow sections, the bearing of the embedded beam against the CLT panel was uniform and symmetrical yielding of the beam at the interface ensured that the beam did not buckle out-of-plane. As a result, the system exhibited the desired post-yield behaviour, where high deformation capacity was achieved in the system without significant loss of strength under monotonic and cyclic loads during the test Series 4 and Series 5.

The load-deformation curves obtained from the current study can be used to develop the analytical model of the FFTT system for multi-storey buildings and monitor the structural behaviour under the gravity and lateral loads. The hysteresis curves can be used to define plastic hinge properties for nonlinear modeling of the system. However the results obtained by the LVDTs attached to the embedded wide flange I-sections were not highly accurate. The LVDTs were attached to the flange of the section and hence captured twisting of the beam in addition to the in-plane deformation which could not be separated at the later stage. Thus, more accurate load-deformation relationship of the wide-flange I-sections can be obtained by monitoring the movement of the web of the embedded beam section.
5.2 Conclusions

Based on the material level testing, it can be concluded that the dowel-bearing strength values can be used conservatively to design the FFTT system where the force transfer occurs through the bearing of the embedded steel beam sections. The difference between the embedment strength values for majority parallel and majority perpendicular is much smaller than that obtained from the CSA 086 equations.

Partial embedment of the wide flange I-sections in the CLT panel pulled the embedded section out of the panel before the beam yielding and shifted the mode of failure from ductile to brittle. Furthermore, partial embedment did not add any additional benefits to the system in terms of load carrying capacity, deformation capacity and energy dissipation. Therefore, full depth embedment of the beam sections is highly recommended.

A strong bolted connection was a solution for restraining out-of-plane pull out of the embedded beam from the panel but the buckling of the wide flange I-section beams at the cantilever end was inevitable. The reduction of cross-section of the beam by 16 % did not influence the load carrying capacity of the system but the beam uplift was increased by 2 times which considerably reduced the energy dissipation capacity of the system.

The cyclic tests conducted on the systems with the embedded hollow sections with three embedment lengths, confirmed that the reduction in the embedment length increased the energy dissipation capacity of the system due to the increased rotational capacity of the beam. The desired failure mechanism can be achieved up to the embedment length of 1/3 the width of the wall panel. However, the wood crushing at the interface was 5 times greater than for the series.
with full length embedment. A further decrease in the embedment length is not recommended owing to the minimization of wood crushing in the CLT panel. Therefore minimum embedment length for the hollow sections suggested is 1/3 the width of the wall panel. Further testing on the systems with shorter embedment length would confirm this limit.

The study also confirmed that the system behaviour was highly dependent on the profile (cross-section) of the embedded beam. For the construction of buildings in highly seismic regions such as coastal BC, which demands high system ductility, the embedment of hollow sections that achieves failure loads without any excessive wood crushing is preferred over the wide-flange I-sections. However, in order to satisfy the code requirements of ultimate strength and serviceability conditions, larger cross-section sizes of the embedded beam would be required due to lower inherent strength of the hollow sections compared to the wide-flange I-sections and thus, cause increase in the weight of the building. This is a trade-off between the system ductility and seismic weight considerations. Therefore, the selection of the beam profile should be made after fully comprehending the behaviour of the embedded wide-flange I-section with increased resistance against the out-of-plane buckling.

5.3 Recommendations for Future Studies

The hybridization of timber and steel at a structural level using the FFTT methodology for the construction of tall wood buildings in seismic regions shows a considerable promise. Future studies that can contribute towards practical implementation of the system include:

[1] A finite element numerical model that accurately represents the experimentally observed behaviour and optimizes the component level configuration by investigating the parameters
influencing the behaviour of the FFTT system, is recommended as the immediate step towards providing the design guidance for the system.

[2] The study can also be extended to the embedment of channel sections and establish the behaviour of the system under monotonic and cyclic loads.

[3] An additional testing program investigating ways to mitigate the out-of-plane buckling action and the beam uplift issues at the end of the cantilever section of the embedded wide flange I-sections should be carried out. Different combination of bolted connection, that is, bolt size, numbers, spacing, withdrawal capacity etc., should be tested in order to optimize the resistance of the beam against twisting and uplift from the embedded portion.

[4] A wall testing program that establishes the system level behaviour of the FFTT system; key design parameters such as shear capacity of the wall, ultimate drift, strength and stiffness degradation under cyclic loads, ductility factor, overstrength factor and the failure mode.
Bibliography


