BEHAVIOUR OF MASS-WOOD SHEAR-CORE STRUCTURES SUBJECTED TO STATIC WIND LOADS

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ABSTRACT

As the desire to construct tall wood buildings grows, so too does the need to understand the behaviour of these structures under all loading conditions. Work is progressing towards understanding their behaviour under extreme lateral loads, but relatively little focus is given in the literature to their behaviour under normal service-level loads. This study investigates the behaviour of 10-storey mass-wood shear-core structures subjected to static wind loads, and identifies major differences between the behaviour of the wood-core buildings and that of a traditional shear-core building. The design process is described, and both Ultimate and Serviceability Limit States results are presented, including base shear and overturning moments for lateral loads, shear lag in the core walls and lateral deformation behaviour. Special attention is paid to the impact that the low in-plane shear modulus of the mass-wood panels has on the behaviour of mass-wood structures. This investigation contributes to a fundamental understanding of the lateral response of mass-wood shear-core structures subjected to service-level wind loads.

Keywords: lateral deformation, limit states analysis, mass timber, static wind loads, shear lag, shear deformation

1. INTRODUCTION AND RESEARCH OUTLINE

Traditional tall buildings are constructed from concrete or steel, and often from a combination of the two. However, acquisition of their raw material is environmentally destructive, and the subsequent processing and disposal causes large volumes of Greenhouse Gases (GHGs) to be released into the atmosphere (Ferguson et al., 1996). As sustainable design is becoming increasingly important in today’s rapidly expanding world, wood is a natural choice for decreasing the emission of GHGs, as it sequesters carbon dioxide, and it is the only naturally renewable major construction material (Bowyer et al., 2008). For many years, wood could not meet the demands of high-rise construction, but recent advances in Engineered Wood technology have facilitated the design and construction of taller buildings. In 2009, the world’s first modern tall wood building – the Stadthaus – was constructed in England, reaching nine storeys and using Cross-Laminated Timber (CLT) for the entire structural system. Since then, many tall wood buildings have been constructed, and plans to build upwards of 20 and 30 storey wood buildings have been developed, including the recently announced 18 storey residence at the University of British Columbia (GHL Consultants, 2015).

If wood, concrete and steel buildings have similar lateral load resisting systems, it may be assumed that their responses to lateral loads will be similar. Wood is an orthotropic material, however, whereas concrete and steel are isotropic. Additionally, wood has a relatively low shear modulus, in the range of 5-8% of its elastic modulus, while concrete and steel typically have shear moduli in the range of 40-50% of their elastic moduli. These material property differences cause mass-wood structures to have quite different responses than similar concrete or steel structures. Research into the behaviour of mass-wood buildings under earthquake loads is rapidly progressing, including shake table tests of full-scale structures (Ceccotti et al., 2013), testing the performance of various panel/connection assemblies to determine optimal configurations for energy dissipation (Schneider et al., 2013), and numerically simulating building responses to earthquake loads (Liu and Lam, 2014). While the body of work is smaller, the behaviour of mass-wood buildings under dynamic wind loads has also been studied, focused particularly...
on their vibration response, using an equivalent static procedure to quantify peak accelerations and displacements (Reynolds et al., 2011).

While quantifying the behaviour and dynamic responses of mass-wood buildings under extreme lateral load conditions is necessary to develop design solutions that minimize damage and ensure public safety, it is more common for buildings to be subjected to service-level loads that can be analyzed statically. Knowledge of this behaviour is necessary to gain a better understanding of the performance of mass-wood buildings under lateral loads, identify key details for structural designers, and develop more rigorous design methods.

This study investigates the Ultimate and Serviceability Limit States behaviour of two 10 storey shear-core structures subjected to static wind loads, one with a CLT core and one with a Laminated Strand Lumber (LSL) core. Both buildings have the same floor plan, and are designed in accordance with Section 4 of the National Building Code of Canada (NBCC) (NRC Canada, 2010a). Section 2 will present the design data and design process for the buildings, first describing the Gravity Load Resisting System (GLRS), and then discussing the design of the Lateral Load Resisting Systems (LLRS). Section 3 will then discuss the numerical models, including the modelling assumptions, idealizations, and analysis process. Section 4 will present the results of the analysis, and discuss the unique behaviour of the buildings. Conclusions will be presented in Section 5.

2. BUILDING DESIGN

In shear-core structures, the core resists all applied lateral loads and a small portion of the gravity loads, while the perimeter framing resists the majority of the gravity loads. For design, preliminary member sizes were selected to satisfy Ultimate Limit States (ULS), using the load combinations presented in Table 4.1.3.2 of the NBCC (NRC Canada, 2010a). Serviceability Limit States (SLS) were then assessed using these member sizes to investigate if deflections were within code-specified limits.

Figure 1 shows the completed building model. The building location is assumed to be Toronto, where there are relatively high specified wind pressures and low seismic demands. Table 1 summarizes the magnitudes of the specified design loads and their sources. Wind loads were computed using the procedure specified in Cl. 4.1.7 of the NBCC, accounting for the relevant exposure, gust and pressure coefficients (NRC Canada, 2010a). Seismic loads were calculated using the Equivalent Static Procedure in Cl. 4.1.8 of the NBCC, using a conservative value of 1 for $R_o$ – seismic data can be found in Table C-2 of the NBCC. Live loads were reduced for tributary areas in accordance with NBCC criteria.

![Figure 1: Isometric view of completed building model](image)

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Magnitude</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superimposed</td>
<td>1.75 kPa</td>
<td>Table 11.3.1 – Wood Design Manual</td>
</tr>
<tr>
<td>Dead - Partitions</td>
<td>1.0 kPa</td>
<td>(CWC, 2010)</td>
</tr>
<tr>
<td>Dead - Mechanical + ceiling</td>
<td>0.45 kPa</td>
<td></td>
</tr>
<tr>
<td>Dead - Floor finish</td>
<td>0.1 kPa</td>
<td></td>
</tr>
<tr>
<td>Roof Mechanical</td>
<td>3.0 kPa</td>
<td>Assumed; Conservative (NRC Canada, 2010a)</td>
</tr>
<tr>
<td>Ground Snow</td>
<td>0.9 kPa</td>
<td>Table C-2 – NBCC (NRC Canada, 2010a)</td>
</tr>
<tr>
<td>Roof Snow</td>
<td>1.12 kPa</td>
<td>Cl. 4.1.6.2 – NBCC</td>
</tr>
<tr>
<td>Live</td>
<td>1.9 kPa</td>
<td>Table 4.1.5.3 – NBCC</td>
</tr>
<tr>
<td>Roof Live</td>
<td>2.0 kPa</td>
<td>Assumed; Conservative</td>
</tr>
<tr>
<td>Wind Velocity Pressure, $q$ (1:50)</td>
<td>0.52 kPa</td>
<td>Table C-2 – NBCC</td>
</tr>
<tr>
<td>$I_w$</td>
<td>0.75(SLS); 1.0(ULS)</td>
<td>Cl. 4.1.7.1. – NBCC</td>
</tr>
<tr>
<td>$C_v$</td>
<td>0.7(h/12)$^{0.3}$</td>
<td>Cl. 4.1.7.1.5 – NBCC</td>
</tr>
<tr>
<td>$C_d$</td>
<td>2.0</td>
<td>Cl. 4.1.7.6 – NBCC</td>
</tr>
<tr>
<td>$C_p$</td>
<td>0.8 – Windward; 0.5 – Leeward</td>
<td>Commentary I – NBCC (NRC Canada, 2010b)</td>
</tr>
</tbody>
</table>
In mass-wood structures, the connections are usually designed to dissipate inelastic energy during seismic events, while the panels respond as rigid bodies because their stiffness is much larger than that of the connections (Popovski et al., 2010). Since buildings are subjected to wind events much more frequently than seismic events, the connections must remain rigid to prevent structural damage when service-level wind loads are applied. Although the body of work is smaller, there is evidence that rigid connections are achievable in mass-wood buildings. For example, Vessby et al. (2009) indicate that appropriate inter-panel connection design can result in near-monolithic panel behaviour, while Sanders (2011) developed a rigid base connection design between a CLT panel and a concrete foundation. For this reason, it was assumed that the wood-core walls are fixed at the base and continuous over the full height of the building.

2.1 Design Procedure – Ultimate Limit States (ULS)

2.2.1 Gravity Load Resisting System (GLRS)

Figure 2 shows the floor plan of the structures. The layout and member dimensions of the GLRS are identical for each floor because the live loads due to occupancy remain consistent. For this study, all beams and girders were assumed to be simply-supported, and column loads were calculated using their respective tributary areas. A storey height of 3 m was chosen, allowing for a minimum clear storey height of 2.4 m. The beams and columns were dimensioned to resist factored loads and checked against SLS deflection requirements. Spruce-Pine Glulam with a stress grade of 20f-E was chosen for the beams, while the columns were selected to be 16c-E Douglas Fir-Larch Glulam because of its higher load-carrying capacity (CWC, 2010). Table 2 presents the member sizes and the sources used to determine each member size, annotated as shown in Figure 2.

![Figure 2: Building Floor Plan](image)

### Table 2: Floor Slab, Beam and Girder Dimensions

<table>
<thead>
<tr>
<th>Element</th>
<th>Size</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 (Glulam)</td>
<td>130 mm x 418 mm</td>
<td>Beam Selection Tables</td>
</tr>
<tr>
<td>B2 (Glulam)</td>
<td>175 mm x 418 mm</td>
<td>Manual</td>
</tr>
<tr>
<td>B3 (Glulam)</td>
<td>215 mm x 494 mm</td>
<td>– Wood Design</td>
</tr>
<tr>
<td>G1 (Glulam)</td>
<td>365 mm x 494 mm</td>
<td>(CWC, 2010)</td>
</tr>
<tr>
<td>G2 (Glulam)</td>
<td>315 mm x 494 mm</td>
<td></td>
</tr>
<tr>
<td>G3 (Glulam)</td>
<td>365 mm x 532 mm</td>
<td></td>
</tr>
<tr>
<td>C1 (Glulam)</td>
<td>265 mm x 266 mm</td>
<td>Column Selection</td>
</tr>
<tr>
<td>C2 (Glulam)</td>
<td>315 mm x 342 mm</td>
<td>Tables – Wood Design</td>
</tr>
<tr>
<td>C3 (Glulam)</td>
<td>365 mm x 418 mm</td>
<td>Manual</td>
</tr>
<tr>
<td>C4 (Glulam)</td>
<td>365 mm x 456 mm</td>
<td>(CWC, 2010)</td>
</tr>
<tr>
<td>Slab (CLT)</td>
<td>99 mm</td>
<td>CLT Design Guide (Structurlam, 2011)</td>
</tr>
</tbody>
</table>

2.2.2 Lateral Load Resisting System (LLRS)

Designing the LLRS of a shear-core building requires the selection of appropriate material strengths and wall thicknesses to resist the maximum factored load effects occurring at the base of the core walls. At SLS, inter-storey drifts must be less than 0.2% of the storey height (NRC Canada, 2010a). Designs of the buildings were based on ULS criteria, with SLS subsequently checked using Euler-Bernoulli beam theory, idealizing the building as a cantilever beam and the wind load as a Uniformly Distributed Load (UDL). A linear stress distribution in the core due to wind loads was assumed.

There is little guidance in the literature on accounting for slenderness effects when calculating the axial compressive capacity of mass-wood panels. The CLT Handbook (FPInnovations, 2011) suggests using the method presented in the Wood Design Manual (CWC, 2010) for compression members, assuming the resistance of the cross-layers of the panel to be negligible. This method was used in the development of the Cross-Laminated Timber Design Guide.
Wood is a natural material and its specified material strength depends on its type and grade. Many factors affect the capacity of a wood member, including its size, the load duration and service moisture conditions (CWC, 2010). For example, a wind event is a short-term load (resulting in an increase of the standard-term load-carrying capacity by 1.15) while dead and sustained live loads are long-term loads, and so cause the specified material strength to be reduced to 0.65 of the standard-term loading values. Since the material strength of raw wood cannot be altered, the only way to increase the load-carrying capacity of a solid or built-up wood member is to increase its cross-section, and for LSL and similar products, alter the epoxy design of the wood fibre-epoxy matrix (Stark et al., 2010). As the size of a solid wood member increases, the likelihood that a strength-reducing defect will occur in the member also increases, and to account for this, a size effect factor is included when calculating the capacity of a member. Since CLT is made up of smaller members glued together and LSL consists of small wood fibres and epoxy, it is likely that the size effect factor from the Wood Design Manual for solid members will result in conservative panel strength values. Nevertheless, until guidance on the size effect factor for mass-wood walls is published, the method in the Wood Design Manual for the size effect factor is adopted.

For the cores, the maximum compressive stress, \( \sigma_{C,max} \), in the leeward core wall can be calculated using:

\[
[1] \quad \sigma_{C,max} = \frac{P_{(D+L)}}{A_{core}} + \frac{M_{f,W}y_{max}}{I_{core}}
\]

where \( P_{(D+L)} \) is the total factored dead and live load supported by the core, \( M_{f,W} \) is the maximum moment due to factored wind loads, \( y_{max} \) is the distance from the centroid to the extreme fibre of the core, \( A_{core} \) is the area of the core and \( I_{core} \) is the moment of inertia of the core. An initial wall thickness was selected, the associated core area and moment of inertia were computed, and then the appropriate panel thickness was chosen to ensure the wall capacity was sufficient. For lightweight structures, the weight of the building may not prevent tensile stresses due to overturning caused by wind loads, so the maximum tensile stress, \( \sigma_{T,max} \), in the windward core wall can be calculated using:

\[
[2] \quad \sigma_{T,max} = \frac{P_{(D+L)}}{A_{core}} - \frac{M_{f,W}y_{max}}{I_{core}}
\]

It was determined that the governing ULS load condition for these buildings is a 1-in-50 year wind event, and the governing panel strengths were determined assuming short-term loading conditions. Using a dead load factor of 1.25 with Eq. [1] and a dead load factor of 0.9 with Eq. [2], the maximum computed compressive and tensile stresses for the CLT core are 3.79 MPa and 2.72 MPa, respectively, and for the LSL core, 4.53 MPa and 3.23 MPa, respectively. The CLT panels have a specified compressive strength of 11.5 MPa and a specified tensile strength of 5.5 MPa – the same as that of dimensional lumber – and only the layers oriented along the vertical axis are assumed to resist the applied lateral loads (Structurlam, 2011). A 5-ply panel with a thickness of 169 mm has a compressive resistance of 6.01 MPa and a tensile resistance of 5.69 MPa, and was selected for the CLT-core walls. The computed compressive demand is approximately 61% of the capacity, and the computed tensile demand is approximately 48% of the capacity. For the LSL panels, the entire cross-sectional area of the panel is effective. The specified compressive strength of a 1.3E LSL panel is 15.4 MPa and the specified tensile strength is 13.7 MPa (CCMC, 2015). The core wall thickness selected was 140 mm, with a compressive resistance of 8.64 MPa and a tensile resistance of 14.2 MPa. The calculated compressive demand is approximately 52% of the capacity, and the calculated tensile demand is approximately 23% of the capacity.

Traditional shear-core structures can be idealized as cantilever beams, and relatively accurate deflections can be calculated using Euler-Bernoulli beam theory. For example, the deflections of a cantilever beam subjected to a uniformly distributed load (UDL) can be calculated using (e.g., Beer et al., 2007):

\[
[3] \quad \Delta_y = \frac{w}{24E_I} \left( x^4 - 4hx^3 + 6h^2x^2 \right)
\]
where \( w \) is the distributed load, \( E_c \) is the elastic modulus of the material, \( I \) is the moment of inertia, \( h \) is the height of the cantilever, and \( x \) is the position along the beam at which the deformation is being calculated (\( x=0 \) at the fixed end of the cantilever). Using this equation and service-level wind loads, the maximum expected lateral deformation is 14.5 mm for the CLT-core building and 18.3 mm for the LSL-core building. The maximum expected inter-storey drift occurs in the top storey for both buildings, and is 2.0 mm for the CLT-core building, and 2.5 mm for the LSL-core building.

3. **FINITE ELEMENT ANALYSIS**

3.1 **Material Property Data**

Material properties of wood and Engineered Wood Products (EWPs) are orthotropic, and must be appropriately idealized for use in analytical models to obtain accurate results. Unlike for isotropic materials, constitutive relationships are not well established for mass-wood panels. For example, their in-plane shear modulus is estimated empirically to be 5-8% of the principal elastic modulus for softwood lumber (FPInnovations, 2011). Even though wood and EWPs are orthotropic, design guides for wood structures and manufacturer’s data typically provide only material strengths for members oriented parallel or perpendicular to applied loads, and the principal elastic and shear moduli for parallel applied loads only (CWC, 2010; CCMC, 2010; Structurlam, 2011). Therefore, designers typically must refer to published research for specified material properties.

Table 3 displays the orthotropic material properties assumed for the EWPs in the current study. Glulam properties were obtained from Zagari et al. (2009). Properties of CLT were obtained from Ashtari (2009), who adapted experimentally obtained properties from Gsell et al. (2007) for analytical modeling. Orthotropic material properties are not widely available for Laminated Strand Lumber. Since Parallel Strand Lumber (PSL) is similar to LSL – differing only in the length-to-thickness ratios of the individual member fibres – PSL material properties can be used to estimate the properties of LSL (APA Wood, 2015). Material properties of PSL were obtained from Winans (2014), the ratios between the principal modulus and those in orthogonal directions (e.g. – \( E_x/E_z \)) were calculated for the elastic and shear moduli, and the orthogonal material properties for LSL were estimated from the principal elastic and shear moduli values presented by CCMC (2015).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Glulam</th>
<th>CLT</th>
<th>PSL</th>
<th>LSL</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_x ) (MPa)</td>
<td>12000</td>
<td>4000</td>
<td>1400</td>
<td>965</td>
</tr>
<tr>
<td>( E_y ) (MPa)</td>
<td>900</td>
<td>8000</td>
<td>13000</td>
<td>8965</td>
</tr>
<tr>
<td>( E_z ) (MPa)</td>
<td>500</td>
<td>500</td>
<td>820</td>
<td>570</td>
</tr>
<tr>
<td>( G_{xy} ) (MPa)</td>
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<td>600</td>
<td>1060</td>
<td>560</td>
</tr>
<tr>
<td>( G_{xz} ) (MPa)</td>
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<td>95</td>
<td>50</td>
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<tr>
<td>( G_{yz} ) (MPa)</td>
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<td>500</td>
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<td>470</td>
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<td>( \nu_{xy} )</td>
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<td>0.07</td>
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<td>( \nu_{xz} )</td>
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<tr>
<td>( \nu_{yz} )</td>
<td>0.558</td>
<td>0.35</td>
<td>0.634</td>
<td>0.634</td>
</tr>
</tbody>
</table>

3.2 **Assumptions and Validations**

The connection assumptions made during the design phase were implemented in the FE model. The core walls for the buildings are assumed fixed at the base and continuous over the height of the building, while the column, beam and girder ends are assumed pinned. If the shear stiffness of the core exceeds five times that of the columns, then the core can be assumed to brace the frame and so the contribution of the perimeter framing system to the LFRS can be neglected (Johnson and Anderson, 1993). To investigate, an additional CLT-core model was developed with the columns fixed to the foundation, and the beams of the framing system fixed to the core to allow force and moment transfer. This analysis indicated that the columns contributed 0.5% of the overall lateral force resistance; thus, the assumption that the core fully braces the frame and resists the entire lateral load is valid.

The International Building Code (ICC, 2012) and ASCE 7-10 (ASCE, 2013) provide guidelines on classifying floor diaphragms as rigid or flexible. A diaphragm is rigid if its maximum in-plane displacement is less than twice the
lateral displacement of its vertical supporting elements when the structure is subjected to lateral load (Ghosh, 2010). For the structures in this study, both conditions were modeled, and it was found that the diaphragms behave rigidly.

Vessby et al., (2009) indicate that CLT panels exhibit linear elastic behaviour under bending loads – at 238 kN·m, the corresponding tensile and compressive stresses in the panel are 9.5 MPa. These stresses are larger than the computed maximum core stresses in Section 2.2.2; therefore, this study adopts a linear elastic material model for the mass-wood panels in the FE model. Experimental research also indicates that glulam behaviour is linear elastic (e.g., Kim and Harries, 2010), and this assumption has been adopted for the current analytical model.

4. RESULTS AND DISCUSSION

The following sections will discuss the results of both the ULS and SLS analyses performed on the structures investigated in this study. Section 4.1 will discuss ULS behaviour and the effects of shear lag on the core wall stress distributions. Section 4.2 will discuss the lateral deformation of the buildings under SLS-level loading. Section 4.3 will examine the effects of in-plane shear stiffness on this deformation.

4.1. ULS Analysis

4.1.1. Global Building Behaviour

Low-rise structures – considered to be those under 35 m in height (Emporis, 2016) – are usually governed by seismic loads, while taller buildings are more susceptible to wind loads. This is due to a combination of the material mass properties and the fundamental vibration periods of traditional low-rise structures made from either concrete or steel. Table 4 displays the base shears and overturning moments for the CLT- and LSL-core buildings for both earthquake and wind loads. As can be seen, wind loads govern the ULS design of the buildings, causing 57% larger base shears and 18% larger overturning moments in both cases. Wind base shears and overturning moments are based solely on wind load effects, causing the CLT- and LSL-core buildings to have the same values for these response parameters. Seismic base shears and overturning moments are in part dependent on the mass of the building, which accounts for the variability in the reported base shears and overturning moments for the two buildings since CLT and LSL do not have the same densities.

Figure 3 compares the expected base stress distribution of the CLT-core building to that obtained from FE Analysis – assuming a uniform stress distribution across the core flanges – as well as the tensile and compressive capacities for the core walls. Compressive stress in the core flange is the governing design parameter for ULS analysis. For both buildings, the maximum compressive stresses in the cores computed manually were lower than those predicted by FE Analysis; this is a result of load sharing between the GLRS and LLRS, where a portion of the gravity loads from the framing system are resisted by the core. Although these stresses were larger, the original designs remained sufficient for resisting the applied loads – the tensile demand in each building decreases, while for the CLT-core building the compressive demand increases to 71% of its capacity, and for the LSL-core, the demand is 61% of its capacity.

| Table 4: Wind vs. seismic base shears and overturning moments |
|---------------------------------|---------------------------------|---------------------------------|
|                                | Wind                            | Seismic                         | \( MW/ME \)                      |
| CLT Base Shear, \( V \) (kN)    | 1.50E+06                        | 9.55E+05                        | 1.57                             |
| Overturning Moment, \( M \) (kN·m) | 2.95E+08                        | 2.49E+08                        | 1.18                             |
| LSL Base Shear, \( V \) (kN)    | 1.50E+06                        | 9.57E+05                        | 1.57                             |
| Overturning Moment, \( M \) (kN·m) | 2.95E+08                        | 2.50E+08                        | 1.18                             |

Figure 3 – Expected vs. Actual core stress distribution for CLT-core building
4.1.2. Effects of shear lag

Shear lag is a well-known phenomenon in shear-core structures (e.g., Kwan, 1996) that causes higher stresses to occur at the flange-web interfaces of the core and lower stresses at the centres of the flanges. If shear lag is neglected during design, it is possible that the concentrated stresses may exceed the capacity of the core at the web-flange interfaces. Figures 4a and b display the stress distributions for the CLT-core and LSL-core buildings, respectively. For the CLT-core, the average stress of 4.25 MPa is 86% of the maximum stress, and the stress at the centre of the flange is approximately 70% of the maximum stress. Similarly, the average stress in the compression flange of the LSL-core building is 5.27 MPa, which is 84% of the maximum stress, and the stress at the centre of the flange is approximately 66% of the maximum stress. It is evident that assuming the compressive stresses to be uniform along the core wall length and using $\sigma_{avg}$ to size core walls may result in unsatisfactory building designs, with panel capacities exceeded at the web-flange interfaces. Recognizing different boundary conditions along the core walls, a more optimal design can be achieved. The edges of the core wall, where stresses are the largest, are fully braced by the perpendicular core walls and have effective lengths of zero, so the resistance is the capacity of the cross-section. The critical element in the compression flange therefore becomes the centre of the flange, where the panel can be assumed fixed at the base and pinned at the first story, reducing the effective length factor to 0.8, and the effective length to 2.4 m. Since $\sigma_{min}$ occurs at the centre of the flange, and is much lower than $\sigma_{avg}$, a thinner panel can be selected to resist the design loads.

Figure 4: Stress distribution along a) CLT-core compressive flange; and b) LSL-core compressive flange

4.2. Lateral Deformation Behaviour

Following ULS analysis, FE Analysis was used to quantify the total lateral deformation of the structure and check that SLS requirements were satisfied. At SLS, inter-storey drifts must be less than 0.2% of the storey height (NRC Canada, 2010a), which corresponds to 6 mm for the 3 m storey height investigated in this study. If the drifts exceed this limit, it may be necessary to increase the core wall thickness.

Figure 5a displays a comparison of the deformations computed using Eq. [3] and those calculated using FE Analysis. Both wood-core structures exhibit significantly different deformation profiles than a traditional shear-core structure, and the difference between the maximum deflections calculated from both methods is 50% for the CLT building and 56% for the LSL building. In both cases, the deformations calculated by hand are less than those obtained from the FE models. A small portion of these differences are due to the idealization of the wind load as a uniformly distributed load in the hand calculations, whereas it is idealized in the FE software using an exposure factor, $C_e$, that increases with the elevation and reaches a maximum value at the top of the building (NRC Canada, 2010b). Accounting for this variation increases the deformations computed using the FE model. The majority of the difference comes from neglecting shear deformations in the core. No term in Eq. [3] accounts for the shear modulus, $G_{sy}$, of the material, and since the in-plane shear moduli of the mass-wood panels are so low, the results using this equation are inaccurate and unconservative. For materials with low shear moduli, more accurate predictions of a structure’s deformations can be obtained using equations derived through Timoshenko beam theory. The lateral deformation of a cantilever, accounting for shear deformations, can be calculated using (e.g., Ochsner and Merkel, 2013):

$$[4] \Delta_y = \frac{1}{E_s I} \left( \frac{wx^4}{24} - \frac{wl^3}{6} + \frac{wI^2}{2k_s AG} + \frac{wE_s I}{k_s AG_{sy}} \right)$$
where $k_s$ is the shear correction factor (based on geometric properties) and $A$ is the cross-sectional area. Figure 5b compares the lateral deformations from FEA and those computed using Eq. [4]. For the CLT core, the difference decreases from 50% to 14%, and for the LSL core, the difference decreases from 56% to 16%.

**Figure 5**: Lateral deformations of CLT- and LSL-core buildings from Finite Element Analysis – a) comparison of FE Analysis and Eq. [3]; and b) comparison of FE Analysis and Eq. [4]

For any building with a shear core, the maximum inter-storey drift is expected to occur in the top storey. However, both the CLT and LSL buildings have the largest inter-storey drift in lower storeys – 3.16 mm occurring at the 5th storey of the CLT building, and 3.79 mm occurring at the 4th storey of the LSL building. The corresponding inter-storey drift plots can be found in Figures 7a and b in Section 4.3. The location shift of the maximum inter-storey drift can again be attributed to the low in-plane shear modulus of the wood panels, because shear forces – and by extension, shear deformations – are largest at the base of the structure. Since the in-plane shear modulus has such a large impact on the behaviour of mass-wood buildings, the remainder of this paper will investigate different behavioural aspects affected by the relatively low in-plane shear modulus of mass-wood panels.

**4.3 Sensitivity Analysis - In-Plane Shear Modulus Variation Effects on Building Response**

Since the in-plane shear modulus of the wood panels impacts the lateral deflections of the wood-core buildings, an accurate estimate of the in-plane shear modulus is needed. Since mass-wood panels are composite materials, manufacturing methods play a large role in defining their material properties. For CLT, the thickness of the laminate layers can vary, as well as the orientation of the layers (FPInnovations, 2011). For LSL, it is possible to change both the epoxy and the fibre material properties in the epoxy-fibre matrix to vary material properties (CCMC, 2015). Therefore, $G_{xy}$ typically ranges between $E_x/12$ to $E_x/20$ (FPInnovations, 2011). For the CLT-core, this equates to a range of 0.40-0.67 GPa, and for the LSL-core, a range of 0.45-0.75 GPa. Additional experimental results from a static in-plane bend test of a CLT panel (Vessby et al., 2007) indicate an in-plane shear modulus as high as 1.5 GPa.

Figures 6a and b display the lateral deformations of the the CLT- and LSL-core buildings, respectively, computed using different shear moduli. As $G_{xy}$ becomes larger, increasingly discernible bending deformations occur at the base of the structure. Although these effects are more noticeable in the CLT-core building, both buildings show the same trend. By eliminating shear deformations from the buildings, the response echoes that of a traditional shear-core structure. Through this analysis, it was determined that shear deformations in the CLT-core building contribute between 30% and 58% of the total lateral deformation, and for the LSL-core building, between 43% and 55%.
Varying the in-plane shear modulus also affects the inter-storey drifts of the CLT- and LSL-core buildings, as shown in Figures 7a and b. As the in-plane shear modulus increases, the maximum inter-storey drifts in both buildings decrease. For the CLT-building, the increased in-plane shear modulus also shifts the maximum inter-storey drift shifts from the 4th storey to the 7th storey. Because the range of the in-plane shear modulus of the LSL-core building is much smaller, the position of the maximum inter-storey drift shifts only from the 4th to the 5th storey.

Figure 7: Inter-storey drift profiles – varying in-plane shear modulus of a) CLT panels; and b) LSL Panels

5. CONCLUSIONS

This paper has discussed the design and analytical modelling of 10 storey mass-wood shear-core structures, and highlights the unique behavioural characteristics of mass-wood shear-core structures subjected to static wind loads. Section 2 demonstrated that the many factors affecting the strength of wood and EWP s, such as load duration and member sizes, must be considered when determining the governing load conditions and corresponding panel capacities. The challenges of obtaining accurate material property data for mass-wood panels were also presented.

For the wood-core buildings examined in this study, ULS wind loads govern the design, with the compressive core wall being the critical element. Due to the low in-plane shear moduli of the mass-wood panels, it was seen that the effects of shear lag are quite large in the wood-core buildings. For both buildings, the stress at the centre of the compression flange was approximately two-thirds that of the maximum occurring stress at the web-flange interface. Accounting for these effects and designing with two boundary conditions – one at the web-flange interface and one at the centre of the flange – instead of using the average wall stress results in a more economical building solution.

The low in-plane shear modulus of the mass-wood panels also affects the lateral deformation behaviour of the structure. Instead of the classic bending deformation profile exhibited by a traditional shear-core building, the wood-core buildings exhibit a more linear deformation profile, and maximum inter-storey drifts occur in the middle of the building rather than at the top. It was shown that shear deformations contribute a significant portion of the total lateral deformation of each building, and must be considered during design. Deformation equations derived from Timoshenko beam theory provide much more accurate estimations of the deformation profiles of both buildings than those derived from Euler-Bernoulli beam theory, decreasing the difference between the estimated deformations and those obtained from FE modelling from approximately 50% to approximately 15%.

The sensitivity analysis indicated clearly that small changes in the in-plane shear modulus of a mass-wood panel can significantly affect the shape of the deformation profile and the location of the maximum inter-storey drift. For the range of in-plane shear moduli investigated, the maximum inter-storey drift shifts from the 4th to the 7th storey in the CLT-core building, and from the 4th to the 5th in the LSL-core building. Since there are many different manufacturers of mass-wood panels, and varying manufacturing processes, it is imperative to obtain accurate manufacturer’s data for the specific panels being used in a project to accurately predict the building’s behaviour. If this is not possible, then a conservative value for the in-plane shear modulus should be used.

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