Proof of Concept: A Relocatable Panelized Mass Timber Modular Building System Designed for Disassembly and Reassembly

by

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final reviews, as accepted by my examiners.

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Abstract

Canada has been experiencing a housing crisis for several years now with specifically Ontario having the lowest housing supply in the country. To combat this crisis, several rapid housing initiatives and projects have been created; as a result, bringing modular construction back into the mainstream. Modular construction has been the method of choice for a significant portion of rapid housing developments due to its shortened on-site construction timeline and potential for repeatability. Developments in several locations are all designed and prefabricated off-site by a manufacturer and distributed to these locations streamlining the construction process. These projects thus far have focused on the need for rapid affordable and supportive housing in city centres; however it should be noted that with the progression of climate change and its inevitable effects (e.g., fires, flooding, etc.), particularly in more remote regions with limited access to resources, the focus on providing rapid housing should be expanded to include remote regions in addition to meeting the needs of those in cities. Further, the existing rapid housing and modular construction market has gravitated towards the use of traditional light-frame construction methods, steel frame, and in some cases (mostly in literature thus far) cross-laminated timber (CLT) as well as other mass timber products (e.g., glulam). Thus, a relocatable modular building, made using sustainable methods and materials, and designed to meet the conditions in urban, rural, and remote regions in Ontario could provide a solution for rapid housing that can be assembled, disassembled, and relocated to meet the varying housing demands across the province. To focus on the environmental sustainability aspect of the concept, a prototype was designed by implementing CLT wall and floor assemblies and compared to an equivalent light-frame wood solution to assess the feasibility of using mass timber of in this relatively small-scale application.

The aim of this study is to design a panelized modular building prototype that can be disassembled, relocated, and reassembled to meet the housing demand (or demand for any other small-scale buildings) all over Ontario. A complete prototype design is conceptualized including a full panel set with associated assembly information to create three different configurations of the building. The structure consists of CLT panels and structural insulated panels (SIPs) and is designed to withstand the worst-case structural loading conditions in Ontario. A preliminary prefabricated building enclosure that would be pre-installed onto the structural panels is designed. Finally, novel connections that ensure the prototype can be disassembled and reassembled with ease are conceptually designed.

An experimental testing program was developed to evaluate the durability of the CLT assembly and compare it to a light-frame equivalent system by loading wall-to-floor assemblies using the designed connection for the CLT system and a typical hold-down for the light-frame system. The testing included two phases, the first phase consisted of a monotonic test to failure to establish the actual capacity of the system, and the second phase consisting of a round of cyclic testing to the design load to simulate a service life, a series of drop tests to induce any damage that might occur during disassembly or transportation, and finally a monotonic test to establish the

new capacity of the panel when compared to the capacity found in the phase 1 testing. Ultimately, the light-frame panel lived up to its reputation as the residential structural material of choice in Ontario and was able to be reassembled for the final monotonic test through which a reduction in lateral strength of approximately 12% was observed. The CLT system, when initially monotonically tested achieved a higher maximum load than the light-frame despite being designed for the same design load. Upon conducting phase 2 testing on the CLT system it was observed that no visible damage was caused by the cyclic testing and despite incurring some damage during the drop testing, the system was still easily reassembled for the final monotonic test. Overall, the CLT system saw a reduction in lateral strength of about 20% with a different mode of failure observed between phases 1 and 2.

Finally, a preliminary life cycle assessment (LCA) was conducted on the CLT and equivalent light-frame building systems to investigate specifically the embodied carbon impacts of both systems. The LCA took into account the floor and wall panels of the structure itself, the fasteners between these components, and the insulation required for each system type (as this varies notably between a light-frame and CLT system). The roof panels were omitted from the investigation as the CLT prototype considered a SIP roof which is primarily made up of the same assembly as a light-frame roof and would yield similar LCA results. Further, the foundation system is omitted but would be consistent between the two systems thus also yielding similar LCA results. The initial LCA of the building considering each structure type indicated that the light-frame equivalent had less than half the associated embodied carbon emissions as the CLT prototype. Upon applying reuse parameters in a model to assess if the CLT becomes feasible in a scenario in which it is more durable (can withstand more reuses than the light-frame), it was concluded that the embodied carbon emissions of each system do not significant vary and are therefore comparable.

It is ultimately concluded that the CLT system is a reasonable solution for a durable relocatable small building. The durability of the CLT; however, is only moderately improved over the light-frame equivalent based on the experimental testing conducted. The LCA has shown that while the system is comparable, several factors are to be considered and the outcome would ultimately depend on the duration of each use, the number of uses expected, and the carefulness with which the building is disassembled, relocated, and reassembled. Thus, while CLT provides a feasible solution to small scale relocatable buildings in certain conditions, it does not necessarily provide a clearcut improvement upon traditional light-frame construction.

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Chapter 1 – Introduction

1.1 Background

Canada has been experiencing an on-going housing crisis with the nation having the lowest housing stock per capita among the G7 countries while also having the highest rate of population growth in the G7 and seventh highest among the G20 between 2016 and 2021 (Bharti, 2022; Statistics Canada, 2022). Within Canada, Ontario fell amongst the lowest in housing supply per capita along with Alberta at 398 units per 1,000 people compared to Canada's 424-unit average (Bharti, 2022). In response to the housing crisis, the Canada Mortgage and Housing Corporation (CMHC) announced a $1 billion Rapid Housing Initiative (RHI) in 2020 (Canada Mortgage and Housing Corporation, 2020). The goal for the first round of the initiative was to create 3,000 new housing units per year; this included funding for the construction of new builds, the acquisition of land, and the conversion of existing buildings into affordable housing units provided the projects are completed within a 12-month period of the funds being committed. The first round of the initiative was quickly followed up with a second round of funding announced in 2021 due to the number of applications for funding far exceeding the $1 billion; this brought the RHI to a $2.5 billion initiative (Community Housing Transformation Centre, 2021). The RHI's 12-month timeline called for construction methods that could meet the short timeline for new builds, therefore, the initiative contributed to significant advances in the modular construction industry and research community in Canada based on the range of successful projects funded by the RHI.

The growth of the modular industry within Ontario has primarily seen traditional light-frame wood construction methods applied to small-scale residential projects (i.e., supportive and transitional housing) (Roscetti et al., 2022). This is primarily due to the construction industry's familiarity with this long-standing building method along with the structural capabilities of light-frame that make it suitable for small-scale residential construction. However, in recent years, the growth of the mass timber industry has begun to encroach on the modular construction industry with new studies and projects merging the two industries and construction methods. The cross-laminated timber (CLT) industry alone, a $1.4 billion USD market in 2023, is projected to reach $2.6 billion USD in market size by 2028; this presents a compound annual market growth of nearly 14% during the forecasted period (MarketsandMarkets, 2023). Increasingly, engineered wood and mass timber products have surfaced in research studies including temporary CLT panelized modular residential units as well as in construction projects globally including a comprehensive school in Germany using CLT and laminated veneer lumber (LVL) in its modules (Bhandari et al., 2020; Transsolar Energietechnik GmbH, 2023). The increased presence of mass timber products in the modular construction industry appears to overlap with design concepts that are considered "temporary", "reusable", or "movable". This is likely due to the nature of modular construction and its ability to be broken down into components along with mass timber's inherent modularity, as it is a prefabricated panel-like product that is manufactured off-site.

The industry of temporary or "relocatable" modular structures is not a new concept, as portable buildings such as classrooms, office trailers, and mobile homes have been in existence for decades. Historically, these buildings have acquired a negative perception from the public for being built to low-quality standards while also being architecturally repetitive and restrictive. However, this industry is now gaining renewed interest as it moves away from its reputation of producing low-quality inexpensive structures with issues such as leakage and poor insulation. Instead, new, and better-quality materials and construction methods are being employed, leading to increased flexibility and adaptability of these buildings (M. J. Ribeirinho et al., 2020). Durability is a term commonly associated with portable structures, as they must withstand relocation while still maintaining a similar lifespan to traditional permanent structures BOXX Modular, 2022). In the past, durability has been achieved through the manufacturing of volumetric modules, which are essentially portable boxes that offer limited options for use or programming. However, the industry is now moving towards a more adaptable approach, which requires a shift away from simply creating prefabricated, indestructible box-shaped portables. This new approach involves using combinations of volumetric components, panelized structures, and smaller individual components to achieve a more flexible and circular "temporary" or "relocatable" modular industry. Rather than solely focusing on meeting the strength and durability criteria, the emphasis is moving towards optimal material usage, flexible and reversible connections, and overall structural systems that can provide the necessary flexibility. Furthermore, there is currently little to no design consideration for what happens at the end-of-life of such structures, which now plays an increasingly larger role in terms of sustainability and life cycle assessment (LCA).

Sustainability in general can be defined as meeting our own needs as a society without sacrificing the ability of future generations to meet their own needs (University of Alberta, 2023). The concept can be divided into 3 main pillars of sustainability which include environment, economy, and society. Holistic sustainability can only be met when sustainability of all 3 pillars is achieve. Throughout this study, however, sustainability primarily refers to the environmental sustainability pillar. More specifically, sustainability within this study is defined as low life cycle carbon emissions; thus, the most sustainable solution is the least embodied carbon intensive solution.

LCA is a procedure in which both the embodied and operational carbon emissions as well as environmental impact of a product (e.g., a building) can be accounted for over its entire lifespan (London Energy Transformation Initiative, 2017). The LCA procedure ties into the overarching push for a circular economy in the building and construction industries. The circular economy is a concept developed to combat the traditional "take-make-waste" (i.e., linear) economy; an economy in which products are produced by taking resources, making the product, and disposing of the product once it is no longer useful (ARUP, 2023b). The existing linear economy is prominent in the construction industry by the sourcing significant amounts of materials, processing and manufacturing them to be used in construction, and then disposing of them once the building or development is demolished. As of 2021, the building industry accounted for 37% of global carbon emissions and while operational carbon emissions have been reduced in recent years by way of more efficient building systems, embodied carbon emissions must be addressed (Global Alliance for Buildings and Construction, 2022). By implementing the use and reuse of renewable materials and products and extending the service life, thus delaying the end-of-life disposal, the construction industry could see significant reduction in the embodied carbon emissions it produces.

CLT and other mass timber products are marketed as sustainable alternatives to traditional concrete and steel construction as they manufactured using wood which is a renewable resource. However, when implemented in smaller-scale construction projects that could also be achieved using traditional light-frame methods, it brings into question whether mass timber products are the more sustainable choice. While there seems to be an increase in mass timber modular projects being deemed “relocatable” or “portable”, it raises the question as to if the material is being used for its durability, particularly in the disassembly, transportation, and reassembly of each building as these processes have proven to be a significant factor in defects of modular buildings (Johnsson & Meiling, 2009). Similarly, a number of light-frame modular construction projects, specifically in Canada, have been deemed temporary and relocatable, these include the City of Vancouver’s Temporary Modular Housing Project along with the Durham Region’s Oshawa Micro-Housing Pilot Project (Durham Region, 2021; NRB Modular Solutions, 2020). An additional “rapidly deployable” panelized modular house design came from a prototype developed by Canmet Energy Natural Resources Canada (NRCan) (Haslip & Sinha, 2016). The prototype consisted of primarily structural insulated panels (SIPs) and a lightweight, adjustable above ground steel foundation system; however, the building itself required a significant number of parts to be assembled on-site and further, was only assembled on a site one time and thus not specifically tested for durability. Although the definitions of temporary and relocatable structures are generally agreed upon, the construction type might be different from project to project. For example, a tiny house on a trailer bed or portable office trailer can be relocated much more easily than a building on a temporary foundation. The type of modularity chosen (e.g., volumetric, panelized), material (e.g., light-frame, CLT, steel, etc.), and connection system will also affect the logistics surrounding the transportation and, if applicable, disassembly and reassembly.

1.2 Research Needs

The need to meet the increasing demand of rapid housing solutions combined with a desire for a more environmentally sustainable and circular construction industry has led to the development and implementation of novel modular concepts for affordable housing. In modular construction, factors such as the location, available transportation means, intended service life, and project scale will play a key role on the chosen solution. Due to the different levels of prefabrication and modularity, a detailed investigation of appropriate materials and structural systems must be conducted in order to select the most appropriate and efficient solution that satisfies the imposed constraints. For example, remote regions may be limited in terms of access to equipment for on-site assembly or shorter-term housing supply would benefit from an easily transported modular building system to be reused on another site. An investigation of recent modular projects in Ontario has revealed that most are geared towards being permanent with no possibility for relocation (Roscetti et al., 2022). Increasing occurrences of severe weather due to climate change (e.g., floods, forest fire, tornados) raises the need for the development of non-volumetric relocatable buildings for both permanent housing and temporary (e.g., emergency) housing scenarios.

1.3 Research Objectives

The overarching aim of this research is to develop a prototype design for a relocatable panelized modular building using sustainable products that can be disassembled and reassembled for any site in Ontario and to investigate the durability and feasibility of such a design through experimental testing and life cycle assessment (LCA).

Whereas the entirety of the prototype will be designed conceptually, in order to assess the durability of the prototype, a portion of the CLT panelized assembly was fabricated for experimental testing. The wall-to-floor connection was selected to evaluate its feasibility and durability when subjected to loads representative of in-service conditions. The test set up required the development and fabrication of a novel wall-to-floor connection intended for easy disassembly and reassembly. The structural behaviour and durability performance of the CLT wall-to-floor system was compared to that of a light-frame wall which was designed for disassembly and reassembly using conventional construction methods.

The observations and results obtained from the experimental testing program are then used to inform the parameters used in the life cycle assessment (LCA). The experimental investigation of the novel CLT wall-to-floor connection as well as LCA will provide valuable information into the ability to deploy and implement a panelized modular building system that is specifically aimed for disassembly and reassembly in the industry.

1.4 Scope

The overall objective will be met by the completion of the following:

1. Detailed literature review on modular construction, wood as a material for modular construction, circular economy, and life cycle assessment.
2. Development and conceptual design of a scalable one-storey panelized modular building system for disassembly and reassembly using sustainable materials.
3. Preliminary detailing of a prefabricated enclosure system that can be pre-installed onto the panels off-site.
4. Development and design of a novel CLT wall-to-floor panelized modular structural system capable of withstanding the worst-case loading conditions in Ontario.

5. Design of a LF wall-to-floor panelized modular structural system capable of withstanding the worst-case loading conditions in Ontario.
6. Experimental investigation of the reassembly durability of the CLT structural system by conducting monotonic and cyclic loading tests on a portion of the CLT system and an equivalent light-frame system to compare and analyze the feasibility of the developed systems.
7. Discussion of the experimental results by comparing the behaviour of the two assemblies.
8. Completion of a preliminary life cycle assessment (LCA) on the CLT prototype design and equivalent light-frame design. Use LCA results and reuse parameters guided by experimental results and observations to assess the feasibility of each system after varying reuse cycles.

The scope of this project is limited to the structural design of the building system prototype which includes a full set of roof, wall, and floor panels that can be used in different combinations to achieve the desired size (e.g., studio, one bedroom, two bedrooms, etc.), within the limitations of the structural design of the system. The experimental component is limited to testing representative CLT and LF wall-to-floor assemblies. While outside the scope of the thesis, preliminary detailing of a prefabricated enclosure system that can be attached to the panels off-site is also provided. Specific limitations of the research are further discussed throughout the thesis as well as in the future recommendation section of Chapter 7.

1.5 Thesis Organization

The thesis focuses on the development of a novel CLT wall-to-floor connection designed for disassembly and reassembly in a scalable one-storey panelized modular building system that is compared to the performance of a light-frame wall assembly representative of conventional materials used in residential low-rise buildings. The thesis can be divided into three distinct phases: conceptual development of the building system for disassembly and reassembly with a novel CLT wall-to-floor connection; experimental testing of the CLT and LF wall-to-floor systems; and life cycle assessment of the CLT and LF systems. The following provides a brief description of each chapter in the thesis:

Chapter 1 introduces the research topic and provides background information, research needs, objectives, and scope.

Chapter 2 presents a detailed literature review on modular construction, the state of the modular construction industry in Ontario, the circular economy, and its overlap with the wood industry, and finally wood as a building material and its implementation in modular construction.

Chapter 3 presents the design of the relocatable panelized modular prototype. This includes an overview on the architectural design and configuration of modular components, the structural design of the roof, wall, and floor components, the enclosure design for each structural components, connection design, and finally the complete prototype assembly.

Chapter 4 presents the experimental testing program. This section includes the material preparation and laboratory test setup, and a detailed overview on the testing completed.

Chapter 5 presents and discusses the results obtained from the experimental tests and their implications on the feasibility of the systems.

Chapter 6 provides a preliminary life cycle assessment of the proposed CLT prototype in comparison to a light-frame equivalent. The LCA results are then used to project how the compared numbers vary in the case of reuse cycles with imposed parameters on the two structural systems.

Chapter 7 provides conclusions and recommendations on future work.

Chapter 2 – Literature Review

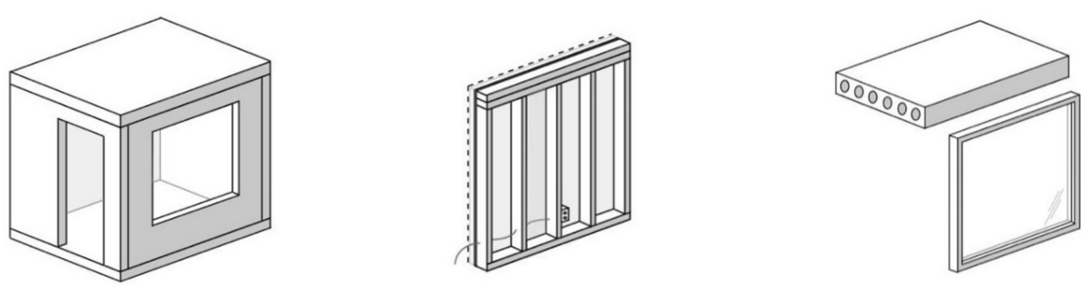
2.1 Overview of Literature Review

This section provides a literature review on modular construction, the state of the modular construction industry in Ontario, the circular economy, and its overlap with the wood industry, and finally wood as a building material and its implementation in modular construction. The literature review serves as both a foundation and precedent for the development of a novel modular panelized structural system that can be located on any site in Ontario including in city centres (e.g., for temporary housing on vacant land) and more rural and remote regions (e.g., emergency housing for areas experiencing flooding, fires, or any effects of climate change.

2.2 Overview on Modular Construction

2.2.1 Modular Systems and Materials

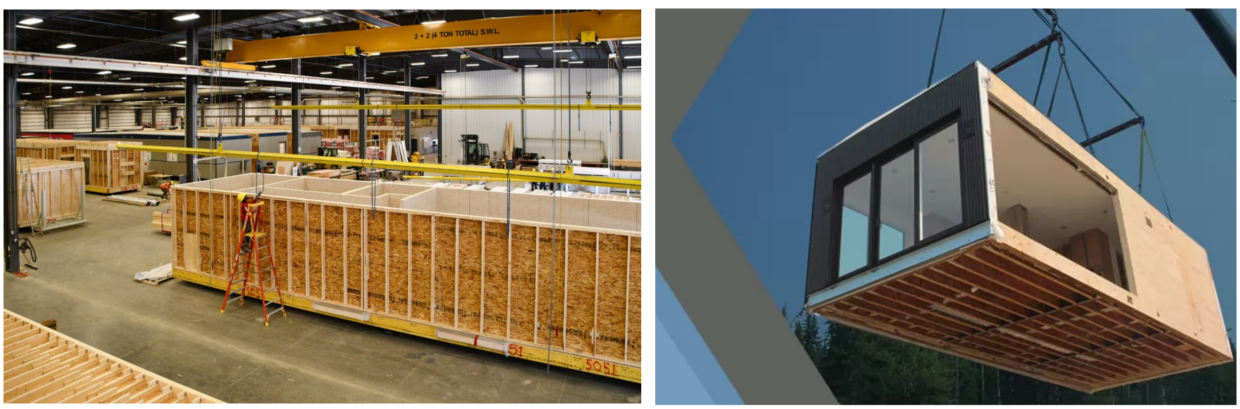
Modular construction is a building method in which the components, or "modules", of a building are manufactured off-site in a controlled environment (e.g., facility, manufacturing yard) that are then transported and assembled on-site (Modular Building Institute, 2022). Modularity can be achieved through different levels of prefabrication as illustrated in Figure 2.1; including volumetric modules (Figure 2.1a), panelized modules (Figure 2.1b), and smaller, individually prefabricated building elements (Figure 2.1c). Common structural materials used in modular construction include, but are not limited to, concrete, steel, and wood; the use of these materials will depend on the application and the desired type of modularity. Given the nature of modularity and the associated requirements for transportation and maneuvering for off-site construction and on-site assembly, structures with a higher level of modularity and prefabrication are more commonly built with lighter material systems such as timber and steel-framing, rather than heavier alternatives, such as large concrete components.



(a) Volumetric (b) Panelized (c) Prefabricated

Figure 2.1 – Levels of Modularity

The volumetric modular building approach (Figure 2.1a) consists of the manufacturing of 3-dimensional modules that can be assembled horizontally side-by-side or vertically stacked on-site. This approach results in the least amount of on-site work as the modules are only required to be lifted into place and attached (e.g., bolted) to the surrounding structure, before the installation of building service connections on-site (Henderson, 2018). However, it often results in a less economical use of materials with walls, floors, and ceilings being doubled-up when modules are placed adjacent to each other. While a volumetric module can be built using wood, steel, or concrete; concrete is not commonly used in these types of projects due to its significant self-weight. Light-frame wood and steel framed structures are therefore commonly used for volumetric modular construction as they provide adequate structural stability while keeping the total weight of the structure relatively low thereby optimizing the material use. For example, NRB Modular Solutions is a modular building manufacturer owned by Dexterra Group and has provided light-frame wood modules for affordable housing projects across Canada (Modular Building Institute, 2023). These projects largely consist of housing funded by Canada's Rapid Housing Initiative (RHI) such as the City of Toronto's Modular Housing Initiative (100+ units) and the City of Vancouver's Temporary Modular Housing (600+ units) (City of Toronto, 2023; City of Vancouver, 2023). The prefabricated light-frame wood modules are manufactured in NRB's Grimsby and Cambridge facilities, which include the structural framing (Figure 2.2a), as well as insulation and finishes (Figure 2.2b) prior to being transported and assembled on various sites.



(a) NRB Modular Solutions Off-Site Construction (b) NRB Modular Solutions Completed Module

Figure 2.2 – Light-Frame Volumetric Modules

*Reproduced from (a) NRB Modular Solutions (2023) and (b) Dexterra Group (2023)

As previously mentioned, steel framing can also be used for a volumetric modular structure. Steel framed modules can be manufactured and transported to site with the primary structural framed system only (Figure 2.3a), or can be built up with finishes, similarly to the wood based NRB housing modules, to reduce on-site work. Retired shipping containers built for transportation purposes were introduced into the modular construction sector as part of the building sector's shift toward more sustainable practices. Figure 2.3b shows an example where the containers were converted to residential units for an alternative housing project in the Waterloo Region (EngageWR, 2021). Although wood is a less carbon intensive material alternative when sourcing and manufacturing new elements from raw materials to be used in a new build, retired shipping containers present a solution that involves the reuse of steel.

Like the framed systems previously discussed, the shipping container structure can be built up with a complete enclosure and delivered to site.

(a) Steel Framed Volumetric Module

(b) Shipping Container Modules

Figure 2.3 – Steel-based Volumetric Modules

*Reproduced from (a) NSC (2005) and (b) EngageWR (2021)

The panelized modular building approach (Figure 2.1b) is the manufacturing of flat, or 2-dimensional, building components off-site. The panelized approach might consist of something as simple as a structural panel (e.g., a prefabricated stud wall) or as complex as a complete wall assembly including the structural elements, insulation, cladding, etc. Although this method requires more on-site assembly than the volumetric modular approach, it eliminates the need for doubled-up components (e.g., walls, floors, ceilings), reduces shipping volume, and ultimately may allow for more flexibility in the design of the space as it is not limited to the physical dimensions of a volumetric module. Panelized modules can be built using steel or wood-based structural panels; light-frame wood, CLT, or steel frame systems can be prefabricated in open or closed panel assemblies. Figure 2.4a shows a closed wall panel including sheathing, insulation, studs, and pre-installed windows. CLT panels can also be built up into closed wall assemblies like the Cross Laminated Insulated Panel (CLIP) manufactured by Element5, as shown in Figure 2.4b.

(a) Prefabricated Closed Wall Panel (b) Cross Laminated Insulated Panels (CLIPs)

Figure 2.4 – Panelized Modules

*Reproduced from (a) Paradigm Panels (2022) and (b) Element5 (2022)

The prefabrication of individual building elements offsite (Figure 2.1c) is the simplest form of modular construction as it includes any other building components that are manufactured off-site but do not meet the volumetric or panelized modular descriptions. This may include structural elements such as prefabricated mass timber or precast concrete columns, beams or slabs that are transported and connected in place on-site. This approach requires the most amount of on-site work but also allows for the most flexibility in the resulting structure as it does not need to adhere to standardized volumetric or panelized module sizing.

Prefabrication is commonly employed in concrete, steel, and wood structural systems. Precast concrete is a prefabricated modular construction method used for members like columns, walls, beams, or slabs (Figure 2.5). The use of precast concrete in large scale construction projects is often employed since the concrete can be poured and cured in reusable molds in a controlled environment off-site unlike cast-in-place concrete (Wong & Loo, 2022).

Figure 2.5 – Hollow-core Precast Concrete Slabs

*Reproduced from Fritz-Alder Precast (2022)

Smaller individual structural building components can also be prefabricated using steel (i.e., hot rolled H beams) and mass timber (i.e., glulam columns, CLT panels). In essence, any building project with a structure composed of mass timber or steel elements inherently uses a form of modular construction. Whereas mass timber products are more commonly found in larger scale construction projects, they are also finding their way into low- and mid-rise construction projects as well. For example, R-Hauz is a company focused on providing more affordable housing in Toronto by using wood construction (i.e., light-frame, mass timber) and also built Ontario's first all wood six-storey residential building in 2021 (R-Hauz, 2022). Similarly, Element5 and Edge Architects collaborated on the YWCA Supportive Housing development in Kitchener, Waterloo, a four-storey mass timber residential building completed in 2022, that was fully assembled within 20 days of on-site construction (Hicks, 2021).

2.2.2 Advantages of Modular Construction

Adopting a modular approach to a construction project can come with several advantages over a standard and traditional construction approach such as: 1. Reduced project timelines, 2. Reduction in material waste, 3. Standardized manufacturing process, 4. Quality control, and 5. Optimized working conditions. These five primary advantages will vary alongside the level of modularity implemented.

Modular construction methods can reduce overall project timelines due to the off-site manufacturing of modules or building components. In a standard building project, the construction of the superstructure of a building cannot be started until the site preparation and foundation portions are complete. Off-site fabrication allows for the components of the superstructure to be built while site preparation and building foundations are in progress. Further, the superstructure of the building can be assembled significantly faster on-site due to the level of off-site pre-fabrication; in an ideal modular project, building modules are simply craned into place and connected to adjacent modules. Modular methods can reduce the on-site construction timeline of a project by over 50%; however, this number ranges depending on the level of modularity and amount of pre-fabrication implemented into a project (National Audit Office, 2005).

In a typical construction project, nearly 30% of all materials delivered to site become waste by the end of construction (Osmani, 2011). This can be attributed to a number of factors including materials sitting out on site in ranging weather conditions, inefficient uses of materials, and changes to the structure and/or materials due to any unforeseen circumstances. However, the material waste associated with a construction project can be drastically reduced using modular construction methods (Osmani, 2011) As the majority of the structure is prefabricated in a manufacturing facility, materials are not exposed to any weather conditions that might compromise them. Further, modular projects typically require a more extensive and thorough planning and design phase to ensure the building components are manufactured to the required tolerances for on-site assembly leaving less need for unnecessary changes or additional materials. In a case study completed by Loizou et. al. (2021), the weight of material waste from a conventional construction project and modular project were compared and it was concluded that using modular construction methods can reduce the weight of material waste by up to 83% and reduce the overall project waste by 20-65%, depending on the level of modularity.

When modular construction methods are used in structures with repetitive spaces (e.g., student housing, hotels, affordable housing), modules can be standardized meaning that the component is simply designed once and then manufactured repeatedly as required. This can ultimately result in time and cost savings for these structures with highly repeatable spaces as the design for only one module is required and can then relatively easily be mass produced. The City of Vancouver's Temporary Modular Housing project presents an example of the benefit of the standardization of manufacturing processes; a modular unit design was repeatedly manufactured to provide over 600 units between 2019 and 2020 and distributed to more than 10 different development sites (City of Vancouver, 2023).

The off-site manufacturing that comes with modular construction ensures that modular components can be built in a facility under strict quality control measures (Bertram et al., 2019). Quality control can be optimized in a plant, which reduces the need for any rework or repairs for defective components. The off-site construction also protects the modules from outdoor conditions that might compromise the materials or structures before they've been weatherproofed which would otherwise pose more of a challenge on a traditional construction site.

With a large portion of component construction taking place off-site in a modular project, there is less foot traffic on the construction site which frees up space other project processes to take place (e.g., foundation work). All fabrication of modular components takes place in a controlled plant environment which not only protects workers from working long hours in outdoor weather conditions but reduces the risk of injuries and fatalities that are commonly associated with on-site construction. This may include risks associated with working at heights and near constantly moving heavy machinery (e.g., cranes, excavators, etc.). A manufacturing facility can more easily implement safety measures with designated production lines and spaces.

2.2.3 Disadvantages of Modular Construction

While the modular building approach can provide major benefits to project timelines and overall quality, there are also a number of disadvantages to the construction method that should be considered when investigating if the modular approach is appropriate for the given project. These include 1. Size restrictions and limitations, 2. Requirement for trained personnel, 3. Capital construction costs, 4. Upfront fabricator costs, 5. Potentially un-economical design, and 6. The requirement of a crane.

Due to the prefabrication of modular construction, modules must be transported to site after they are manufactured off-site. Modules must therefore be limited in size based on transport capabilities (i.e., a module being transported by flatbed truck but meet the size and weight requirements to fit on the truck bed) (Lacey et al., 2018). Further, internal layouts of modular buildings are limited by based on structural components and their size requirement.

Contractors, architects, and engineers are all traditionally trained and experienced in standard construction methods; however, they often lack training in the construction and management of structures employing modular methods. Contractors may not be trained in the field to handle modular components and connections while architects and engineers also lack education in this field.

The ability to pre-fabricate modular components of a building off-site relies on the use of a manufacturing facility. This therefore imposes significant capital costs for the acquisition of a factory as well as costs for operating and maintaining it; repaying the capital investment over time would therefore need to be factored into any savings. The investment cost of capital to be considered ranges based on the size of the facility that would be required as well as type of equipment and level of automation required for production; however, it was estimated that this cost falls between $50 million and $100 million based on facilities recently built (Bertram et al., 2019).

In addition to the capital investment required by the fabricator, a modular project requires more investment upfront due, in part, to a much more rigorous design phase to ensure the successful coordination of the project. Further, the reduced timeframe of a modular project also shortens the time over which the project's financing can span, thus resulting in more investment being required upfront. For example, a 2-year traditional building project might require half of the payment upfront while the remainder is spread over the 2-year construction timeline whereas a modular build that is condensed to 1-year might require all payments to be settled upfront due to the shortened timeline (Bertram et al., 2019).

With a volumetric design approach to modular construction, modules are typically stacked on top and adjacent to one another to make up the superstructure of a building. This inevitably results in doubled up walls as well as floors and ceilings resulting in a not particularly economical design and use of some materials. While this is only the case with volumetric modular construction and in projects strictly using these modules, a more economical design can be achieved using a mix of volumetric and panelized or prefabricated elements as well as in purely panelized modular projects or projects using individual prefabricated building components.

Modular construction often requires large components (e.g., walls, floors, modules) to be lifted and stacked on the structure necessitating the use of a crane thus bringing additional risk from a site safety assessment while also being expensive and not accessible for all sites and locations. Depending on the scale of the project as well as the size of individual modular components, a panelized project or project using prefabricated components might be achievable with smaller and more accessible equipment rather than a crane.

2.2.4 Temporary Structures

A "temporary", or portable, structure defines any structure that can be transported and used in a location for a time period less than that of a traditional building's expected service life which can be upward of 50 years based on the 1 in 50-year conditions design requirements in the National Building Code of Canada (NBCC 2015). Currently, there is no set design life or expected service life of a temporary structure in the Ontario Building Code (OBC) or the NBCC; however, Eurocode states that a temporary (‘Category 1’) structure should have a design life of up to 10 years (EN 1990, 2002).

The temporary building industry has been around for decades by way of volumetric portable structures; most commonly known as portable classrooms, mobile homes, and office trailers. Historically, this industry has been perceived as providing low-quality and inefficient buildings with poor insulation and leakages; however, the portable, or “relocatable”, building industry has been gaining traction as it evolves into an industry that provides flexible and adaptable structures that utilize efficient design and high-quality materials (M. J. Ribeirinho et al., 2020). Projects built for relocation have recently seen an uptake within (and outside of) Canada; thus far these projects within Canada appear to be geared toward transitional and supportive housing as they are results of the RHI funding. For example, the City of Vancouver Temporary Modular Housing Project accounts for several different sites around the city on which volumetric wood-frame modular low-rise buildings have been built. The ‘temporary’ aspect of this project is the intention for the modular units to be redistributed around the city as necessary; the buildings are all built atop an above-ground multi-point foundation system that can be reused and reconfigured to suit future sites for the project (NRB Modular Solutions, 2020). The modularity of the units themselves allow for them to be detached from the adjacent units and foundation prior to being lifted off the structure by a crane and relocated. Similarly, Durham Region initiated the Oshawa Micro-Housing Pilot Project in 2021; the 10-unit tiny home community featuring volumetric wood-frame modules was built on a regionally owned lot in the city centre and will need to be relocated within 5 years to a more permanent site, as the current one is set for redevelopment (Region of Durham, 2021).

2.3 State of Modular Construction in Ontario

The modular construction industry in Ontario has been largely implemented in residential construction due to its reduced construction timelines and the natural compatibility between repeating housing units and repetitive structural spans. These factors make modular construction an ideal solution to combat the existing housing crisis; the approach has already been extensively implemented through Canada’s Rapid Housing Initiative (RHI), Toronto’s Modular Housing Initiative, and other housing initiatives throughout the province (City of Toronto, 2023; Modular Building Institute, 2023). Table 2.1 presents a summary of recent modular housing projects in Ontario; however, it should be noted that this presents a sample of the industry and does represent the entire modular housing industry in Ontario. For each project, Table 2.1 includes the location, type of modularity, main structural system, number of units, and finally whether the project is relocatable and/or flat-packable for transportation and storage purposes (Roscetti et al., 2022).

Table 2.1 – Summary of Modular Projects in Ontario

Project	City	Module Type	Primary Structural Material	Number of Units	Relocatable?	Flat-Packable?
City of Toronto Modular Housing Initiative	Toronto	Volumetric	Wood-frame	100 (to date)		
Waterloo Region Alternative Housing	Waterloo	Volumetric	Steel (shipping containers)	6		
Oshawa Micro-Housing Pilot Project	Oshawa	Volumetric	Wood-frame	10 (to date)	✓	
JHS Supportive Housing (Carling Avenue)	Ottawa	Volumetric	Wood-frame	40		
JHS Supportive Housing (Lisgar Street)	Ottawa	Panelized	Cast-in-place concrete & steel frame	28		✓
NRCan Rapidly Deployable House Prototype	-	Panelized	SIPs & aluminum	1	✓	✓
City of London Supportive Housing	London	Panelized	Steel frame	61		✓
YW of Kitchener Waterloo Supportive Housing	Waterloo	Prefabricated	Mass timber	41		✓
R-Hauz V6	Toronto	Prefabricated	Mass timber	10		✓
R-Hauz R-Suite	Toronto	Prefabricated	Wood-frame	1 (to date)		✓

*Reproduced from Roscetti et al. (2022)

From the survey of the projects considered by Roscetti et al. (2022), it was observed that all levels of modular and prefabrication have been implemented in affordable housing projects across Ontario; however, not all projects can be efficiently transported (e.g., volumetric cannot be flat-packed) and even fewer projects were built with the possibility of relocation being considered. This implies a lack of consideration with regards to the durability of the structural systems with respect to withstanding the loading associated with reassembly and relocation along with a lack of consideration for reusable connections that can be easily dismantled.

The study concludes that the gap in the current modular industry in Ontario falls within panelized relocatable housing. The need for this stems from fluctuating housing needs and the potential for temporary emergency and transitional housing that can meet the demand and be relocated as necessary (e.g., emergency housing for Northern Ontario experiencing negative effects of climate change or vacant lots in city centres that can be used to house those transitioning out of homelessness) (Roscetti et al. 2022). A deployable panelized modular design is therefore required as it offers the highest level of prefabrication without sacrificing transportation efficiency and can rapidly meet housing needs. The design must be durable enough to withstand any Ontario site conditions along with transportation and reassembly impact while also being a lightweight and easily assembled structural system to meet different demands (e.g., remote regions with limited access to site equipment).

2.4 The Circular Economy

2.4.1 Overview on the Circular Economy

The circular economy is the concept of an economy that creates more value from resources to combat and replace the “take-make-waste” linear economy that is customary to the construction industry (ARUP, 2023b). A circular economy can be achieved by designing out waste and pollution, keeping products in use for longer, and regenerating natural systems which all aim to reduce the amount of Earth’s finite resources being consumed and disposed of in landfills. Figure 2.6 illustrates how the circular economy concept can be applied to products based on both finite and renewable resources; ideally the inner/smaller cycles of the diagram are where the product is kept in use for as long as possible before moving to the larger cycles.

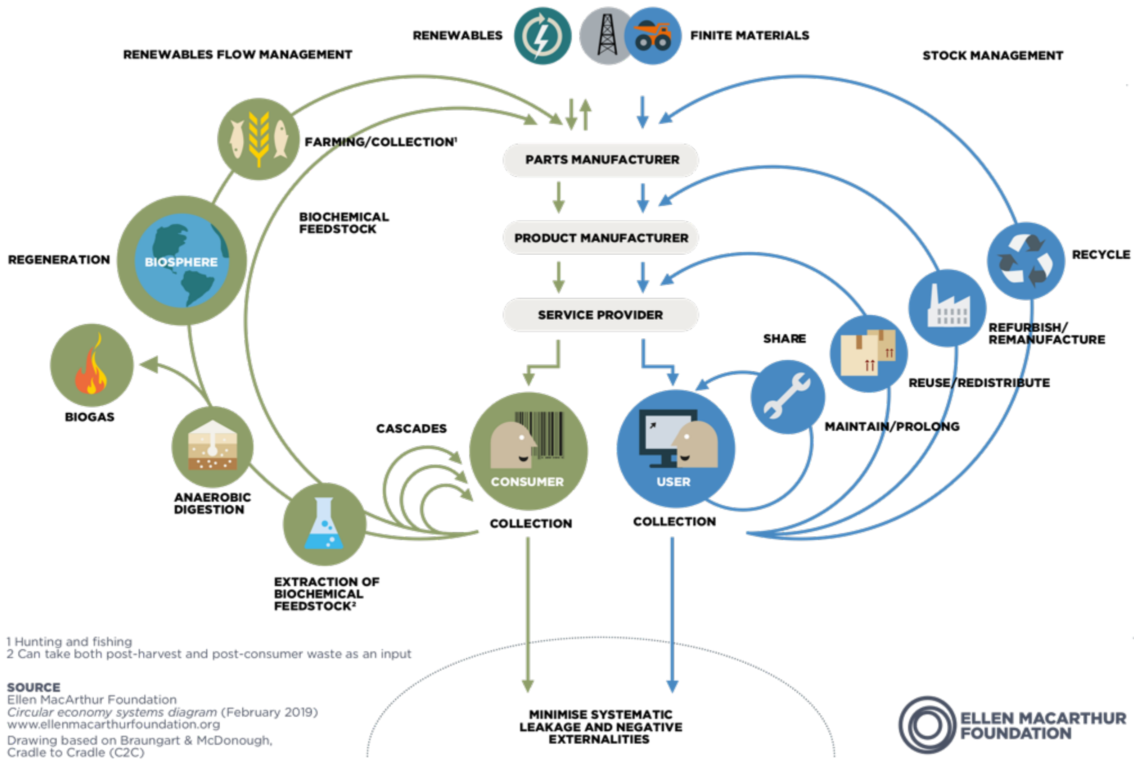


Figure 2.6 – Circular Economy Butterfly Diagram

*Reproduced from Ellen MacArthur Foundation (Ellen MacArthur Foundation, 2022b)

Transitioning to a more circular economy is particularly necessary in the building industry as this industry alone accounted for approximately 37% of global carbon emissions in 2021 (Global Alliance for Buildings and Construction, 2022). This 37% includes both the operational and embodied carbon emissions produced by the building industry; operational consisting of carbon emitted throughout the service life of a building associated with heating, cooling, lighting, and ventilation while embodied accounts for carbon emitted through production of building materials, construction, maintenance, repairs, and end of life processes (e.g., demolition, waste processing, etc.). However, over the past few decades, the industry has already made significant progress in reducing the operational carbon emissions of buildings through the push for more energy efficient systems and technology. Thus, if left unaddressed, embodied carbon will consequently make up a more significant contribution of total carbon emissions relative to the operational contribution (Kamali & Hewage, 2016). The focus is now shifting toward addressing the embodied carbon emissions of the construction industry by using sustainable materials and implementing circular strategies to the industry which will in turn reduce its overall embodied carbon emissions.

In March 2022, the Ellen MacArthur Foundation, a U.K.-based charity focused on promoting a circular economy, partnered with ARUP, an international design group, to launch the Circular Buildings Toolkit (Adlington, 2022). The Circular Buildings Toolkit provides a framework to guide developers, designers, and companies in general toward applying circularity into their projects. The framework provides strategies and corresponding action items generally focused on efficient material selection, extending material use, and ideal construction processes as well as provides case studies to further inspire the application of circular design principles (Ellen MacArthur Foundation, 2022a). The strategies provided in the framework include designing for longevity, adaptability, and disassembly. Designing for longevity promotes the idea of designing and building for durability as well as reuse in order to keep the product circulating in the economy for as long as possible. For example, a durable modular solution can be relocated to meet future demands and therefore be kept in use for longer. Designing for adaptability allows for the product to be reused in different capacities and its use can therefore be dictated based on demand. For example, the demand for a structure may vary between residential use and commercial use and modularity can be applied to a structure in order to provide the necessary flexibility in space and use. Designing for disassembly goes hand in hand with modular construction methods; when a structure is prefabricated in components, the structure can be designed such that it may be disassembled back into the original components. Additional notable strategies provided in the framework suggested reducing the use of carbon intensive materials as well as increasing material efficiency; these can also be addressed through modular construction given the ability to achieve modularity through various materials and structural systems as discussed in Section 2.2.1.

As a result of the Circular Buildings Toolkit, Futur2k partnered with ARUP in 2022 to design and build a prototype for the "ADPT" project (Futur2K, 2022). "ADPT" is a modular building system that is scalable, multifunctional, flexible, transportable, and designed for disassembly (ARUP, 2023a). Figure 2.7 illustrates the modular concept and its components. The goal for this project, ultimately, is to mass produce the adaptable system for use in building projects ranging from new-build residential, office, or education buildings all the way to topping off or expanding existing structures. The flexibility of the structure allows for it to be modified to any use thus enabling it to be re-adapted and re-allocated for different uses at the end of its required service life of its prior intended use.

Figure 2.7 – ADPT Modular System Design Concept

*Reproduced from ARUP (2023a)

The ADPT project is one of many pursuits circular economy implementation in the building industry. In a 2017 study, researchers from the Netherlands and Sweden developed an organizational collaboration tool for implementing circular design principles into a building sector project (Leising et al., 2018). The framework of the tool covers the steps to take within a project from its first design phase to the end of its initial use and preparation for reuse in which circular design principles are to be considered throughout each project phase, including, assembling a multidisciplinary tender team, adopting non-traditional contracting, and designing buildings, or 'material banks', for disassembly. The study emphasises the need for slowing material cycles in the economy by building durable, and long-lasting buildings and building components while ultimately recommending suppliers adopt take-back schemes in order to properly ensure the reuse of the product and proper disposal once the product or its components can no longer be reused.

2.4.2 Life Cycle Assessment

A life cycle assessment (LCA) is a procedure that allows for the carbon emissions and environmental impact of a product over its lifetime to be quantified (London Energy Transformation Initiative, 2017). Figure 2.8 illustrates the complete set of LCA stages as they apply to the building sector. The procedure considers all stages of a product's service, including the product's creation and construction stages (Module A), the product's in-use stage (Module B), the product's end of life stage (Module C), and finally beyond the product's service life stage (Module D). Module D refers to a circular economy in which the product may be reused or refurbished thus extending its life span within the economy and keeping the material in circulation longer. The LCA procedure applies directly to the building sector with the building's materials and components being the "products", this would include the wood or steel components being sourced, processed, and manufactured until they are ready to be transported to site for construction, which refers to the construction stage; all of which make up Module A of a life cycle assessment (London Energy Transformation Initiative, 2017).

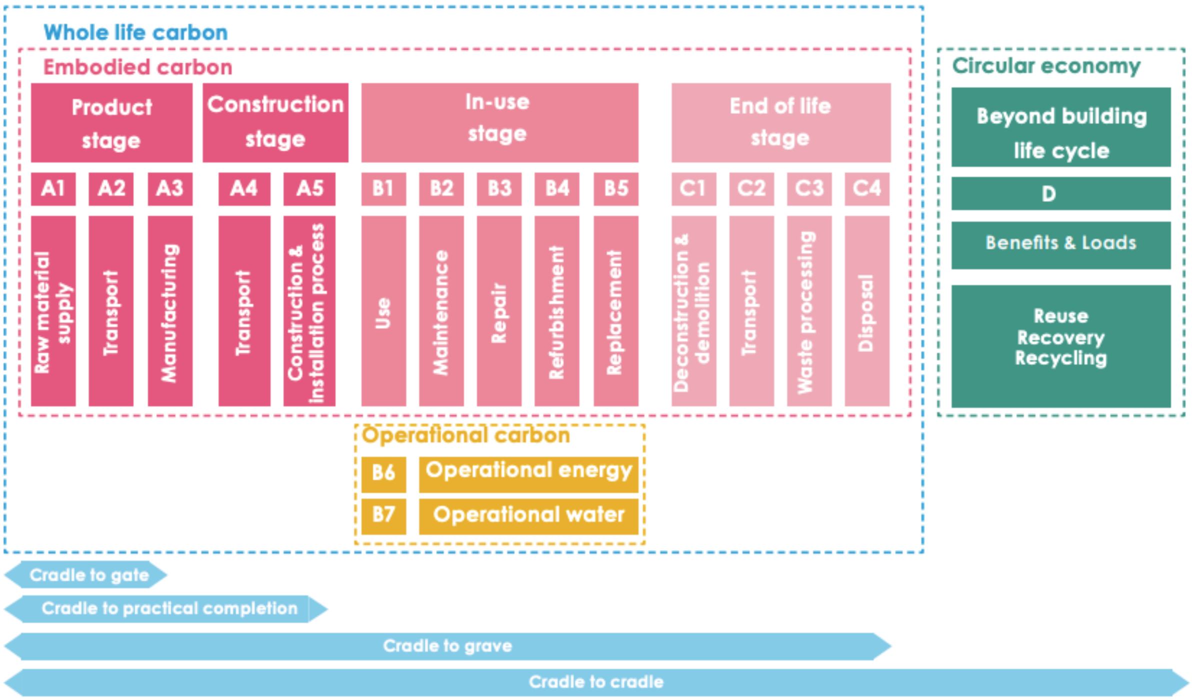
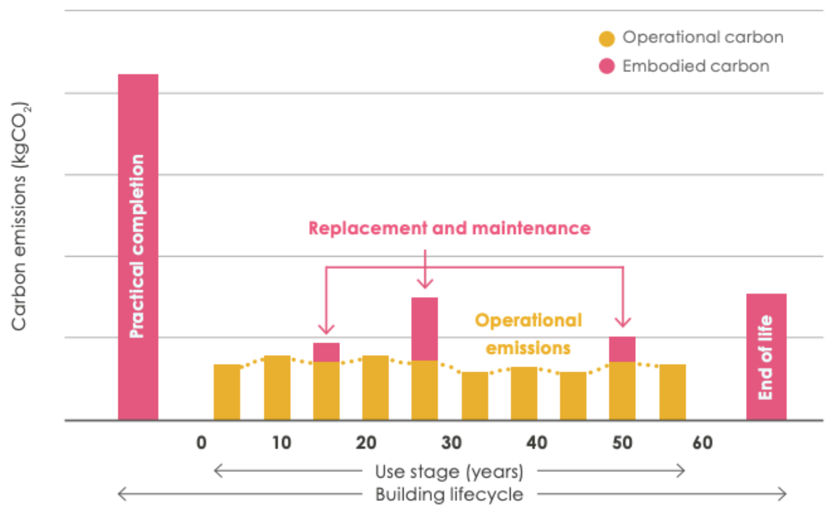


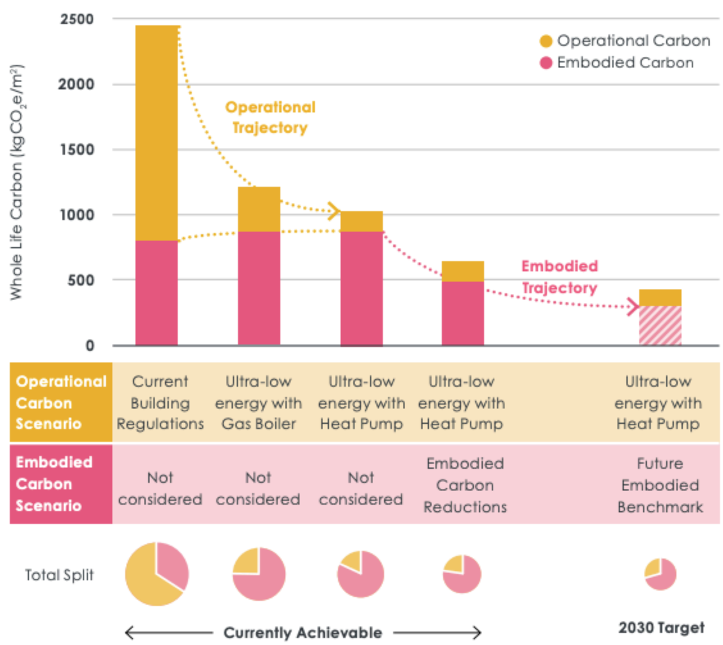
Figure 2.8 – Stages of Life Cycle Assessment in the Building Sector

*Reproduced from London Energy Transformation Initiative (2017)

As shown in Figure 2.8, the carbon emitted throughout a life cycle can be divided into its embodied carbon and operational carbon. Embodied carbon is emitted through all stages of the life cycle assessment; this includes Modules A and C in their entirety as well as any required maintenance, repair, refurbishment, or replacement throughout Module B, as shown in Figure 2.9a. Operational carbon refers only to the carbon emitted through the actual use of the building (within Module B); this includes emissions associated with heating, cooling, lighting, ventilation, etc. (Figure 2.9a). Module D does not fall within the boundaries of operational or embodied carbon emissions for the life cycle of the product as it accounts for carbon savings earned through the reuse or recovery of the product. For example, if a portion of the materials can be recovered from the end of a building's life, Module D will account for the embodied carbon that would be saved in the material extraction and processing stages (Module A) of the next project that will utilize these materials, thus the cradle-to-cradle approach versus the cradle to grave (Modules A-C).



(a) Relative Carbon Emissions over Building's Lifecycle



(b) Operational vs. Embodied Trajectories

Figure 2.9 – Carbon Emissions Breakdown of a Building's Life Cycle

*Reproduced from London Energy Transformation Initiative (2017)

Reducing operational carbon emissions has been in progress for years; however, reducing embodied carbon emissions has only recently come into focus. From a 2020 embodied carbon report, and as shown in Figure 2.9b, this is due primarily due to the current trajectory of the industry which shows how without interference, embodied carbon emissions will outweigh operational carbon emissions (London Energy Transformation Initiative, 2017).

2.4.3 Reassembly

The key to reassembly in the building sector is to first address disassembly. A structure must be designed and built with disassembly in mind in order to dismantle the structure without damaging its components. This then allows for the components to be reused or repurposed in a new project rather than demolished and disposed of, as presented in Figure 2.10. While disassembly and reuse (i.e., the Circular Economy) can be applied to any material-consuming industry, applying circular principles to an industry with a massive material consumption such as the building and construction industry could truly maximize the environmental benefits of the circular economy (Cruz Rios & Grau, 2020).

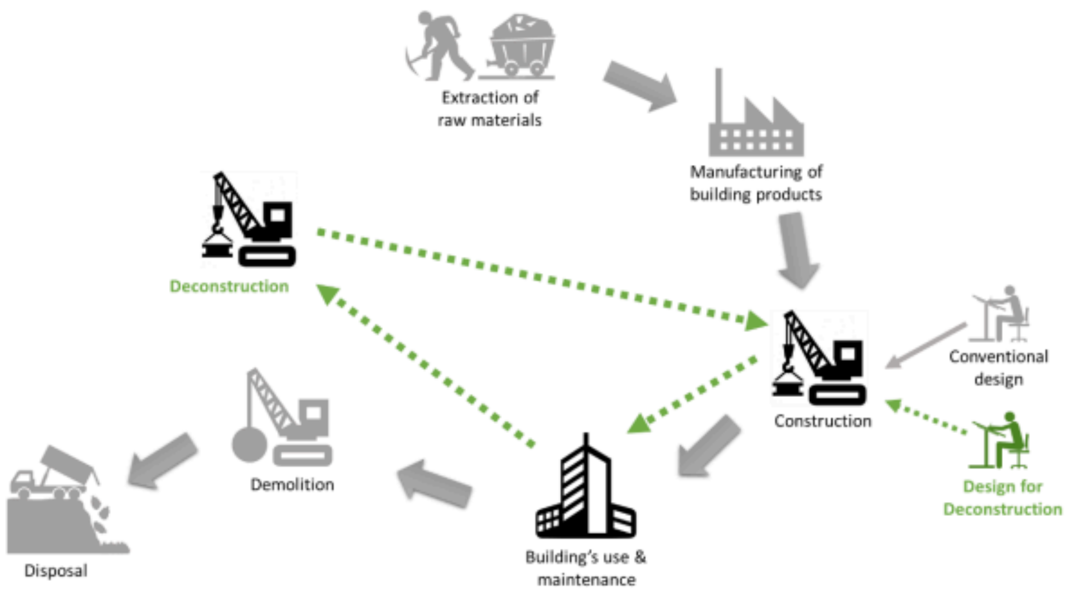


Figure 2.10 – Linear vs. Circular Economy

*Reproduced from Cruz Rios & Grau (2020)

Cruz Rios et al. (2020) investigated the implementation of the circular economy approach in the built environment and set out a list of key 'Design for Disassembly' principles; these include designing easily disassembled connections, designing for production that facilitates disassembly (i.e., modular), and standardizing components, assemblies, and dimensions among other key principles (Cruz Rios & Grau, 2020). The article goes on to emphasize how Product-Service Systems (PSS) act as a potential future driver for adoption of the Circular Economy as it aims to further dematerialize the economy and move toward a service-oriented system. The overall concepts discussed in the article emphasize the need to build using modular approaches as they serve as an ideal means for achieving a circular and dematerialized market in the construction industry. Modular structures can essentially be provided as a service, reclaimed, and properly reused or deconstructed and refurbished as necessary with this responsibility falling on the provider or owner of the product rather than consumers and occupants.

Modular buildings meet several of the key design principles set out for achieving circularity; however, a major consideration for achieving proper disassembly for reuse is reversible connections. If connections cannot be taken apart without damaging the structure or other building components, this renders the modularity useless in trying to achieve circularity. Connections such as bolts and nails can be removed with less labour than chemical or welded connections are intended for permanent applications. Figure 2.11 shows the Legacy Living Lab (L3) located in Perth, Australia and consists of a 2-storey building made from 8 steel-framed modules with a disassemblable building connection system.

The disassemblable structure was built using alternative connections like bolts over welds for the steel framing, magnets and fixed joints over glues and adhesives for ceiling and floor tiles, and a screwed fixing method for the corrugated steel roofing and cladding among other alternative connections throughout the building (O'Grady et al., 2021)

Figure 2.11 – Legacy Living Lab (L3)

*Reproduced from O'Grady et al. (2021)

The study concludes that despite the fact that demolition remains the preferred method over deconstruction and disassembly in the construction industry due to the difficulties of disconnection including lack of education and high labour rates, L3 achieved a viable and high-quality alternative to the traditional building and challenges the negative perception toward modular construction and design for disassembly (O'Grady et al., 2021). Furthermore, it was reported that in the case of the magnetic connections for the ceiling, disassembly and reassembly of the panels significantly weakened the connection thus indicating a need for additional research into connections and their ability to withstand multiple cycles of disassembly and reassembly (O'Grady et al., 2021)

2.4.4 Wood and the Circular Economy

Wood is often aligned with the implementation of a circular economy in the building industry as it is the one of the only building materials that can be sourced from a renewable resource and has an overall lower carbon impact on the environment when compared to other traditional building materials (Felmer et al., 2022; Gustavsson & Sathre, 2006). Alternative building materials such as concrete and steel require the extraction of soils, rocks, and ores from the ground, which consist of a complex process that is energy intensive with finite resources. Further, this process removes minerals from the ground and its surroundings while harvested timber holds little nutrient mineral content and therefore does not present any significant nutrient loss to its surroundings (Ramage et al., 2017). While a circular economy can be achieved using other building materials, due to finite resources, the current study focuses on wood as it presents to be a viable and sustainable solution.

Wood also has the ability to remove carbon from its surrounding environment and store it (Canada Wood, 2023). This sequestration happens over a tree's lifetime through the photosynthesis process. The carbon sequestered, or 'negative carbon', in a tree that is then used for wood in construction can then reduce the overall carbon officially emitted by the construction project as the carbon will continue to be stored within that structure. However, forest regeneration is required, at the same rate of tree cutting, in order to ensure that the sequestration process can be preserved and not be negatively affected by the trees being removed for construction material sourcing.

The life-cycle carbon emissions for a 4-storey residential building built with a concrete structure and equivalent wood light-frame structure were compared and the results indicated that a wood light-frame equivalent of a concrete structure emits less carbon into the environment and has a better overall carbon dioxide balance, as presented in Figure 2.12 (Gustavsson & Sathre, 2006). The scope and parameters of this LCA included Module A (material production) and Module C (end of life) over a 100-year lifespan and did not consider operational carbon or on-site construction as these were assumed to be similar between the wood and concrete structural systems.

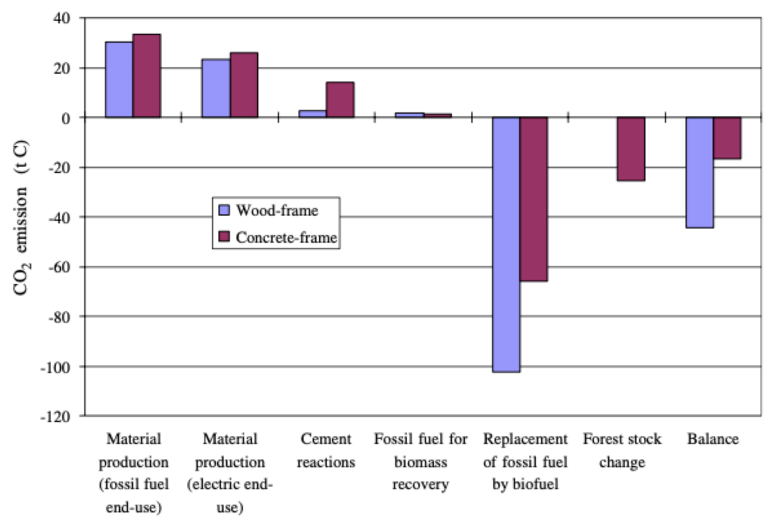
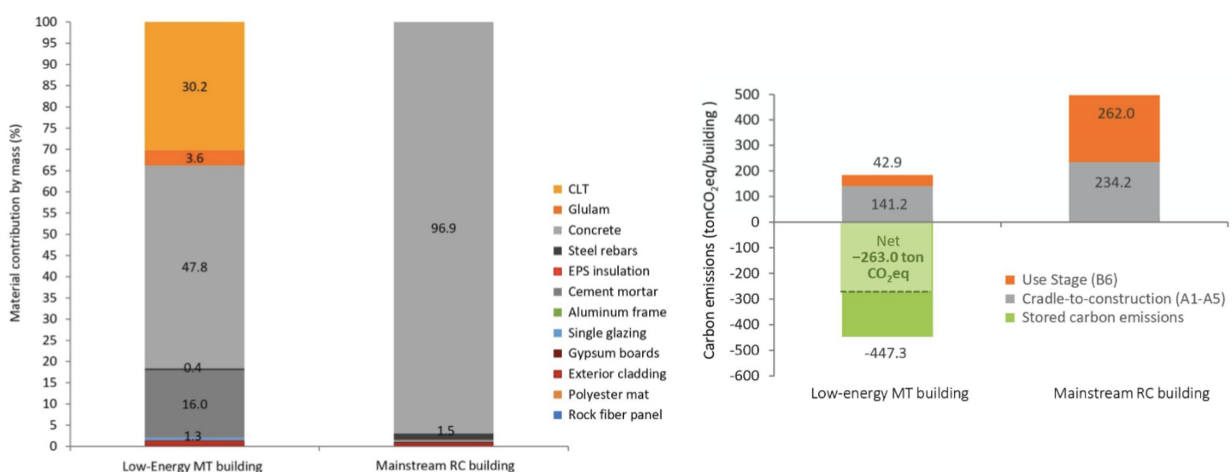


Figure 2.12 – Equivalent Concrete and Wood Structure Carbon Emissions\

*Reproduced from Gustavsson & Sathre (2006)

However, wood light-frame systems cannot always replace equivalent steel and concrete structures due to load resistance requirements and limitations on what wood-frame systems can withstand. Mass timber therefore provides a wood-based alternative to concrete and steel that presents comparable strength and structural capabilities Similarly, a comparison between a concrete and equivalent mass timber building was conducted (Felmer et al., 2022). The material breakdown for each structure is presented in Figure 2.13a and while the mass timber structure only implements the use of wood structural components for 30-35% of the overall structure, the embodied carbon emissions were 40% lower than the concrete building (Figure 2.13b).



(a) Material contributions (b) Carbon emissions by material

Figure 2.13 – Equivalent Reinforced Concrete and Mass-Timber Structures

*Reproduced from Felmer et al. (2022)

While mass timber presents a more sustainable building system alternative to concrete and steel, the use of mass timber products may not necessarily be the most sustainable alternative. For example, employing mass timber products to build a residential house that is traditionally built using dimensional lumber (i.e., light-frame construction) due to the amount of raw material required for production and added processing required to produce mass timber products, (e.g., CLT). Mass timber requires heavy machinery and resins to manufacture the product and additionally, CLT requires significant manufacturing given its physical geometry/large lamination areas thus resulting in more significant carbon emissions throughout Module A of a life cycle assessment.

The end-of-life scenario of a mass timber structure must also be considered when evaluating its sustainability and circularity over alternative materials. The use of mass timber comes with the benefit of carbon sequestration, or biogenic carbon; however, depending on the end-of-life scenario this carbon will either be partially or entirely released back into the atmosphere. When wood is incinerated, all of its biogenic carbon is released but when it ends up in a landfill, only a portion is released while the remainder is permanently stored in the landfilled material (WoodWorks, 2023). Mass timber can also be recycled at its end-of-life; however, this process is more difficult and energy intensive than it would be with an equivalent light-frame solution as the material is more heavily processed and thus harder to recover the raw material within the product. Alternatively, mass timber can be reused, which presents the most ideal way to put off its ultimate end-of-life scenario; however, this requires the use of standard and easily reused component sizes along with reversible connections that minimize any permanent alterations (i.e., damage) to the mass timber as well as a well-defined secondary application from day one (i.e., having a plan for the reuse of material after its first intended application).

A life cycle assessment was conducted on a foldable, modular CLT structural system (Figure 2.14) for residential use in Japan to assess the environmental benefits of reuse (Passarelli, 2019). The LCA parameters considered that the system was to be reused once, for a total of 2 uses, over a 60-year service life. The study found considerable decreases in global warming potential (GWP) in the reuse cases specifically considering stable or growing forest conditions; further solidifying that timber can only be considered a sustainable alternative when properly sourced. Despite proving that the reuse of CLT can increase the feasibility of its use rather than in single use scenarios the study did not consider the use of a light-frame system in their investigation. If structurally feasible, a reusable light-frame solution could provide a significantly more sustainable alternative by using less raw materials.

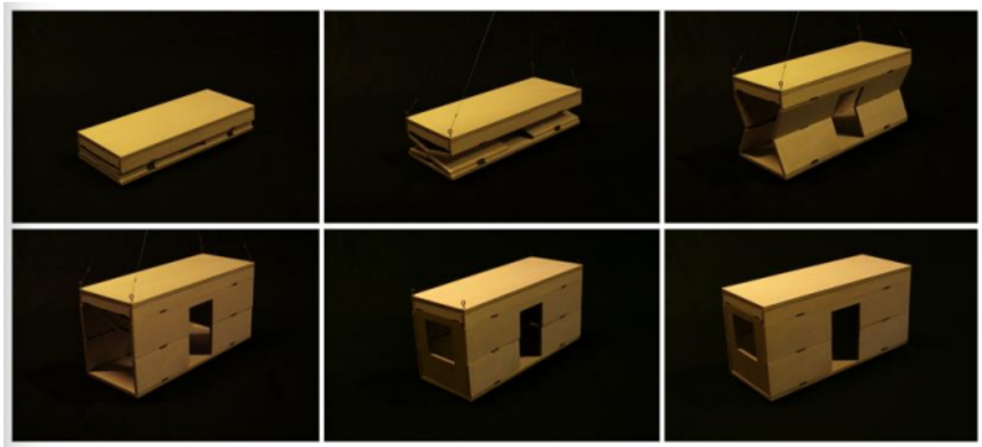


Figure 2.14 – Reusable Modular Mass Timber System

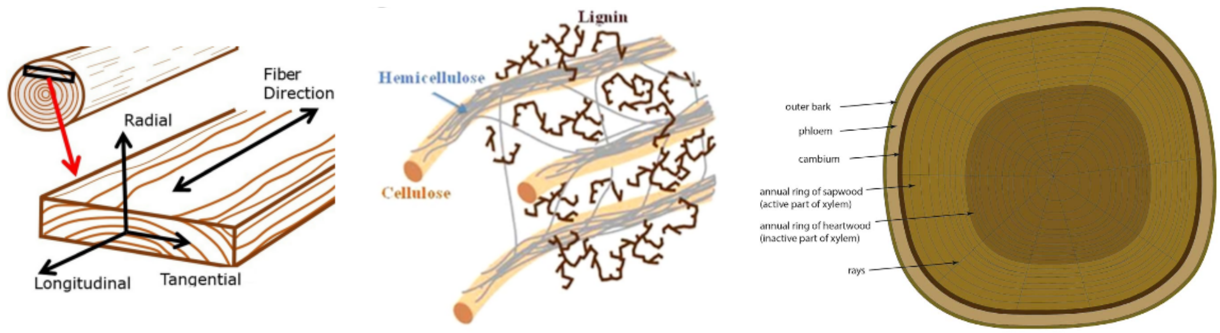
*Reproduced from Passarelli (2019)

2.5 Wood & Modularity

2.5.1 Overview on Wood

Wood is among the oldest and most widely used building materials in the construction industry to date (Smith & Snow, 2008). The renewable material has historically been used in smaller scale light-frame construction but is gaining popularity in larger scale construction due to advancement in manufacturing technologies and the wider availability of engineered wood and the mass timber products. Wood has been widely used and continues to be a material of choice as its mechanical properties and overall high strength to weight ratio offer a high performing and light-weight solution (Ramage et al., 2017; Smith & Snow, 2008).

Wood is an orthotropic material which means that its material properties are different in the three primary directions as shown in Figure 2.15a. The material is made of up fibres spanning in the longitudinal direction of the tree and are made up of cellulose, hemicellulose, and lignin, per Figure 2.15b. Trees have annual growth cycles that can be observed through growth rings in a cross section of the trunk, as shown in Figure 2.15c. Each annual growth ring will have a varying cellular/fibre structure based on earlywood (sapwood) and latewood growth (heartwood). The sapwood grows at a faster rate yielding a less dense cellular structure with thinner walls while the heartwood grows at a much faster rate yielding a denser cellular structure with thicker walls (Ramage et al., 2017)structure with thicker walls (Ramage et al., 2017).



(a) Orthotropic directions (b) Cellular structure (c) Growth ring in cross section

Figure 2.15 – Material Structure of Wood

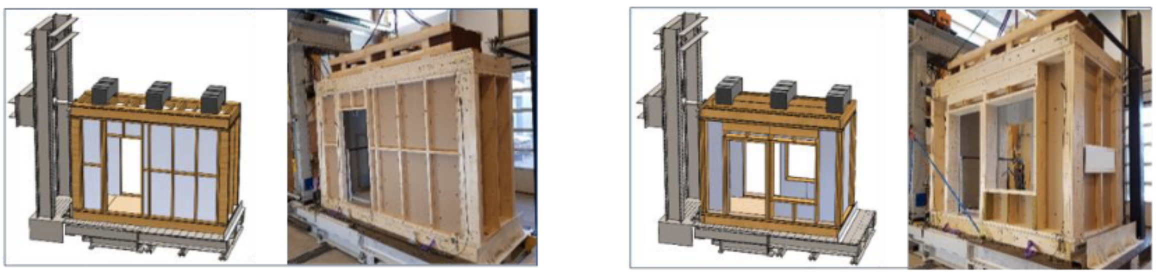
*Reproduced from Ramage et al. (2017)

When building with wood, important properties of the material to consider are the strength, stiffness (modulus of elasticity (MOE)), and the moisture content of the wood. The strength of wood is classified into various strength property categories based on the cellular structure of wood; these include bending strength, tension parallel to the grain, shear parallel to the grain, and compression parallel and perpendicular to the grain (Green, 2001). The stiffness of individual wood planks as well as well as engineered and mass timber products will often what determines if the material is suitable for a structural application given that serviceability often governs; thus, higher quality lumber (e.g., structural grade vs number 3) have larger stiffness (Legg & Bradley, 2016). The moisture content of wood is another important factor to consider when building with wood as it is a hygroscopic material. This means that the moisture content of wood fluctuates depending on its surrounding environment; within a conditioned building it will typically reach an equilibrium moisture content between 8-12%. Based on these fluctuations and the cellular structure of wood, it will experience shrinkage as it dries out, and this should always be considered and accounted for when using it as a building material (Ramage et al., 2017).

2.5.2 Light-Frame Construction

Light-frame wood construction is a prominent structural system in modular construction, particularly in the volumetric modular approach, as previously discussed in Section 2.2.1. This construction method has become a popular solution for low- to mid-rise modular residential construction within Canada (e.g., the NRB Modular Solutions developments for various RHI projects) as it is a lightweight and material efficient, and thus a cost-efficient approach that has been successfully used. Given that light-frame wood construction is widely used for residential and low-rise commercial in North America, most builders are skilled in light-frame construction and requires minimal additional training. The light-frame volumetric modular approach has also become popular outside of Canada, particularly around Europe, with various studies emerging on the design and structural behaviour of light-frame timber modules under different loading conditions (Kuai et al., 2020; Ormarsson et al., 2020).

In their design-based study on light-frame volumetric modules, Ormarsson et al. (2020) conducted experimental tests on four 1.2m x 3.6m x 3m (width x depth x height) light-frame wood modules with the primary objective of establishing the effects of different sheathing materials, opening sizes, and types of fasteners on the overall performance through measurement of shear deformations, racking stiffness, and load carrying capacity of the modules. The sheathing for module types 1 and 2 consisted of a screwed double-layered gypsum and a nailed double-layered plywood/gypsum, respectively. For each module type, one module had a door opening on each side (Fig. 2.16a) whereas the second module had two openings on each side (Fig 2.16b). Based on the test results, it was concluded that the majority of the test modules failed due to cracking in the sheathing panels as well as damage in the joints between the framing and sheathing panels.



(a) Screwed double-layered gypsum

(b) Nailed double-layered plywood/gypsum

Figure 2.16 – Light-Frame Test Modules

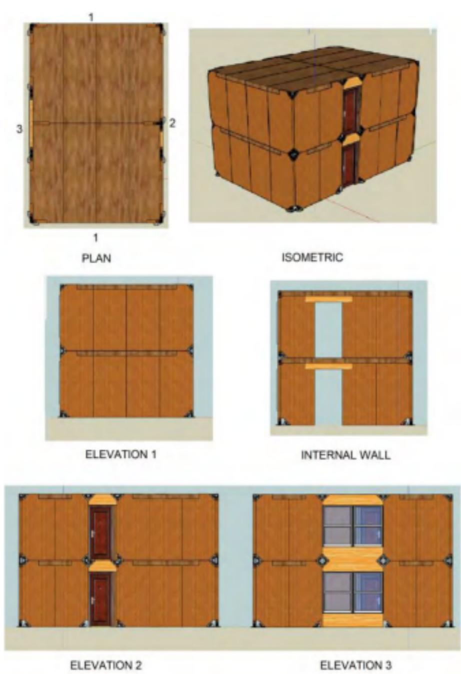
*Reproduced from Ormarsson et al. (2020)

The effects of opening size and number of openings on the lateral behaviour of light-frame modules (Figure 2.16) has shown to be significant (Kuai et al., 2020; Ormarsson et al., 2020). Potentiometers to track displacements at various points along the face of the module with the openings; the results concluded that while openings had a significant effect on the behaviour of the module loaded in shear, damages to the hold-down brackets as well as failure of the framing to sheathing fasteners also had significant effects on the displacement along the length of face and the overall movement of the module (Kuai et al., 2020; Ormarsson et al., 2020).

2.5.3 Mass Timber

Mass timber as a product itself is modular as it is processed and engineered – or prefabricated – offsite and then transported to site for construction/assembly with any pre-drilling or holes being done either in the manufacturing plant or on-site. These modular prefabricated components include mass timber products like glue-laminated timber (glulam), which is often used for beams and columns, and cross-laminated timber (CLT) which is often used for slabs and walls, among other products and applications. While any mass timber development implies a level of modularity, the overlap between the mass timber and modular industries is growing with mass timber studies and projects applying higher levels of modularity (i.e., more prefabrication) than just the manufacturing of the mass timber component offsite. The use of CLT in a deployable panelized modular structure (Figure 2.17a) was investigated; the modular design approach considered CLT manufactured from low grade timber as well as loading associated with assembly, service, disassembly, and transportation (Bhandari et al., 2020). The panels were to be cut to size off-site with chamfered corners and pre-installed components of Rothoblaas' X-RAD connector; the on-site work thus requires connecting the corners to the bases/core components of the X-RAD connector. The proposed system consists only of the structure of the building and requires the full enclosure to be installed on-site. It was concluded in the study that various 3- and 5- ply panels for walls and floors were structurally feasible in the design (Bhandari et al., 2020).

Mass timber is also present in the volumetric industry. Nordic Structures, a Canadian company that manufactures and specialises in the design of mass timber and engineered wood products, launched a project for mass timber portable buildings for schools (Nordic Structures, 2021). The modules come in large volumetric components that can be easily and quickly assembled on-site as shown in Figure 2.17b. It should be noted that due to classroom size requirements, the portable mass timber classroom concept can be scaled in size and overall layout based upon the clients' requirements, thereby demanding engineering design and analysis for custom projects.



(a) Deployable CLT Low-Rise Residential Module (b) Nordic Portable Mass Timber Classrooms

Figure 2.17 – CLT in Panelized and Volumetric Modular

*Reproduced from (a) Bhandari et al. (2020) and (b) Nordic Structures (2021)

2.5.4 Light-Frame Wood Lateral Systems

The lateral system of a light-frame wood structure is divided into two different elements, namely the diaphragm and shear walls, as shown in Figure 2.18. The diaphragm (i.e., roofs and floors) transfers the resulting loads from the wind pressure on the wind-facing side of the building to the shear walls that are parallel to the wind direction. For a one storey structure, the wind load is transferred from the wind-facing wall into the roof diaphragm and foundation (i.e., tributary area). Through the roof diaphragm, the shear forces are transferred to the shear walls which then transfer all the loads to the foundation.

The behaviour of light-frame shear wall systems subjected to lateral loads due to wind and earthquake is well established and codified (Canadian Standard Association, 2019). Figure 2.19 shows a representative section of a light-frame shear wall which consists of lumber studs (including the top and bottom plates), lumber blocking to stiffen the frame, sheathing to secure the overall shape of the frame in place, and finally nails to fasten the framing together and the sheathing to the framing. The loads are transferred through the sheathing to the framing by the connections (e.g., nails) and the transferred through the framing to the hold-downs to the foundation. The shear capacity of light-frame shear walls is thus limited by the shear capacity of the sheathing and connections between sheathing and framing (Canadian Standard Association, 2019).

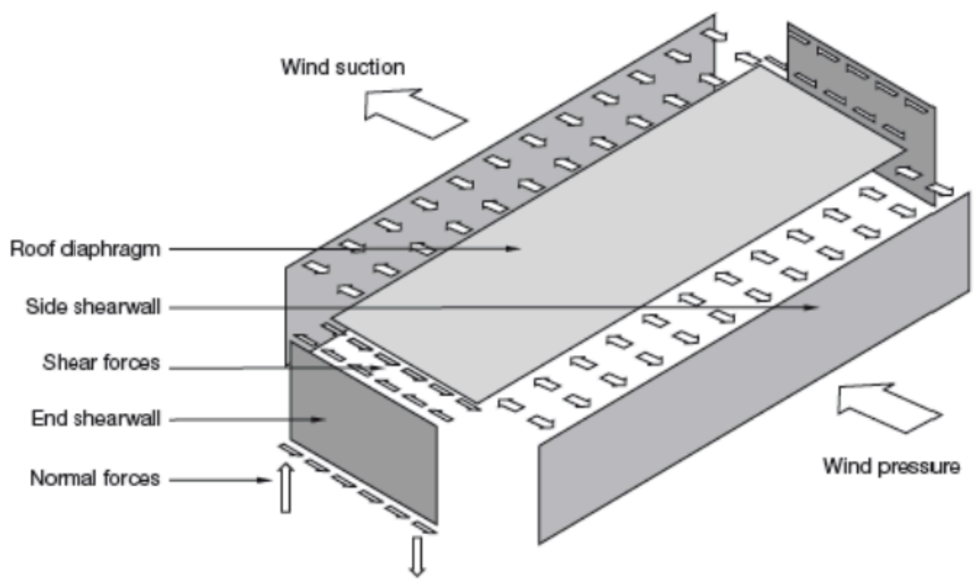


Figure 2.18 – Lateral Load Resisting Structural System

*Reproduced from Canadian Wood Council (2020)

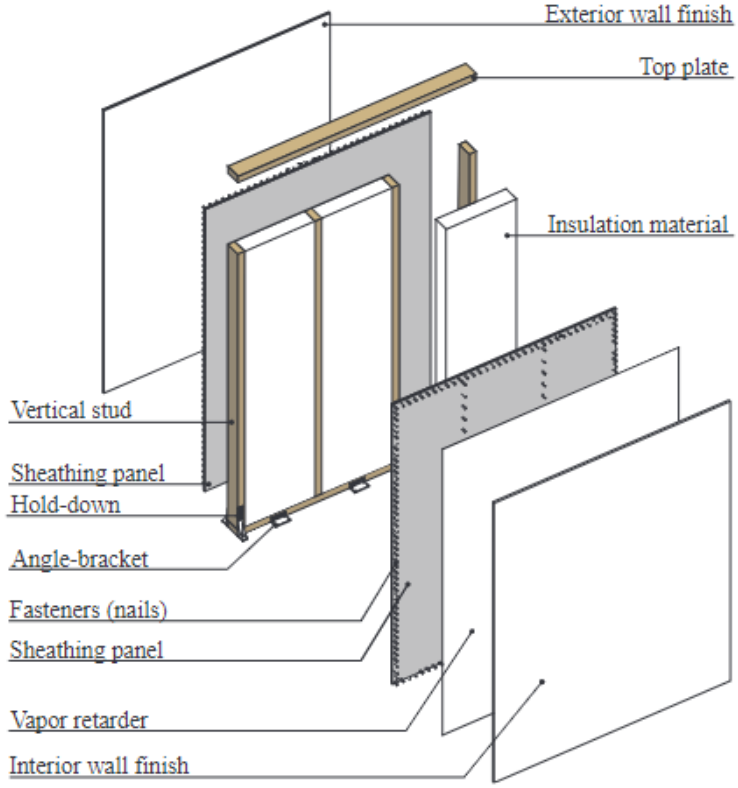


Figure 2.19 – Components of a Shear Wall

*Reproduced from Di Giangi et al. (2020)

Due to the numerous components (i.e., studs, sheathing, fasteners, hold-down) contributing to the energy dissipation, and hence overall deflection, they must be considered into the deflection equation. Reproduced from the Engineering design in wood standard (Canadian Standard Association, 2019), Equation 2-1 presents the deflection equation for a blocked shear wall; in the order presented the terms in the equation represent bending, shear deformation of the shear wall, nail slip, and anchorage slip.

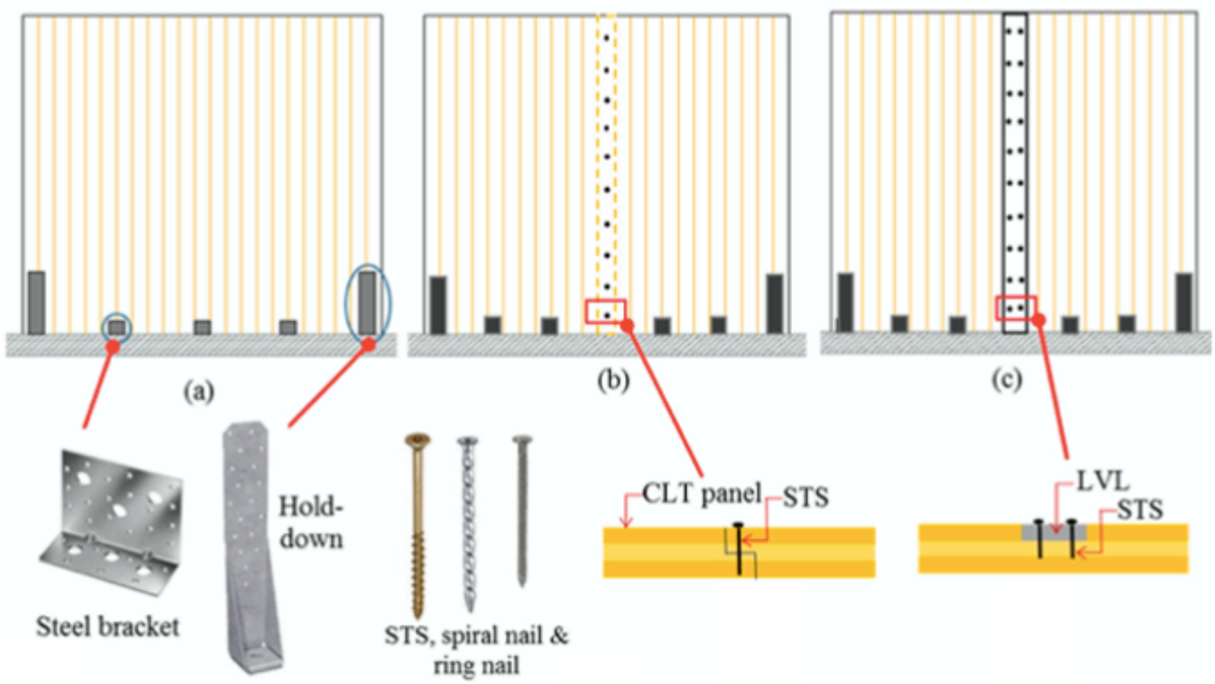
$$\Delta_{sw} = \frac{2vH_s{}^3}{3EAL_s} + \frac{vH_s}{B_v} + 0.0025H_s e_n + \frac{H_s}{L_s} d_a \qquad [2\text{-}1]$$

Where v = maximum shear due to specified loads at the top of the wall (N/mm), H_s = height of shear wall segment (mm), E = modulus of elasticity of vertical boundary framing members (N/mm^2), A = cross-sectional area of the boundary member (mm^2), L_s = length of shear wall segment (mm^2), B_v = shear-through-thickness rigidity of the sheathing (N/mm), e_n = sheathing-to-framing connection deformation (mm), and d_a = total vertical elongation of the wall overturning restraint system at the induced shear load (mm).

2.5.5 Cross-Laminated Timber Shear Walls

In CLT buildings, the lateral loads are transferred through similar mechanisms as within wood-frame buildings, via diaphragms and shear walls. The primary difference is that CLT diaphragms and shear walls are assumed to act as a rigid component with high planar stiffness that remain elastic (Canadian Standard Association, 2019). The energy dissipation in such systems come from the connections between the CLT panels as well as the connections between the CLT panels and foundation. Both rely on fastener yielding, similarly to light-frame wood shear walls. The connections must therefore accommodate the movement of the CLT as it resists lateral loads; thereby resulting in sliding and rocking of the CLT shear walls (Canadian Standard Association, 2019). For seismic design of CLT shear walls, energy dissipative connections are used for vertical joints between panels as well as wall to floor or wall to foundation connections for uplift while non-dissipative connections are used for wall to floor or wall to foundation connections for horizontal shear and connections between perpendicular walls. These are required to not yield before the energy dissipative connections yield (Canadian Standard Association, 2019). To promote a mechanism of rocking and sliding, CLT shear walls should have an aspect ratio no less than 2:1 and no more than 4:1.

The existing design provisions for determining the strength and resistance of CLT shear walls and diaphragms state that they are to be governed by the resistance of connections between the shear walls and the foundation or floor, and connections between the individual panels, using methods of mechanics. CLT shear walls can act as single shear walls, shown in Figure 2.20a, and coupled shear walls, shown in Figure 2.20b.



(a) Single Shear Wall (b) Coupled Shear Wall with Lap Joint c) Coupled Shear Wall with Spline Joint

Figure 2.20 – CLT Shear Wall Components

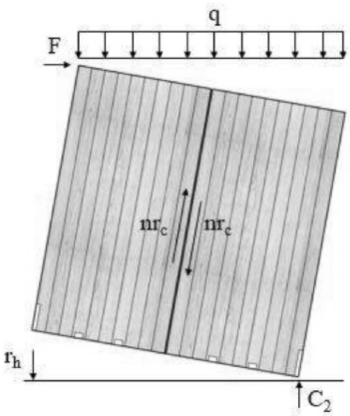
*Reproduced from Shahnewaz et al. (2018)

Single and coupled shear wall behaviour can be analyzed using Equations 2-2 and 2-3, respectively, to calculate the maximum lateral shear force that can be resisted by the shear wall, Figure 2.21 presents the variables factored into the analysis for both types of shear wall behaviour.

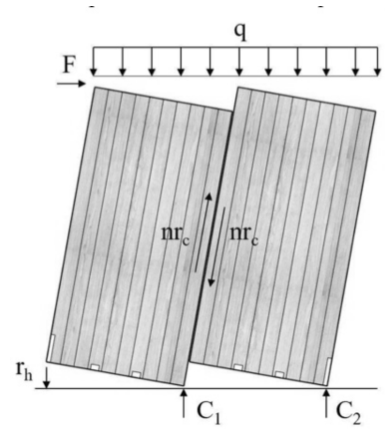
$$R_w = F = \min\left(\frac{n \cdot r_c \cdot 2b}{h}, \frac{r_h \cdot 2b}{h} + \frac{q \cdot (2b)^2}{2h}\right) \qquad [2\text{-}2]$$

$$R_w = F = \frac{b}{h}(r_h + qb + nr_c) \qquad [2\text{-}3]$$

Where b = width of wall individual shear wall segment (mm), h = height of wall shear wall segment (mm), r_h = reaction force at hold down (kN), q = distributed gravity load on shear wall (kN/m), n = number of fasteners along vertical joint, and r_c = elastic resistance of vertical joint fastener (kN).



(a) Single Shear Wall Variables



(b) Coupled Shear Wall Variables

Figure 2.21 – CLT Shear Wall Behaviour

*Reproduced from University of Victoria (2019)

2.6 Summary of Literature Review

The review of literature conducted herein has shown significant interest for reducing construction timelines through adopting some level of modular construction as well as for reducing embodied carbon emitted as a result of our infrastructure. Following a review of modular construction, as well as the trends throughout the province of Ontario, it was noted that there is a need for more research and investigation of the panelized modular approach designed for relocation, thereby implying design considerations for disassembly and reassembly. With the growth of the mass timber industry and the push toward a more circular economy within the construction industry, studies are now shifting the focus to the implementation of mass timber in the modular construction industry, particularly for reuse. While volumetric modules made of wood or any other materials are popular within larger centres, the use of these becomes impractical for locations that are inaccessible by major trans-Canadian routes. There is thus a need to investigate the feasibility of using CLT in a small-scale panelized modular building built for disassembly and reuse to establish if there is in fact a need for the use of CLT over traditional light frame construction.

Chapter 3 – Design of Prototype

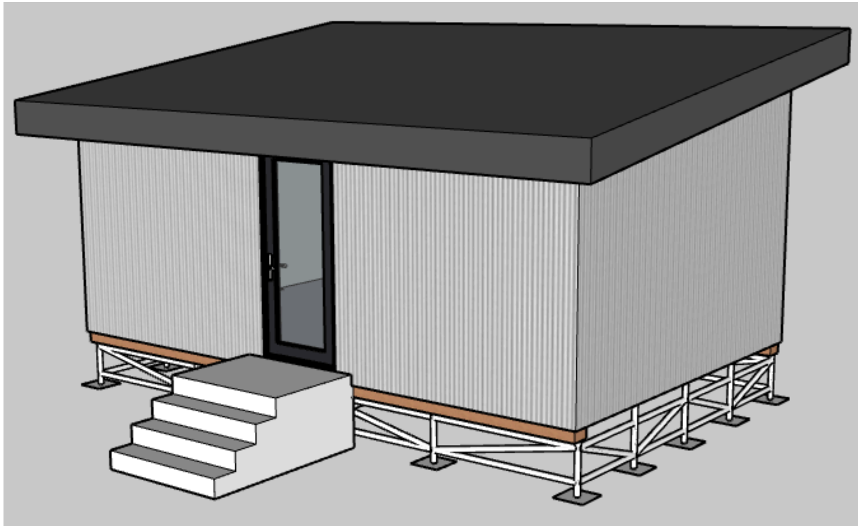
3.1 Design Approach & Purpose

Based on the literature review, it is determined that 1) there is a need for panelized modular structures designed for disassembly and reassembly that can be easily deployed to anywhere in Ontario (urban, remote, etc.) and 2) there is a need to investigate the use of CLT in this type of reusable structure in terms of structural durability and environmental feasibility. This section presents the design of a prototype structure as a proof of concept for a panelized relocatable modular building. The approach to this prototype considers four main factors: durability, ease of disassembly and reassembly, scalability, and ease of transportation. The system must be strong enough to withstand Ontario's worst-case loading conditions in order to design a truly relocatable system for the province while also being durable enough to withstand the impact associated with disassembly, relocation, and reassembly. The prototype must also be relatively easy to disassemble and reassemble to ensure that it can be efficiently deployed for use; this is to be taken into account particularly in the number of components that make up the system, the complexity of their connections, and the tools required for assembly. The system must be easily scaled to meet the necessary demand; the prototype developed is a smaller scale variation of the scalable system. Finally, the prototype should be transportable to anywhere in the province: the entire set of components should ideally fit on a standard size truck bed and fall within any weight restrictions. Throughout this chapter, a prototype that is designed for worst-case Ontario loading conditions, easy disassembly and reassembly, and overall panelized modularity will be discussed. The discussion will include the design of the prototype's structure and connections, whereas an overview of the architectural approach and preliminary design of the building enclosure are also presented here.

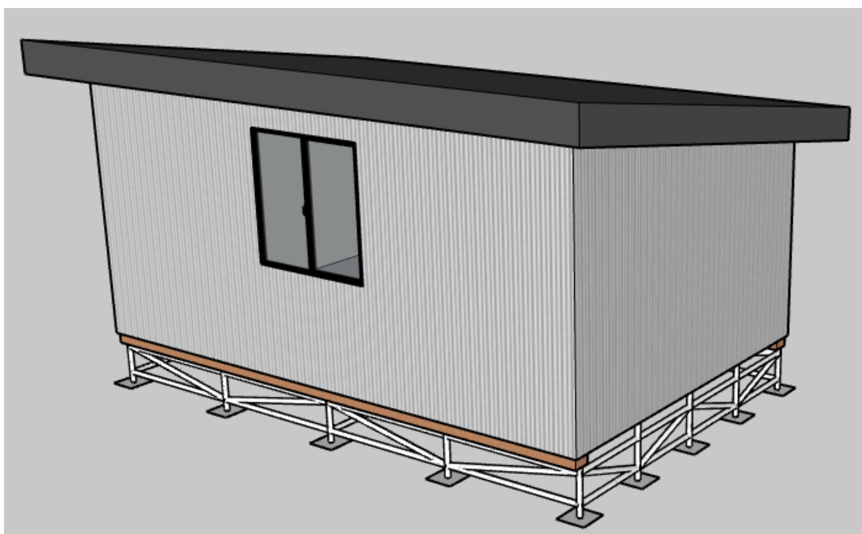
3.2 Overview of Prototype

An overview of the finalized prototype design is presented in Figure 3.1. The configuration shown is one variation that can be built from the panel set developed for mass production and scalability, which can be easily scaled up using additional panels. It should however be noted that the building can be scaled up to a maximum of 5 panels or 10 m in length for structural purposes and limitations. Extending the building past 5 panels in length would require additional internal lateral load resisting walls for the forces experienced by the long (built-up) sides of the building.

The building can be supported on beams along the longer sides which can be placed and secured onto an above-ground steel foundation system that it adjustable and relocatable, similar to the system used in the City of Vancouver's Temporary Modular Housing developments. This system concept is considered for the design; however, it and the connection to the prototype falls outside the scope of the design for this study.



a) Front/Right Side View of Prototype



b) Back/Left Side View of Prototype

Figure 3.1 – Panelized Modular Prototype

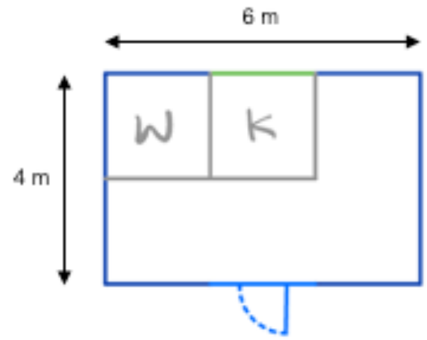
The prototype has a 4 m by 6 m footprint with a mono-sloped roof that varies the internal height of the structure from 2.2 m to 2.8 m. All panels are prefabricated with their respective enclosure secured while the connections are easily slid and bolted into place between components to ensure on-site assembly is easy and efficient. The approach taken for a modular and scalable concept relies on a panel set that uses consistent dimensions throughout for flexibility in how they are assembled. A complete set of components was conceptualized; including floor, wall, and roof panels, as well as the various connections required to connect these to each other for an assembly process that is as simplified as possible.

3.3 Architectural Design & Configuration

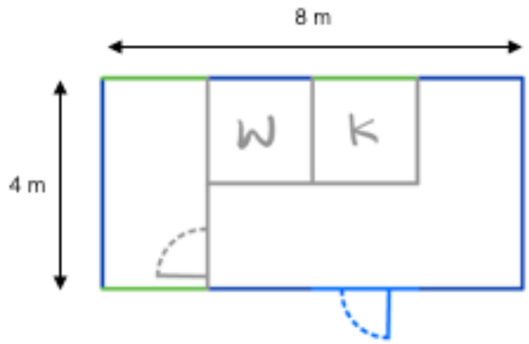
3.3.1 Architectural Approach

The architectural approach to the panelized modules was to develop a set of panels that could make up an easily scalable structure with minimal components. Based upon the literature review and motivation behind the thesis, the focus on sizing the module configurations and panel set was centred around being used as a residential space, for which three configurations can be found in Figure 3.2. It should be noted that the module can be scaled up or down in the direction of the longer dimension as the lateral design of structure and its connections is based upon two sets of two panels (two panel widths on either short side of the prototype).

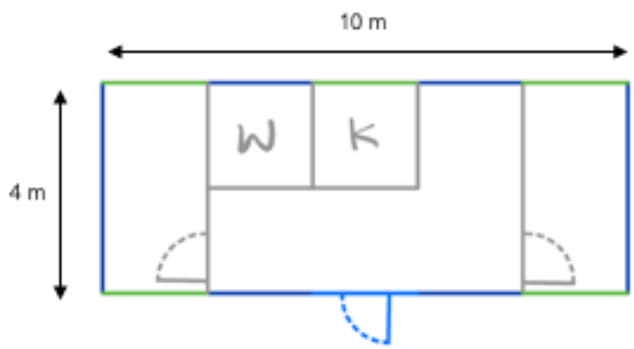
Upon further development of the conceptual prototype design, it was established that the modules could suit a range of demand with a more flexible space. Thus, while the general size of the space available in each configuration considered the minimum OBC Part 9 size requirements, the interior of the modules was kept open and would be divided and partitioned using pods inserted into the structure in addition to standard partitions (for example, bathroom pods, kitchen pods, etc.) as shown in Figure 3.3. Building services would also be contained within their respective pod inserts while services that need to be run to the outside of the module can be routed through small openings in the CLT structure. It should be noted that these openings would not be large enough to affect the overall structural integrity of the CLT. The design therefore consists of a shell structure requiring no load bearing interior walls with a sloped roof spanning across the constant width (shorter dimension) of the module. The varying colours of exterior wall panels correspond with those presented in Figure 3.4.



a) Studio Module



b) 1-Bedroon Module



c) 2-Bedroom Module

Figure 3.2 – Panelized Module Configurations

(a) Washroom Pod

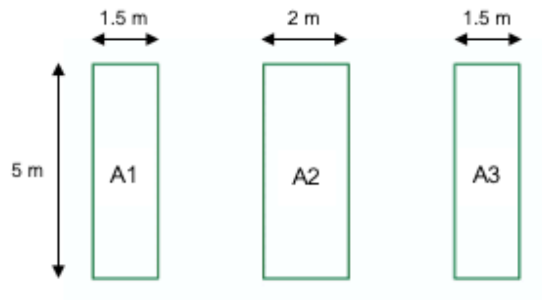
(b) Kitchen Pod

Figure 3.3 – Pod Inserts for Modules

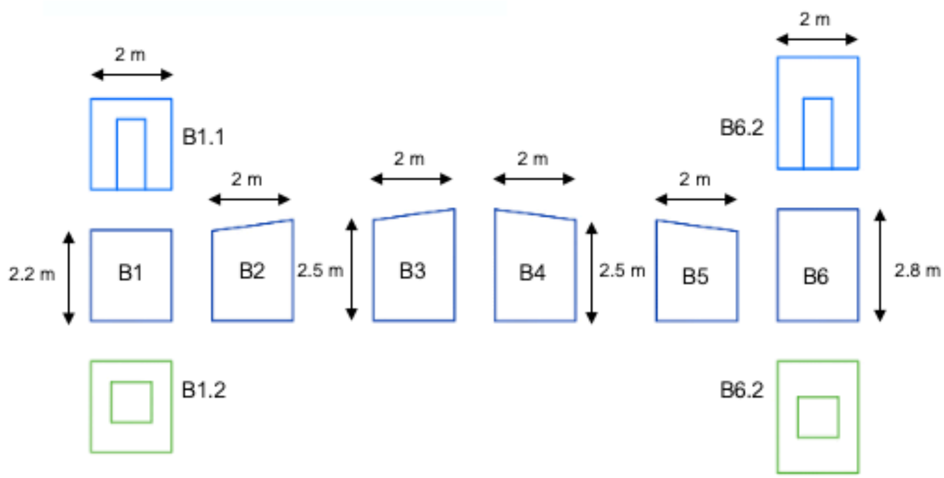
*Reproduced from (a) Howick Ltd. (2023) and (b) Bathsystem (2023)

3.3.2 Panel Set

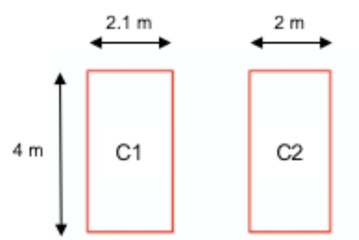
While the design being evaluated is limited to a 4 m by 6 m prototype, a complete panel set is proposed that allows the design to be scaled up or down and customized to accommodate for the type of space needed. The complete panel set is shown in Figure 3.4.



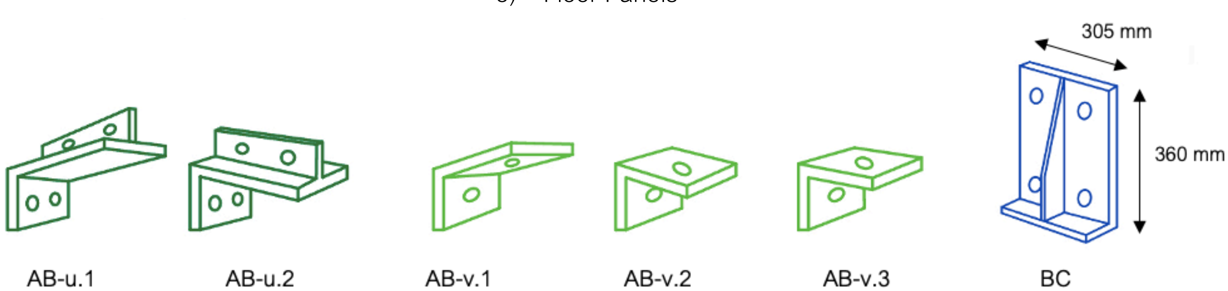
a) Roof Panels



b) Wall Panels



c) Floor Panels



d) Connections

Figure 3.4 – Set of Components

The panel sizing was limited based on the dimensions of a transport truck (O. Reg. 413/05, 2021) as well as a size that could be lifted and assembled with little to no heavy equipment. Floor panels include two different panel types: end panels that are 2.1 m by 4 m in size (to account for how the wall panels interact at corners) and middle panels that are 2 m by 4 m in size allowing them to be built up using the width of each while the 4 m length can span the entire structure. Roof panels are sized at 5 m lengths, with end panel and middle panel widths, to span the 4 m structure width (including an overhang) but include two different end panel types to account for the building enclosure layers and connections required to accommodate the slanted roof. The continuity of the panels across the entire width of the structure mitigates the need for additional joints and supports at midspan for both the roof and floor as this would create structural and building enclosure challenges for both. The wall panel types include 2.2 m and 2.8 m tall panels to make up the length of the building and a set of sloping-top panels to make up the two-panel width on either side of building and to accommodate for the slanted roof that slope down at 8.5° from the taller (2.8 m) side. Rather than keeping all wall panels dimensionally identical, the side panels and sloped roof were designed to ensure proper drainage and avoid any excessive accumulation of rain or snow on the roof, which in the case of a flat roof would lead to concerns over leakage. Further, the overhang considered when sizing the roof panels aims to reduce or mitigate any damage to the wall panels due to rain or snow by diverting precipitation away from the wall panels.

The floor and wall panels are made up of CLT structural panels while the roof panels consist of structural insulated panels. CLT was selected for the floor and wall panels due to its potential based on conclusions made in the literature review; however, it was decided that CLT roof panels would add unnecessary weight to the structure as well as pose more of a challenge when assembling on a site with limited equipment (while floor and wall panels could be placed with a forklift and propped up when necessary). Structural insulated panels, or SIPs, are sandwich panels consisting of a rigid insulation in the centre (e.g., XPS insulation) between two sheets of sheathing panels (e.g. OSB). SIPs have become a commonly used product in small-scale residential construction as they eliminate the need for on-site wood framing and provide good thermal resistance in ad(Mortgage & Corporation, 2016).

In general, panel dimensions were kept as standard and consistent as possible to allow for flexibility in the built-up structure. The current panel set allows for the structure to be scaled up in length (up to five panels long) with the 4 m width (sloping-top sides) of the structure remaining constant. All rectangular wall panels (panels 2.2 m and 2.8 m in height) can include a window opening, door opening, or no opening. The structure is designed such that lateral loads are only resisted by solid wall panels (no openings) thus the slanted panels designated for the two-panel width do not come with opening variations and further, there must be two solid panels (with no openings) along each long face of the selected building configuration. However, future iterations of this panel set may allow for flexibility in the width, length, and allowable openings with further optimization of the structure and its panel combinations. Furthermore, the addition of interior wall and floor panels may allow for the set to be built-up vertically to a achieve a multi-level structure.

The different connection types illustrated in Figure 3.4 include wall-to-roof connections (i.e., connecting A to B components) and wall-to-floor connections (i.e., connecting B to C components). The AB connections are divided into two major groups, connections design to resist uplift forces (i.e., includes “u” in naming) and connections designed to resist lateral, or shear, forces (i.e., includes “v” in naming). The AB connections are also further divided based on the specific roof-to-panel joint; this is dictated by the roof’s angle. AB connections with a “1” in the name can be used to connect B6 wall panels to the roof (greater than 90 degree joint), connections with “2” in the name can be used to connect B1 wall panels to the roof, and finally connections with “3” in the name can be used to connect B2, B3, B4, and B5 wall panels to roof panels.

3.3.3 Selected Prototype Configuration for Investigation

The final prototype that will be designed for and evaluated is a 2-panel-by-3-panel (4 m by 6 m) structure. Figure 3.5 illustrates the panel set required to make up the prototype as well as how they are to be assembled. Note that for the experimental investigation discussed in Chapters 4 and 5, panel B3 will be tested on a portion of a floor panel with the wall to floor (BC) connections installed.

Component		Quantity
Roof Panels		4
A1		1
A2		2
A3		1
Wall Panels		10
B1		3
B2		1
B3		1
B4		1
B5		1
B6		3
Floor Panels		3
C1		2
C2		1
Connections		46
Roof Uplift	AB-u.1	3
	AB-u.2	3
Roof Shear	AB-v.1	6
	AB-v.2	6
	AB-v.3	8
Wall to Floor Shear & Uplift	BC	20

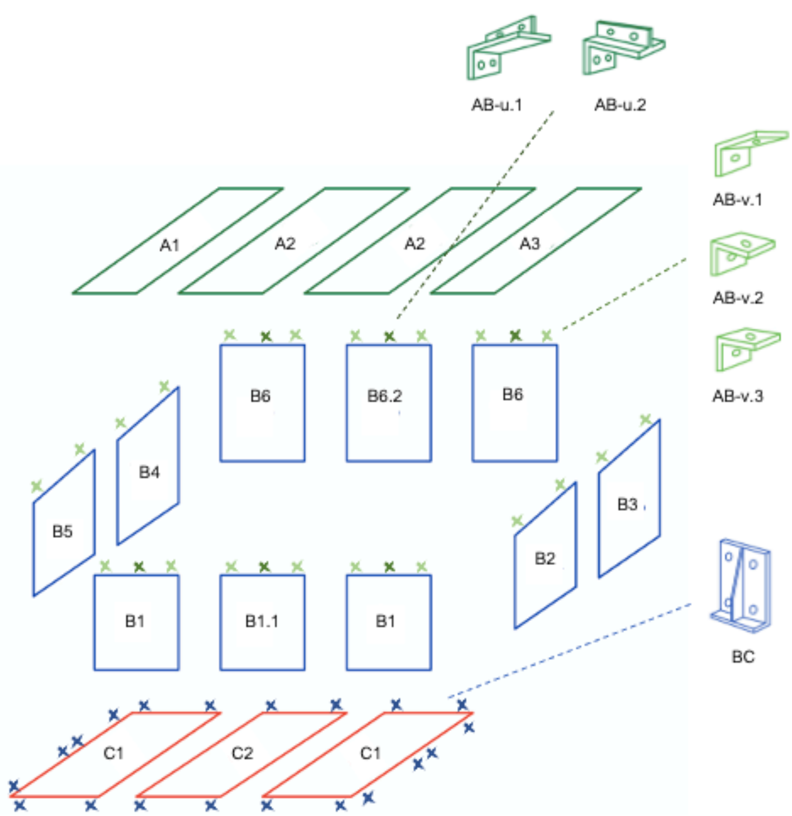


Figure 3.5 – Prototype Panel Set

3.4 Structure

3.4.1 Overview of the Structure

The focus of this study is the use of CLT in a durable reusable building as it is deemed to be the best candidate for the major supporting structural elements based on literature review. The majority of the structural system is therefore made up of CLT due to its high in-plane strength. The walls and base (or floor) panels of the structure are made up of 3-ply CLT while structural insulated panels (SIPs) were chosen for the structure of the roof. The material selection for each component is further discussed in their respective sections (3.4.3, 3.4.4, 3.4.5).

To ensure that the prototype is relocatable to any location in Ontario, the province's worst-case loading conditions, according to NBCC 2015, are considered and all components are sized and checked according to this loading.

3.4.2 Loads

The structure is designed to withstand worst-case loading conditions in Ontario to ensure it can be relocated anywhere within the province. Gravity (snow, dead, and live) and lateral (wind and seismic) loads were therefore calculated using the National Building Code of Canada (NBCC) 2015 using values associated with Ontario regions that have the worst-case conditions. The compact size of the prototype structure places it within the limitations of Part 9 of NBCC; however, NBCC Part 9 also states that it only applies to standard light-frame construction. Part 4 of the code was therefore used since the prototype utilizes an unconventional CLT and SIP structural system.

The Ontario region with the highest 1-in-50-year snow load is Chapleau according to NBCC 2015, with snow, S_s, and associated rain, S_r, loads or 3.6 and 0.4, respectively. These values are used in Equation 3-1 along with an importance factor, I_s, of 1.0 and conservative estimates for the remaining equation factors to obtain a maximum snow load of 3.28 kPa. The full calculation and factors used are summarized in Appendix A.

$$S = I_s[C_b C_w C_s C_a S_s + S_r] \qquad [3\text{-}1]$$

Where I_s = importance factor for snow loads, C_b = basic roof snow load factor, C_w = wind exposure factor, C_s = roof slope factor, C_a = accumulation factor, S_s = ground snow load (kPa), S_r = associated rain load (kPa). Dead loads imposed on the structure include the self-weight of the structure, 0.2 kPa assumed for ceiling finishes and lighting on the SIPs, and 1.5 kPa assumed on the floor panels to account for interior partitions, finishes, mechanical, etc. The self-weight of the structure is therefore calculated based on the structural design (member sizing) of the prototype which is discussed in the following subsections. The calculations for self-weight dead loads can be found in Appendix A.

In addition to ensuring that the structural system can withstand all climate-related loading conditions in Ontario, the use and programming of the structure must also be adaptable. The structure is therefore designed to withstand worst-case live loads which are designated for structures that are to be used for office or commercial space; according to NBCC 2015 a structure built for office or commercial loading must withstand up to 4.8 kPa in live load in the basement and first levels and 2.4 kPa for the upper levels (*National Building Code of Canada 2015 Volume 1*, 2015).

The maximum wind loading on the structure was calculated using Equation 3-2 and a reference velocity pressure, q, of 0.55 kPa for Kincardine, Ontario (maximum 1-in-50-year value for Ontario). The resulting maximum wind load on the structure considering an importance factor, Iw, of 1.0 and conservative estimates for all remaining factors was found to be 1.81 kPa. The worst-case orientation is considered to conservatively design the structure and connections; the wind pressure is therefore assumed to act on the largest face of the prototype. The full calculation for wind loads, including all factors used and load paths can be found in Appendix A.

$$p = I_w q C_e C_t C_g C_p \qquad [3\text{-}2]$$

Where I_w = importance factor for wind load, q = reference velocity pressure (kPa), C_e = exposure factor, C_t = topographic factor, C_g = gust effect factor, C_p = external pressure coefficient. The maximum seismic loading was calculated based on the values in Orleans, Ottawa as it experiences the worst seismic conditions within Ontario. The minimum lateral earthquake design force was calculated using Equation [3-3] below in order to compare the resulting force with that calculated for wind to obtain the governing ultimate (ULS) and serviceability (SLS) loading. The snow load considered in this calculation was reduced from 3.28 kPa to 2.5 kPa as none of Ontario’s regions that experience snow loads above 2.5 kPa fall within high seismic regions. This also ensures that the prototype is not significantly overdesigned when placed in regions without high snow or seismic loads (majority of Ontario). The minimum lateral earthquake design force was found to be 40.7 kN for the 2-bedroom configuration (configuration C) of the prototype which translates to a load of 4.9kN/m along the diaphragm. Details on the inputs for seismic loading calculations can be found in Appendix A.

$$V_s = S(T_a) M_v I_E W / (R_d R_o) \qquad [3\text{-}3]$$

Where $S(T_a)$ = spectral response acceleration for the fundamental period T_a (m/s^2), T_a = fundamental lateral period of vibration (s), M_v = factor to account for higher mode effect on base shear, I_E = earthquake importance factor, W = dead load (kN), R_d = ductility related force modification factor, R_o = overstrength related force modification factor. The imposed lateral loads (wind and seismic) are considered for the 4 m by 10 m prototype design configuration (configuration C). Members and connections are sized according to the resulting loads and thus limiting the flexibility of the prototype to a length of 10 m. Should the building be further scaled up, additional lateral-load resisting interior walls would be required.

3.4.3 Roof Structure

Structural insulated panels (SIPs) were chosen for the roof structure as these provide a light-weight alternative to CLT and have shown to be an effective solution in resid(Bharti, 2022; Statistics Canada, 2022). Within Ca. While CLT and its durability is the focus on the relocatable prototype design, it was established that with minimal equipment and no crane on remote sites, placing CLT panels on the roof of a structure would pose a challenge due to the self-weight of CLT. Furthermore, with a heavier roof structure, seismic loading would have ultimately governed over wind loads; this would have therefore increased the already high 'worst case' loads that were calculated the structure – and specifically the connections – would need to withstand.

The roof of the prototype is to be slanted at an 8.5° angle to allow for drainage and must span the entire width of the building to remove the need for intermediate supports within the space. The building's width is limited to 4 m (i.e., 2 wall panels) as the flexibility of this prototype is provided by the ability to increase the length of the building. With the span kept constant, only one SIP length is required for all roof panels and the building can be expanded lengthwise by using more of the same panels.

A SIP product was selected based on manufacturers local to Ontario; however, given the simple assembly of a SIP these can be easily sourced from local manufacturers in other areas, provinces, etc. For the purpose of the prototype, the roof SIPs were selected based on product made by Thermapan Structural Insulated Panels, a manufacturer based in Fort Erie, Ontario. A roof SIP with double lumber splines was selected for the prototype as it provides sufficient structural support for the imposed snow loads over the full span of the structure (Thermapan Structural Insulated Panels, 2022). The selected product is 311 mm thick, can simply-span up to 4.87 m while satisfying up to a L/260 deflection limit, and comes in 1.2 m-wide panels. The SIP product can however alternatively be premanufactured in 2 m wide panels with an intermediate double lumber spline built in. This alternative keeps the span between splines within that of the "off the shelf" product ensuring that the structural capabilities of the product remain intact: a cross section of the SIPs is presented in Figure 3.6. The purpose of the modified SIP width is to ensure better alignment between the roof and wall panels as well as consistency in placements of connections. The design table used for SIP selection along with complete load calculations can be found in Appendix A.

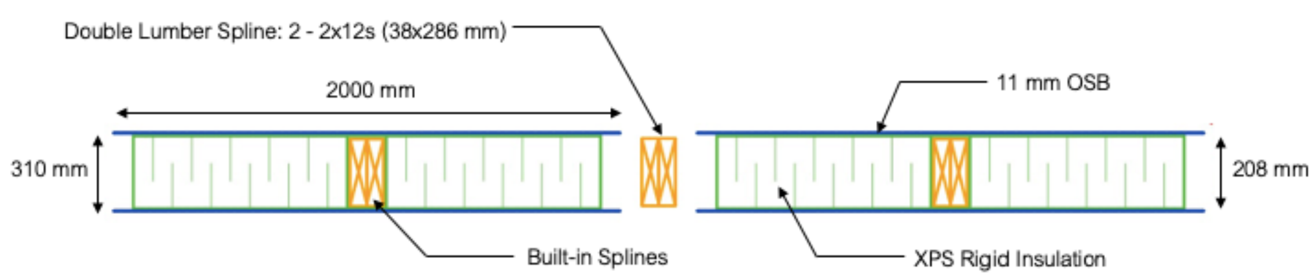


Figure 3.6 – Roof SIP Cross-Section

3.4.4 Wall Panels

CLT was selected for the wall panels of the prototype as it is anticipated to be more durable than light-frame in the case of repeated disassembly and reassembly. This is the case as it has fewer connection points than a stud wall and therefore fewer potential locations for failure of connections (e.g., nails connecting sheathing to framing and nails in framing) This is theory is based upon the literature reviewed in Chapter 2 which indicated that the performance of modules consisting of light-frame wood walls was largely affected by failure in the framing to sheathing connections (i.e., nails). The durability of CLT in comparison to light-frame wood walls will therefore be evaluated through experimental testing and discussed in Chapters 4 and 5.

The prototype, and variations of the prototype (i.e., scaled up configurations) can be assembled using a combination of 6 different wall panel types, including two panel types with window and door opening variations (refer to Figure 3.4). These panels account for the slanted roof shape along the width of the structure while also allowing for the length of the structure to be extended outward by using the desired number of rectangular (B1 and B4) panels along the parallel long faces of the structure.

Based on the imposed dead, wind, and snow loading, 3-ply (105 mm) E2 grade CLT meets all of the required design checks, including connections. Whereas a thinner 3-ply CLT (87 mm in thickness) would also provide sufficient support to withstand the imposed loading on the panels at a cheaper cost and less material usage due to the consideration of the worst loading across Ontario, the 87 mm panel would not pass all connection design checks. The connection design process is further discussed in Section 3.5.

3.4.5 Floor Panels

Similar to the roof panel assembly, the floor panels are designed to span the full width of the structure thereby reducing the number of joints between panels and the number of locations that need to be supported from below. The floor panels; however, are designed using CLT rather than SIPs (unlike the roof). Placing CLT floor panels into place is anticipated to be manageable using simple tools/equipment (i.e., a common forklift) unlike CLT roof panels that would likely require a crane to be lifted into place. Like the wall panels, the floor panels are likely to experience more damage due to lateral and gravity loading in addition to the impact of disassembly and reassembly.

To reinforce the concept of flexibility within the building's programming, the 4.8 kPa live load associated with office and commercial spaces was considered; however, an actual load of 4.8 kPa is not typical in a standard building, and even more unlikely to be experienced by the relatively small size of the prototype. The live load is therefore reduced to 2.4 kPa and considered alongside an imposed 1.5 kPa of dead load to account for interior partitions, mechanical systems, finishes, and other fixtures within the structure. The resulting factored gravity load imposed on the floor panels can be easily supported by E2 grade 3-ply CLT (the same product as used for the wall panels). Should the desired design load be 4.8 kPa, the floor panels can simply be switched out for 5-ply CLT with a slight modification to the wall-to-floor connection to account for the extra thickness associated with a 5-ply CLT floor. Complete calculations and design checks for the imposed loading for the 3-ply CLT panel can be found in Appendix A.

Figure 3.7 presents the floor panel layout for the selected prototype configuration. In scaling the building up in size, up to two additional interior floor panels (C2s) can be added. Based on the floor panel layout and the simply supported span of the panels, the structure can be supported by two glulam beams along the long edges of the structure that can then sit on the chosen foundation (e.g., an above-ground adjustable steel truss foundation system).

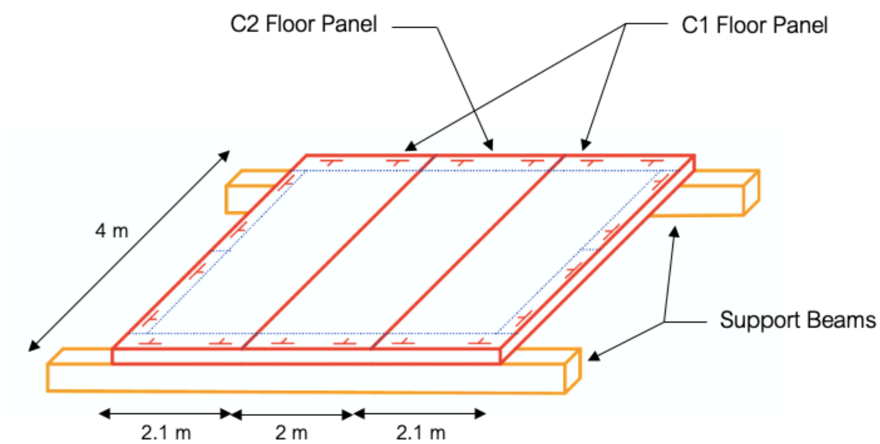


Figure 3.7 – Floor Panels

3.4.6 Alternative Light-Frame Wood Panel Design for Experimental Testing

For experimental testing purposes, a light-frame wood panel was developed to be tested and compared to the prototype's CLT panel system. The light-frame panel is representative of a standard exterior wall construction method, consisting of a SPF 38 x 140 mm (2x6) spaced at 400 mm on centre stud wall with double studs on the end, two rows of blocking, and sheathed on one side with 12.7 mm SPF plywood. The stud wall was designed initially considering the same gravity and lateral loads as the CLT panel, but ultimately overdesigned to achieve a more rigid panel that relies on its hold down connections rather than having failure modes governed or largely affected by the connections between panel components (i.e., framing to sheathing connections). This therefore allows for a more comparable system to the behaviour of a rigid CLT panel that is governed by the behaviour of its hold down connections. Calculations are presented in Appendix A. Additional information on the implementation of this panel in the experimental testing can be found in Chapter 4.

A light-frame wood floor panel was designed alongside the light-frame wall panel as the connection between wall and floor panels was selected for testing. The floor panel would consist of 38 x 286 mm (2x12) SPF joists spaced consistently with the studs in the wall panel and sheathed with 19.05 mm SPF plywood sheets. Design calculations can be found in Appendix A and further discussion on the implementation of this component in the experimental testing program is presented in Chapter 4.

3.5 Connections

3.5.1 Overview on Connections

The design and analysis of all connections, while falling outside of the focus and general scope of this study, were necessary to explore in order to achieve a proof of concept for the relocatable panelized prototype. In a system designed for disassembly and reassembly, connections play an important role in enabling the modular components of the structure to be taken apart and put back together with little to no damage. Connection design was therefore explored to the extent necessary to complete the study. The wall-to-roof connections were explored and conceptualized in terms of shape, placement, and assembly function. The wall-to-base connection (component BC) was conceptualized, designed, and sized to meet the required loading in order to manufacture and use for experimental testing. The wall-to-base bracket was conceptualized to ensure feasibility and functionality in the system but was not materially optimized due to uncertainties in the actual behaviour (Chapter 4).

3.5.2 Wall to Base Connection Design

The wall-to-base connection (component BC) was designed to be a workable solution for the system that could be fabricated and used in experimental testing for the study. The conceptual design of the bracket prioritized easy installation. The resulting bracket can be slotted up through the CLT base panel and bolted into the edge of it; the CLT wall panel is then slotted onto the bracket and can then be bolted into place from the inside of the building. The installation of the wall panel also secures the bolt in the base edge into place ensuring that the external portion of the connection is concealed for security purposes. Figure 3.8 presents the steps described to assembling the wall to base panel connection.

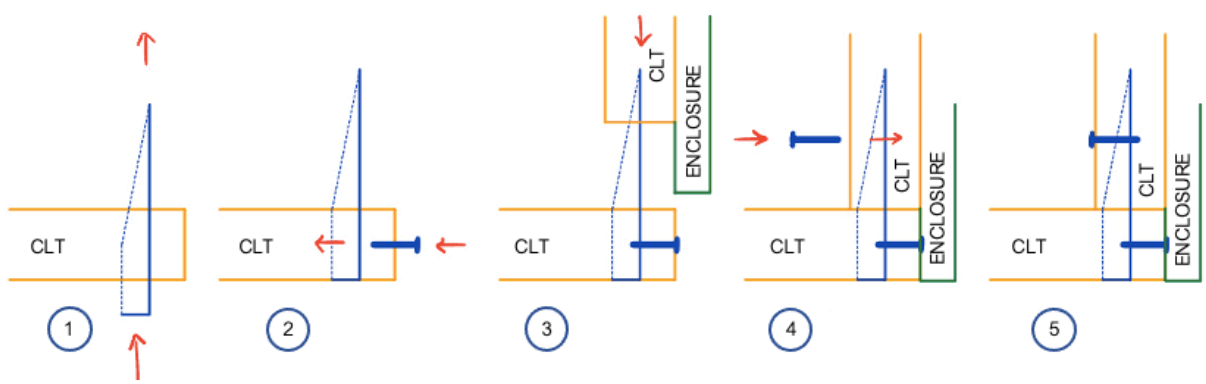


Figure 3.8 – Wall to Base Connection Steps

It was established that each wall panel would be connected to the floor panel directly below using two steel bracket connections as shown in Figure 3.9. The connection count was minimized to only two connections as they were able to resist both shear and overturning forces generated by lateral loads. The wall bears on the floor panel which bears upon the glulam support beam below for gravity loads. Figure 3.10 presents a sketch of the individual bracket shape.

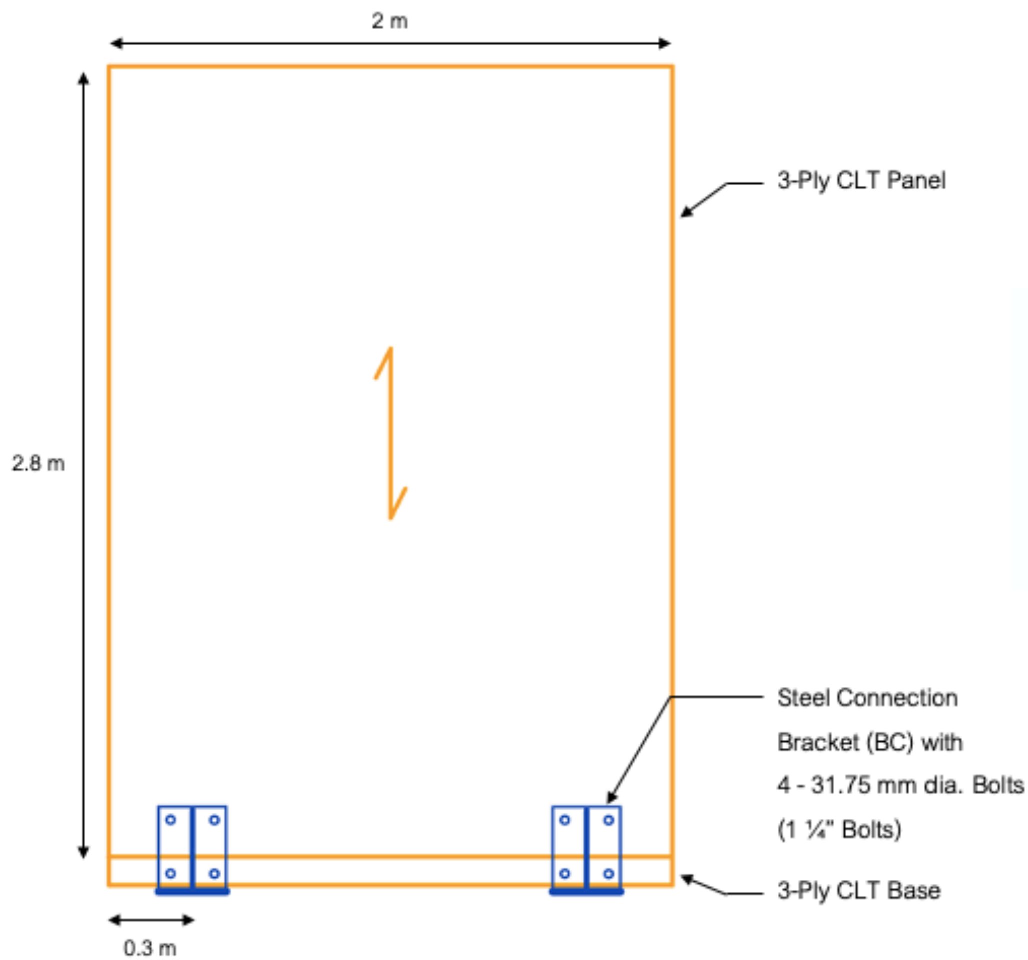


Figure 3.9 – CLT Wall to Base Assembly

Similar to the approach of the structural member design and sizing, worst-case loading conditions were considered to ensure the connection would work between any wall and floor panel in the prototype system. As previously mentioned, wind loads governed in terms of lateral loading; the wind pressure was then applied to one of the longer faces of the structure to impose the largest overturning loads on the wall panels acting as shear walls. These maximum overturning loads, along with maximum gravity loading and out-of-plane lateral loading against the wall panels were applied considering worst-case load combinations to ensure the bracket passed all necessary wood and steel design checks. The prototype is limited to a minimum building width of two panels in order to carry the prescribed design loads. The complete load calculations and design checks can be found in Appendix A.

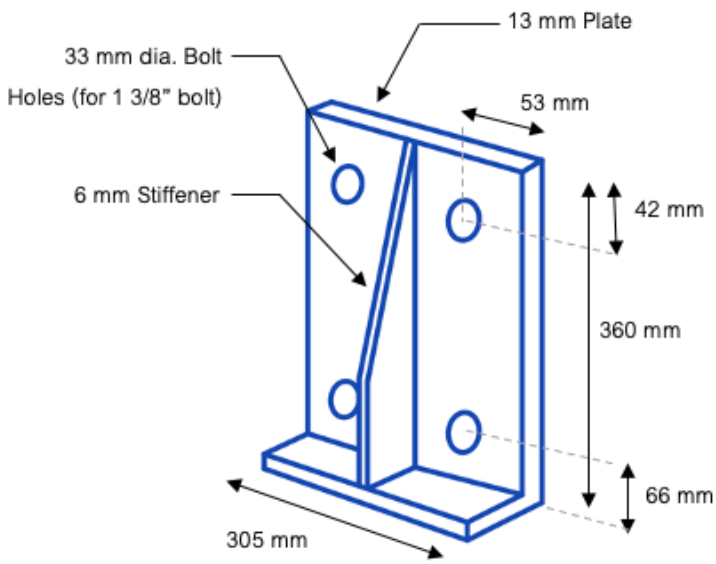


Figure 3.10 – Wall to Floor Connection BC

It should be noted that while the bracket design was an iterative process, this was only iterated until the bracket could pass all required structural checks. The connection design was completed as a placeholder that could be fabricated and used for experimental testing focusing on the durability of panels and was thus not optimized. The bracket design could be further optimized to achieve greater load resistance, more efficient material use, and potentially an overall lighter weight connection. Fabrication drawings for the steel bracket can be found in Appendix A.

3.5.3 Wall-to-Roof Connection

The wall-to-roof connections (AB-u and AB-v) have been preliminarily designed for proof of concept of the prototype. The connectors have been conceptually designed to ensure functionality, or ease of disassembly and reassembly, but have not been fully resolved according to the required design checks. Connection design of the wall-to-roof connection falls outside of the scope of this study in this case as it is assumed that they could be resolved to the degree that the wall-to-floor brackets have been within the same general sizing range as the wall-to-floor brackets.

The conceptualized wall-to-roof uplift connections (AB-u components shown in Figure 3.11) consist of steel connections to secure the roof panels to the wall panels and resist uplift forces; these are to be sandwiched between the SIP splines as they could be bolted through the spline in order to transfer loads through the spline rather than the insulation portion of the SIP. The second set of wall-to-roof connections (AB-v components shown in Figure 3.12) consist of the diaphragm connections. These brackets would be placed at the top corner of each wall panel, inset similarly to the wall-to-base (BC) brackets. These are to be bolted into the SIP as well as the CLT wall in order to resist lateral loads (i.e., shear) by transferring them through the OSB portion of the SIPs. Figure 3.13 presents the joint that implements the AB-u connections while Figure 3.14 presents a roof plan with the locations of both connection types.

The AB-u.1 and AB-v.1 connections are to be placed on the wall to roof joint edge along the long side of the building that coordinates with wall panels B, B1.1, and B1.2 (or the shorter side of the building). Similarly, the AB-u.2 and AB-v.2 connections are to be placed opposite to AB-u.1 and AB-v.1 connections, along the wall to roof joint edge of the taller side of the building. Finally, the AB-v.3 connections are to be placed along the shorter, slanted sides of the building along the wall-to-roof joint edge.

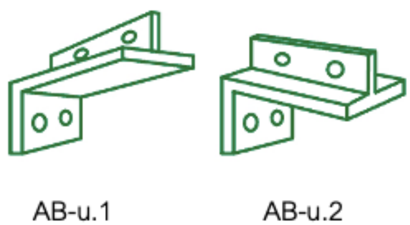


Figure 3.11 – Wall to Roof Uplift Connections

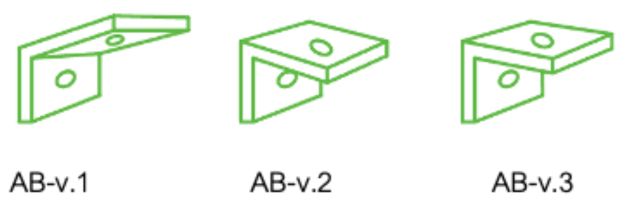


Figure 3.12 – Wall to Roof Shear Connections

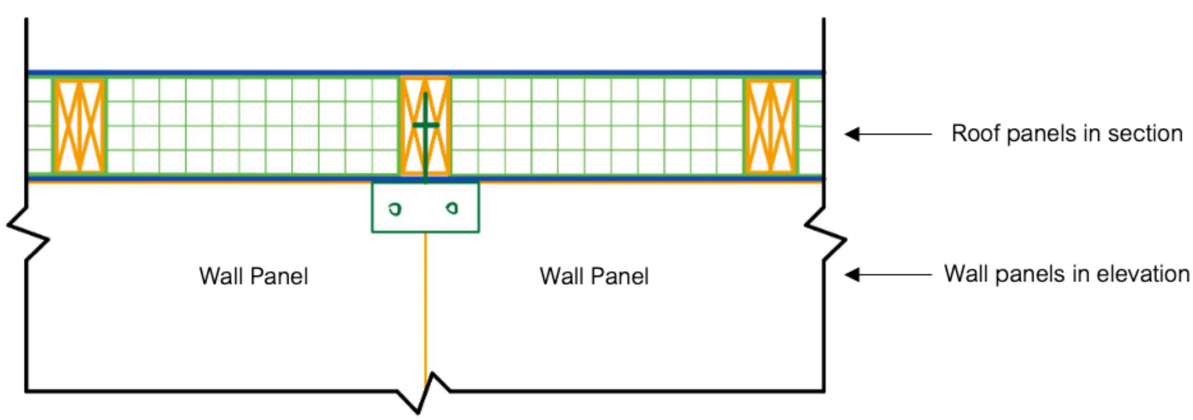


Figure 3.13 – Wall to Roof Panel Uplift Connections

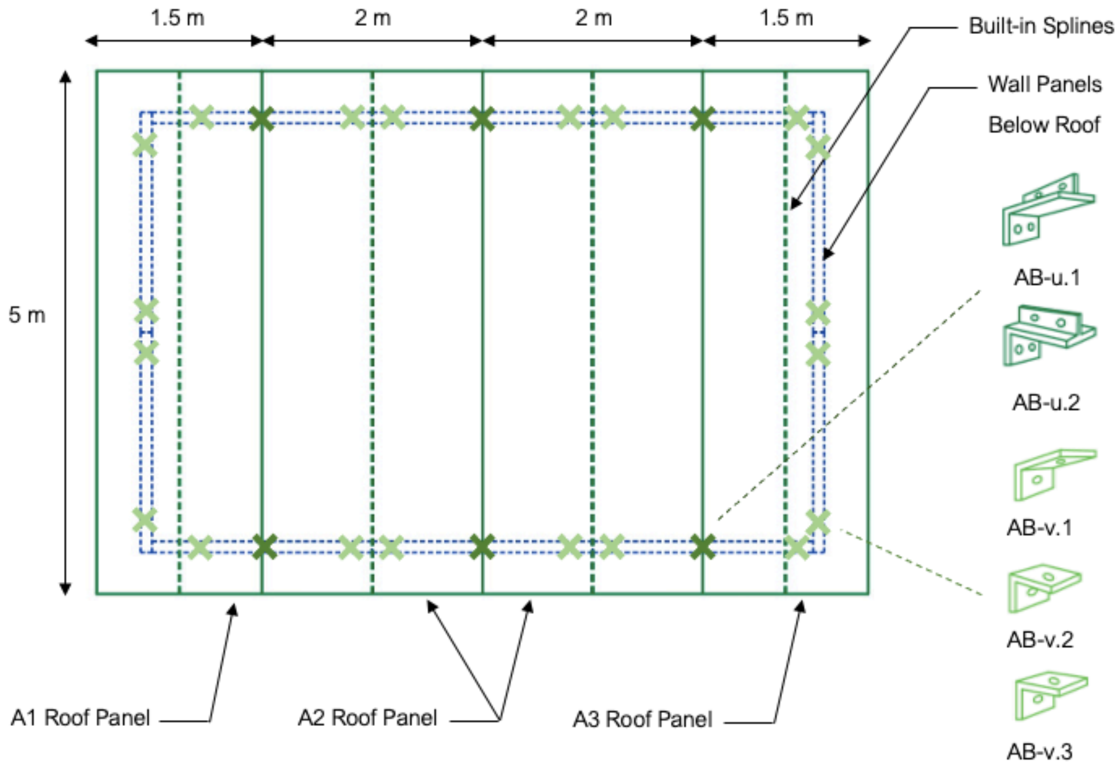


Figure 3.14 – Roof Plan with Connection Locations

3.6 Building Enclosure

3.6.1 Overview on Enclosure

While the focus of the prototype design centres around the structural durability of the panelized system, basic building enclosure assemblies are presented for completion purposes. The prototype must be compatible with a functional enclosure assembly given that the basis of the prototype is a panelized system that evidently presents enclosure continuity challenges. Enclosure assemblies for each of the panelized components of the prototype are selected based on commonly used materials and products in the North American construction industry with the consideration that complete enclosure assemblies will be secured to the panelized components before being transported to site. The enclosure assemblies are selected to meet a minimum RSI of 10.5 m^2K/W for the roof, 4 m^2K/W (of which RSI 1.7 must be continuous insulation) for the walls and 6.6 m^2K/W (of which RSI-1.3 must be continuous insulation) for the floor based on climate zone 7 requirements from the Ontario Building Code (Ontario Building Code, 2012). The approach to the enclosure design considers a sufficiently insulated assembly for all of Ontario's conditions that can be modified to suit the panelized components without greatly sacrificing thermal resistance between panels. It is however, understood that the pre-installed enclosure assemblies will require some on-site steps to create continuity in the entire enclosure; this will be briefly conceptualized in the discussion of each panel assembly but is not designed in detail as it falls outside the scope of this study.

3.6.2 Roof Assembly

The selection of layers for the roof assembly first considers the overall expected thermal resistance value; RSI- 10.5 (m^2K/W) as well as the structural component already selected. In this case, the SIP that was selected as the structural component for the roof panels, made up of EPS insulation between 2 layers of OSB, has an RSI of 13.2 m^2K/W, surpassing the RSI-10.5 standard. The remaining layers in the roof assembly are therefore selected for other functions, including waterproofing, durability, and low self-weight rather than for their thermal resistance values.

An air- and water-resistant barrier roofing membrane is to be installed on each roof SIP over the exterior side of the panel, wrapping over both sides encasing the insulation to ensure any water seepage cannot infiltrate the insulation. Further, an additional strip of roofing membrane can be taped along the edges of the panels with some overhang to allow for it to be overlapped with adjacent roof panels on site in order to provide some level of continuity over the surface of the roof. Horizontal furring strips are secured to the WRB using cap screws attaching the exterior roofing material to the structure. Corrugated steel was selected as the exterior roofing material as steel presents a lightweight and durable solution that comes in the form of panels rather than a shingle roofing approach. Corrugated sheets were selected over standing seam as the cycles of reassembly would leave the flat surface of the standing seam steel more visibly damaged or dented than the corrugated steel which ultimately outweighed the higher strength (but also significantly higher cost) of the standing seam steel option. Figure 3.15 presents the final roof panel assembly.

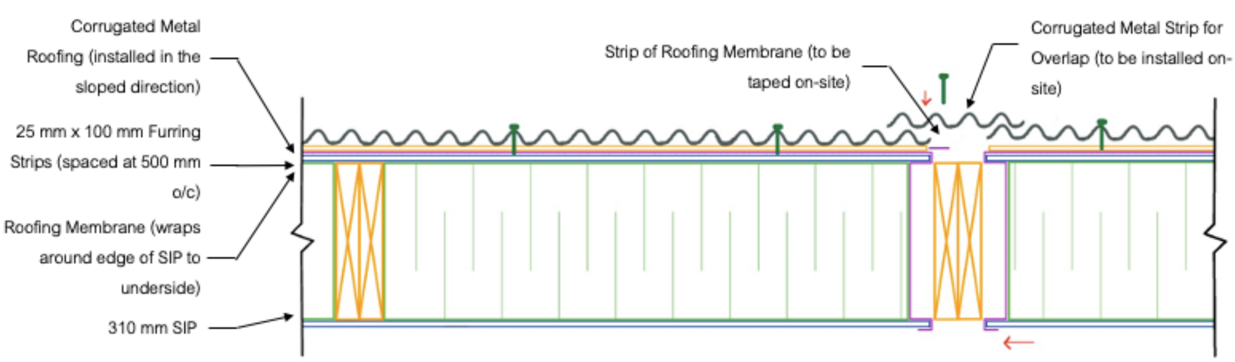


Figure 3.15 – Roof Panel Cross Section & Joint Detail

3.6.3 Wall Assembly

The wall assembly was approached similarly to the roof assembly, first considering the expected RSI value of 4 m^2K/W and then considering the structural layer that has already been chosen. The structural component of the wall panels is a layer of 3-ply (105 mm) CLT. Assuming that CLT has an RSI value of 0.84 m^2K/W per 100 mm of thickness, the 105 mm CLT layer provides and R-value of about 0.88 m^2K/W (Structurlam, 2022). The insulation layer was then selected considering the remaining RSI-3.12 required to achieve an RSI-4 wall assembly as well as the need to select a rigid insulation board to be compatible with a CLT structure. XPS (extruded polystyrene insulation) is chosen for the wall assembly which provides up to RSI-0.029 m^2K/W per mm of thickness (John Straube & Eric Burnett, 2005). A wall assembly with 3-ply CLT and 125 mm (approximately 5") of XPS can provide a total RSI value of 4.5 m^2K/W not including the small contribution from other layers in the assembly; the complete assembly and its layers can be found in Figure 3.16. An air- and water-resistant barrier is to be installed between the insulation and the CLT to protect from condensation or water infiltrating the structure. A vapour barrier is not required as the XPS insulation acts as a vapour impermeable material at thicknesses greater than 25 mm (Lstiburek, 2004). Finally, similar to the roofing material, corrugated steel cladding is selected for the exterior wall finish.

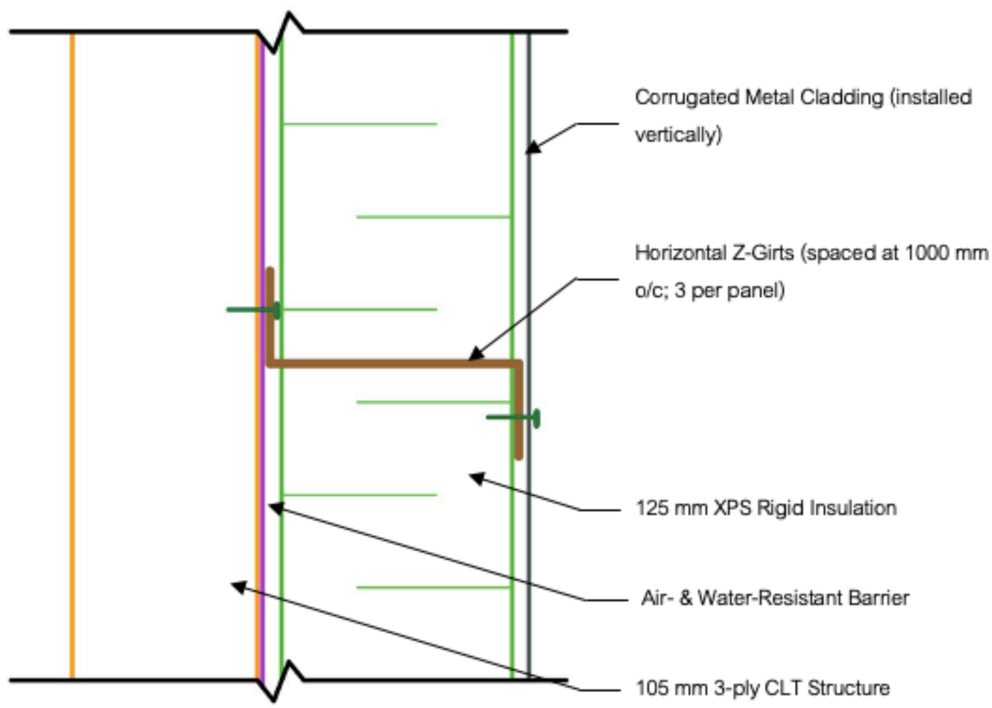


Figure 3.16 – Wall Cross Section

Similar to the roof SIP joints, the CLT walls panels are to be wrapped in an air- and water-resistant barrier on the exterior side of the CLT extending over the edges toward the inside of the panel. The joint along the outer face of the XPS is to be sealed using a strip of vapour impermeable tape to ensure that the XPS can act as a contiguous vapour barrier. Further, a corrugated metal strip, like that in the roof panel assembly, can be included and installed on-site over the joints between wall panels to shield the joint from exterior conditions (i.e., rain or snow). In addition to the proposed on-site modifications components (e.g., taping an additional barrier strip along the joint), the joints may also be lined with a rubber gasket to ensure when the panels are secured in place at their connections, the rubber gaskets are compressed to form an air seal. Figure 3.17 presents the wall-to-wall panel joint between all parallel walls.

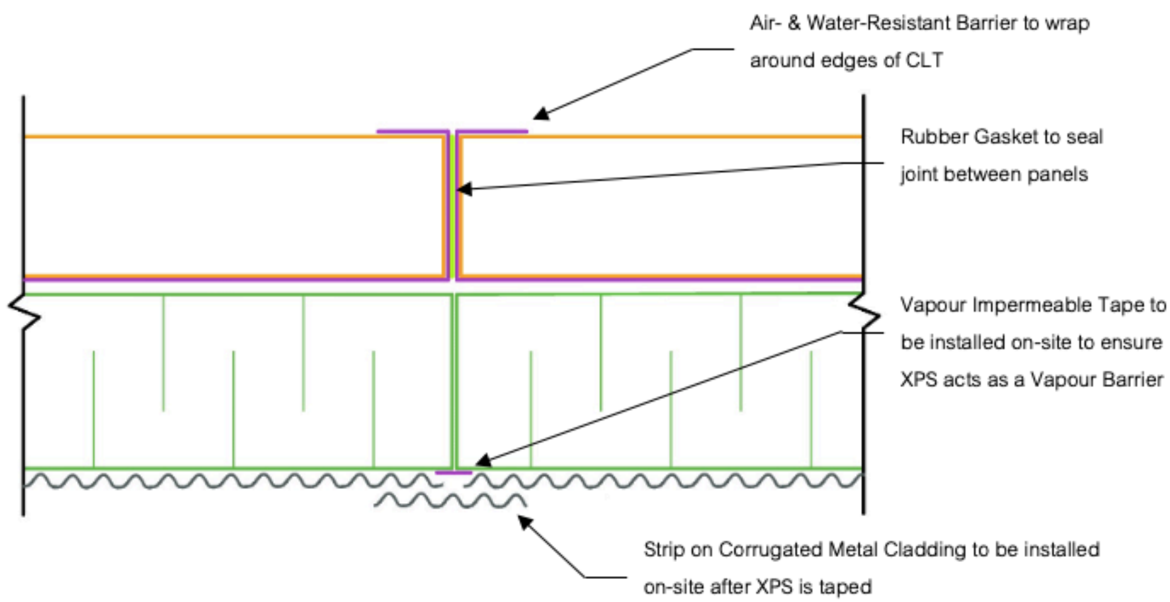


Figure 3.17 – Wall-to-Wall Panel Joint

3.6.4 Floor Assembly

As the floor assembly separates a conditioned space from an unconditioned space, the assembly is similar to that of the wall assembly (which also separates conditioned and unconditioned spaces) but with a 200 mm (approximately 8”) thick layer of XPS insulation as this achieves the required RSI value of 6.6 m^2K/W (including the contribution from the CLT) as well as a panelized PVC material for the exterior finish (on the bottom of the building). The complete assembly can be found in Figure 3.18.

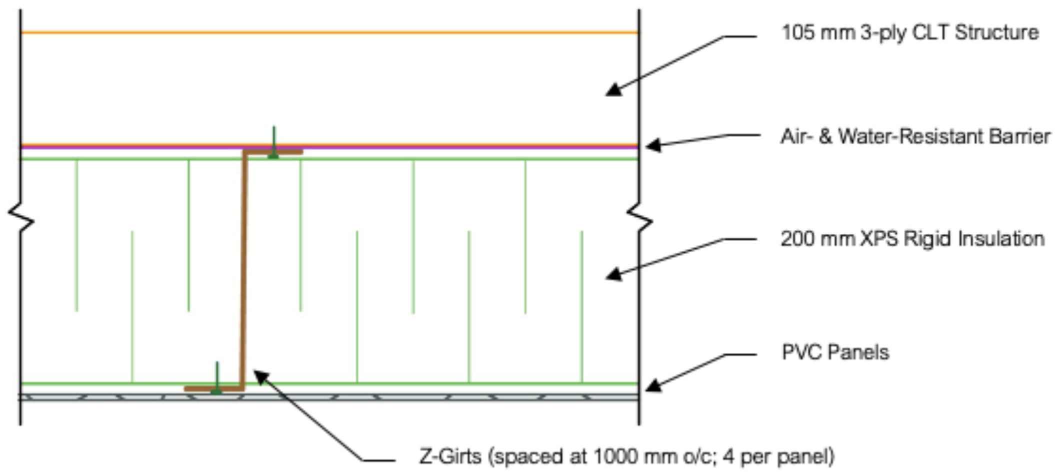


Figure 3.18 – Floor Panel Assembly

The floor-to-floor panel joint, presented in Figure 3.19, includes similar air- and water-resistant barrier wrapping around CLT edges as well as the additional strip of a vapour-impermeable tape along the exterior XPS insulation joint to ensure the XPS can act as a vapour barrier.

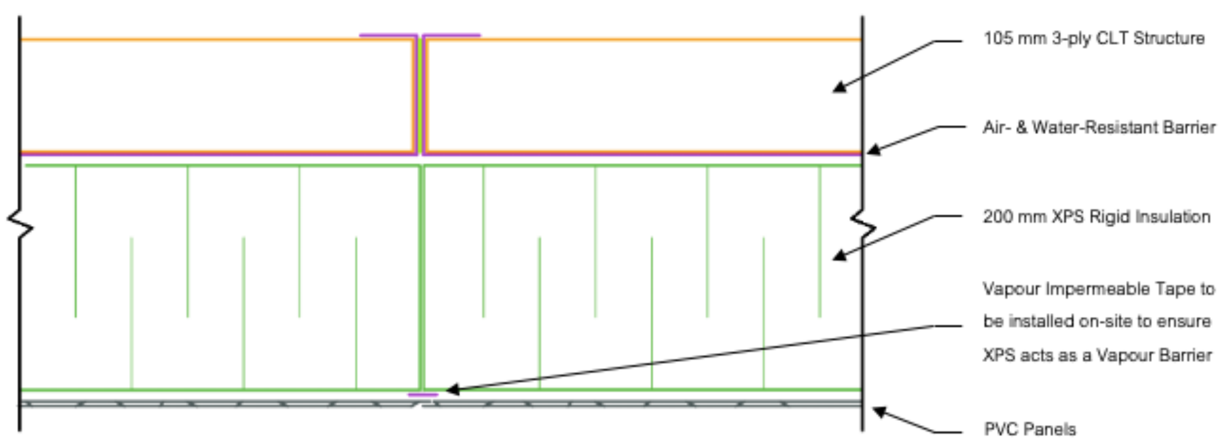


Figure 3.19 – Floor-to-Floor Panel Joint

It should also be noted that a different approach to the joints between similar panels would be to stagger the enclosure joint from the structure joint. This would reduce any potential damage due to leakage; however, this approach was not selected as having a portion of the enclosure offset outward from the edge of the structure would pose a concern with the overall durability of the panel components in transportation and reassembly as this portion of the enclosure would be a weak point on each panelized component,

3.6.5 Enclosure Connections

While all individual panelized components of the prototype include the necessary insulation, air, vapour and water-resistant barriers, and cladding, the joints between connections must also be considered; particularly the joints between different panel types (e.g., wall-to-roof) as the joints between similar panels (e.g., roof-to-roof) have already been factored into the initial panel design. A wall-to-roof joint is presented in Figure 3.20; this represents the joint between the shorter edges of all roof panels and wall panels B1 and B6. A similar joint concept can be modified to suit the slanted edges of the building (along the top of the sloped wall panels: B2-B4) using the various shear (AB-v) connections and would not need to include the uplift (AB-u) connections as these are only required along the lengths of the building.

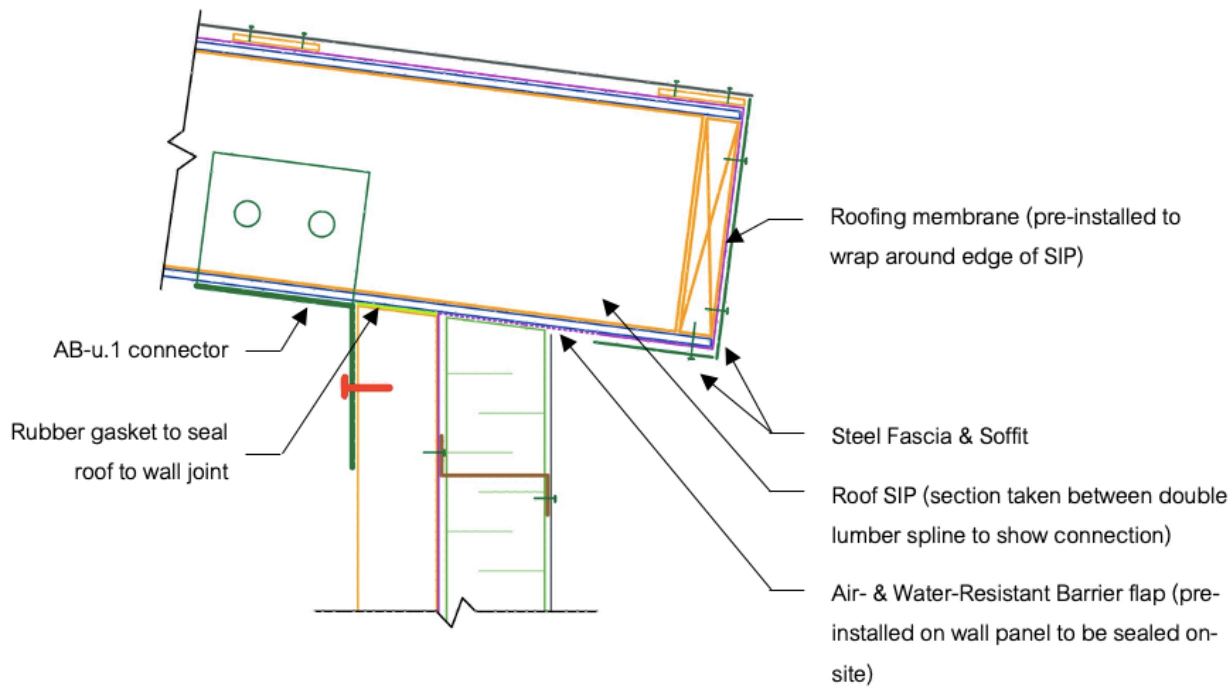


Figure 3.20 – Wall to Roof Panel Enclosure Connection

The wall-to-floor joint is presented in Figure 3.21; this joint represents the connections along the long sides of the building between the floor panels and wall panels B1 and B6. It is understood that the enclosure cannot fully surround the floor panels as the CLT structure must sit on the structural supports below. These glulam beam supports are placed along the two long edges of the structure leaving the area directly beneath the wall panel footprints exposed, or without any enclosure assembly. This exposed area therefore also acts as the access point for the steel bracket connections that connect the base, or floor panels, to the wall panels.

The joint detailing required along the shorter edges of the building is nearly identical but requires additional consideration and detailing for the connection access point below as this will not sit on a glulam beam to provide some level of thermal resistance. These access points along the two sides of the building would therefore require an additional insert to provide adequate thermal resistance that can be locked into place after the connection/wall to floor joints are fully assembled and access to the steel brackets is no longer required. This falls outside the scope of this study and is acknowledged as a challenge in the enclosure design that must be resolved.

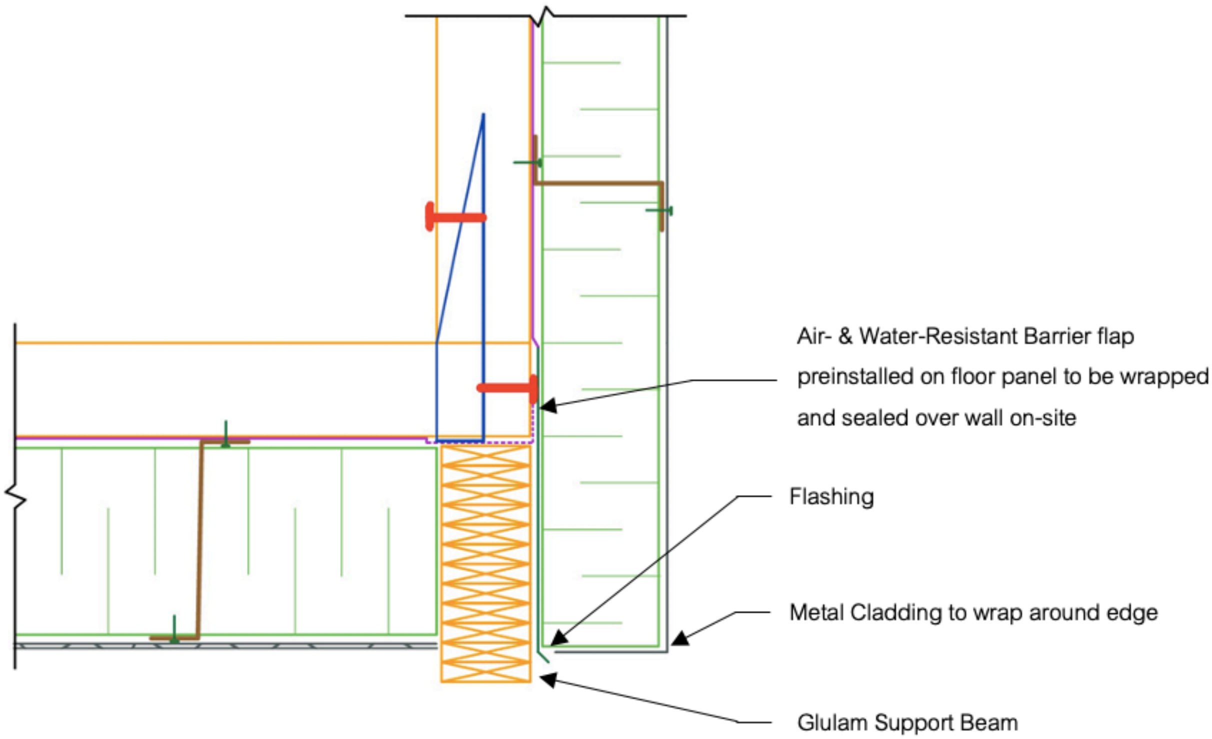


Figure 3.21 – Floor to Wall Panel Enclosure Connection

A panelized modular building will not achieve a continuous enclosure system with its enclosure being prefabricated onto the panels off-site, but it can be designed to achieve some level of continuity with on-site work. The enclosure layers can be gasketed with rubber lining such that when the connections are inserted and bolted into place in the structure, the gasketed edges of the enclosure are compressed to form an air seal. Further, components can be lined and sealed off with a strip of water-resistant barrier along joints between panels to achieve better airtightness however it is acknowledged that the proposed enclosure does not fully resolve the ideals of air-, water-, and vapour continuity and tightness. It is understood that in the case of a modular building with a prefabricated and preinstalled enclosure, many challenges and concerns arise with enclosure continuity and achieving airtightness. The focus of this study falls in the structural system and its durability while enclosure is only explored for the overall proof of the concept. Additional research and design would be necessary to provide a truly adequate enclosure for a panelized modular relocatable building.

Chapter 4 – Experimental Testing Program

4.1 General Overview

The experimental program investigates the feasibility and durability of a novel panelized modular wall-to-floor connection system designed for disassembly and reassembly multiple times over its service life in comparison to traditional systems. The novel wall-to-floor system consists of a 2 m by 2.8 m 3-ply CLT wall panel connected to a 3-ply CLT base panel using two steel bracket connections with removable steel bolts to lock the panel in place. The novel system's behaviour will be compared to a light-frame wood wall-to-floor system with standard Simpson Strong-tie hold-down steel connectors.

Experimental tests were conducted on two different specimens of each system type. One of each system specimen was tested under monotonic loading conditions until failure occurred in the member or connections; the data collected from this testing was then used to determine the cyclical intervals to be used for the CLT and light-frame wood wall-to-floor assemblies. The remaining two specimens, one of each system type, were subjected to repeated loading and reassembly based on calculated design loads that were verified through the Phase 1 testing done on the initial two specimens. Each of the two specimens were assembled, cyclically tested to the design/yield load repeatedly to simulate a service life, taken apart, dropped in various ways to simulate damage endured during transportation and relocation, and finally reassembled and monotonically loaded to failure.

Table 4.1 provides a summary of the completed testing program. The following sections will present the both CLT and light-frame wood wall-to-floor assemblies, the experimental test setups and instrumentation, and the testing procedures.

Table 4.1 – Experimental Test Plan

Panel Type	Phase 1 Testing	Phase 2 Testing
CLT 1	1	
CLT 2		1
Light-Frame 1	1	
Light-Frame 2		1

4.2 CLT Test Specimen

4.2.1 General

The CLT test specimens required for the experimental testing included two (2) 2 m by 2.8 m 3-ply (105 mm) panels and two (2) 2 m by 0.4 m 3-ply (105 mm) bases. These panels were manufactured to size by Element5 and slotted and detailed in the lab.

4.2.2 Preparation

The CLT preparation consists of the necessary slotting and drilling for the steel bracket connection to be installed. This includes two slots into the bottom edge of the wall panels and two T-shaped slots through the bases; both are shown in Figure 4.1.

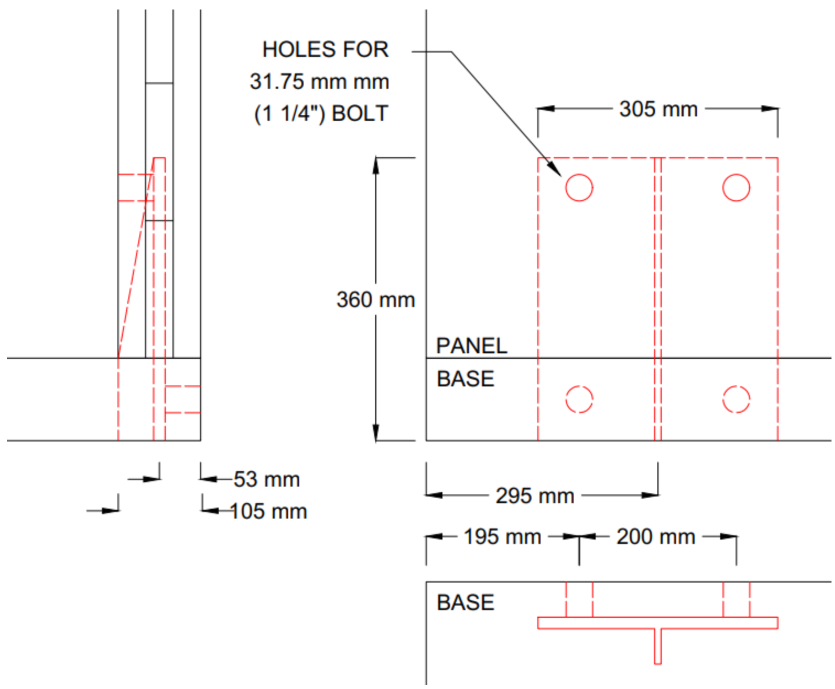


Figure 4.1 – CLT Slot Detailing

4.2.3 Test Assembly

Once the CLT is slotted, the steel bracket connection can be slid through the base and secured with two bolts through the edge of the base (refer to Figure 4.2). Due to an error in the fabrication of the steel brackets; the bolt holes were ultimately sized and threaded for 35 mm (1 3/8") bolts. The base can then be mounted and secured onto the test frame grade beam (refer to Figure 4.8) and the wall panel can be placed onto the base by aligning the edge slots with the bracket sticking up through the base and secured in place with two bolts.

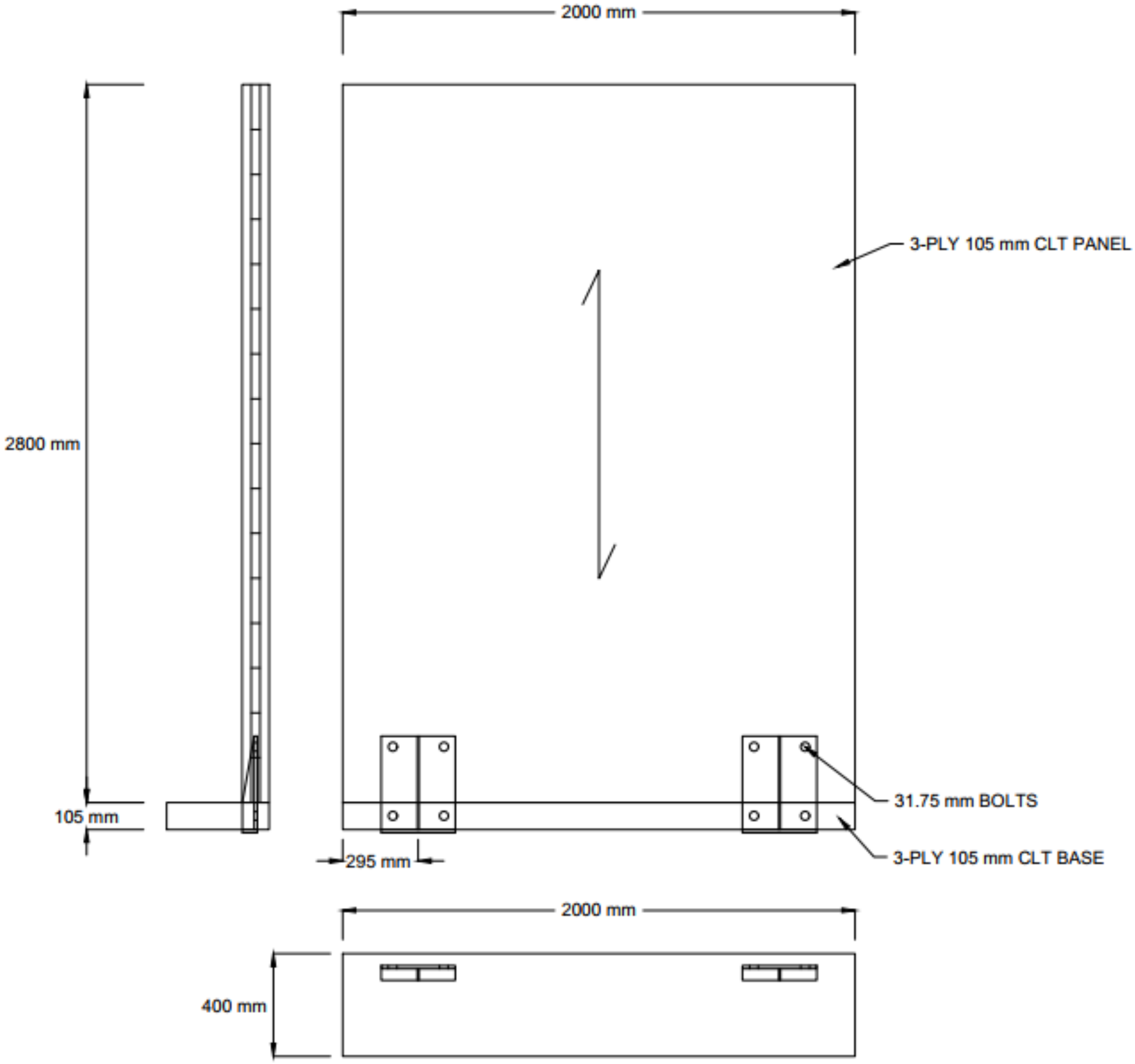


Figure 4.2 – CLT Test Specimen System

4.3 Light-Frame Wood Specimens

4.3.1 General

The light-frame test specimens required for the experimental testing included three (3) 2 m by 2.8 m light-frame stud wall panels and three (3) 2 m by 0.4 m light-frame bases. These specimens were built in the lab with the intention of accurately reflecting standard light-frame construction practices.

4.3.2 Wall

The 2 m by 2.8 m light-frame panel is built using No. 1/No. 2 SPF 38x140 mm (2x6) lumber with studs spaced at 400 mm on centre and doubled end studs (Figure 4.3). The lumber is framed using 76 mm (3”) 10d galvanised steel nails (2 nails per joint) and 102 mm (4”) nails for joints that require securing blocking through the double end studs. Two rows of blocking are installed to stiffen the panel and provide a backing for nailing in the sheathing (shown in Figure 4.3). The lower row of blocking is used to nail sheathing panel edges and must therefore be installed at the same height along the width of the panel. The additional upper row of blocking can be staggered for easier installation as the panel nailing can follow the blocking pattern for stiffening the panel. The panels are sheathed in SPF 12.7 mm (½”) plywood sheets (laid out according to Figure 4.4) using 64 mm (2.5”) 8d galvanised steel nails spaced at 75 mm on centre along the plywood edges and 300 mm along studs and the row of upper blocking. The panel is sheathed only on one side to keep the framing exposed for observation of the behaviour of different elements during testing.

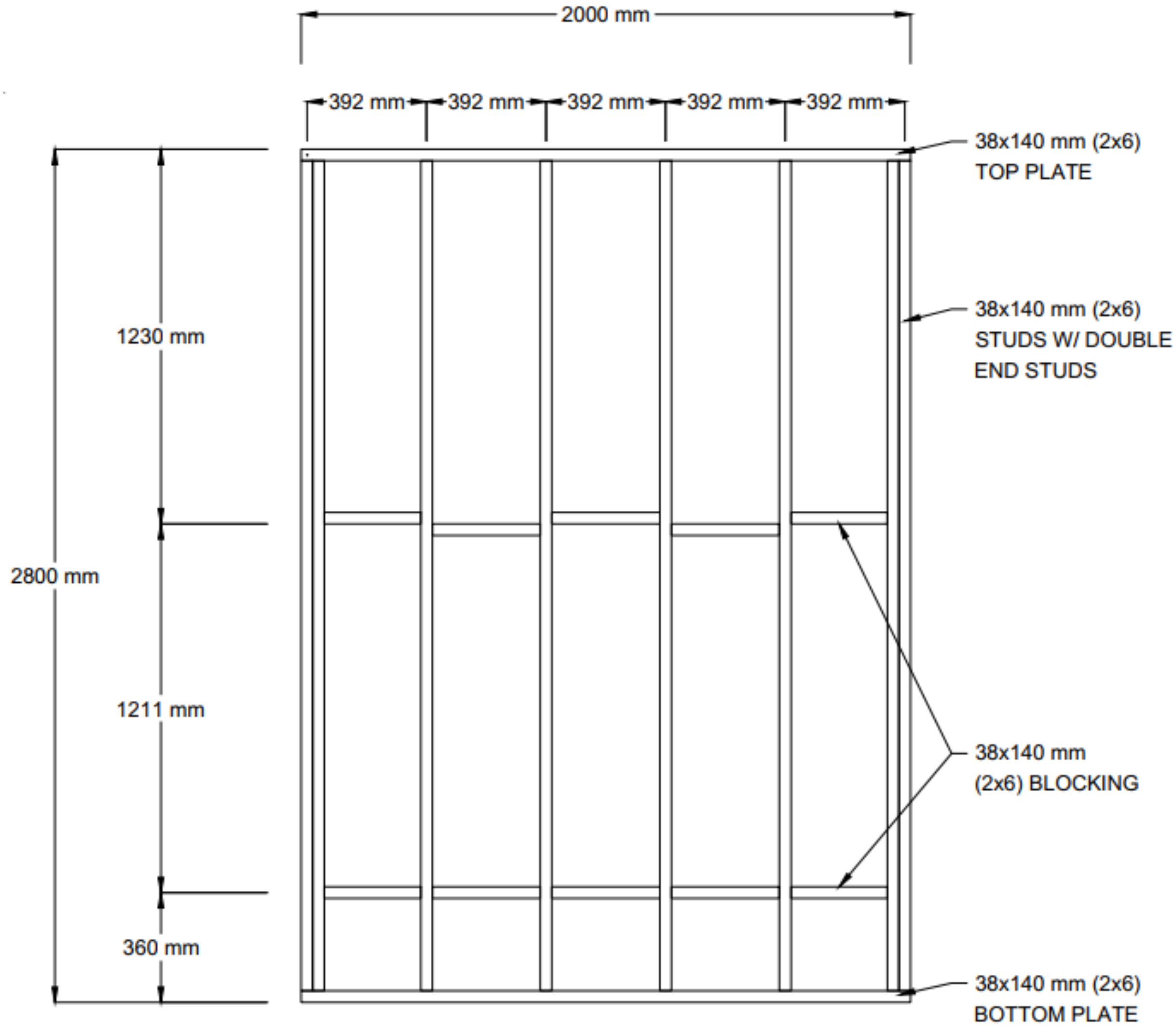


Figure 4.3 - Light-Frame Panel

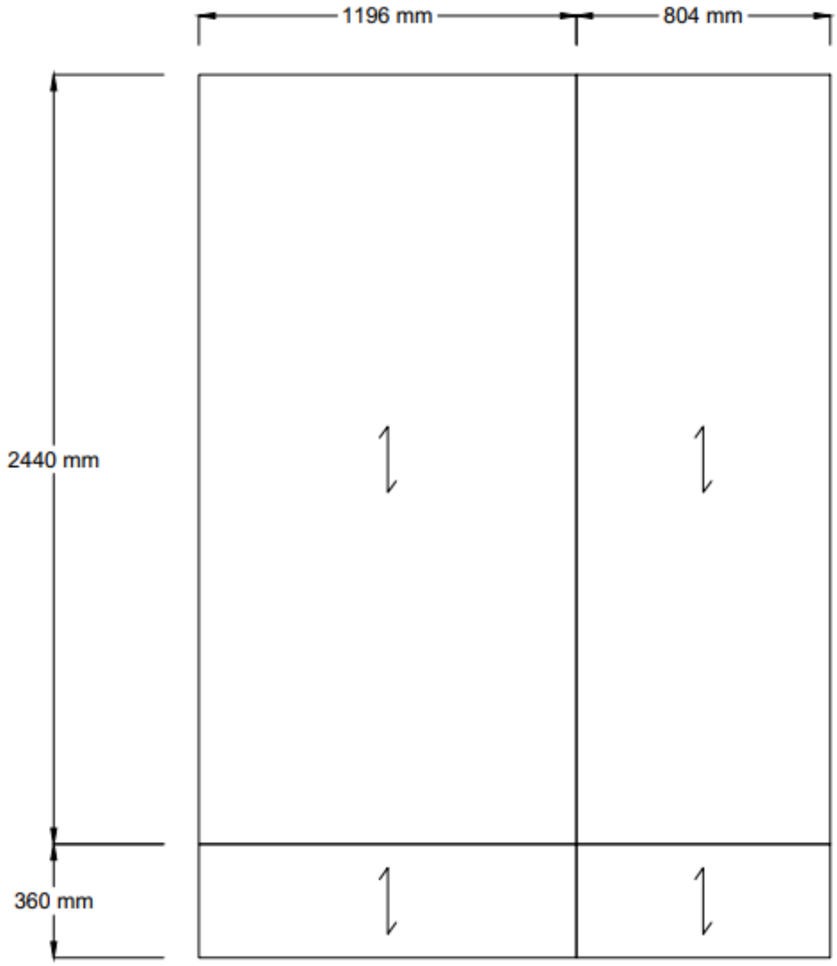


Figure 4.4 – Light-Frame Panel Sheathing

4.3.3 Floor-Base

The 2 m by 0.4 m light-frame base specimens (Figure 4.5) were built using Douglas Fir. 38x286 mm (2x12) lumber; while initially designed for SPF, Douglas Fir was ultimately used due to material availability. The change in material from the initial design specifications is not however expected to affect the overall behaviour of the system as Douglas Fir will only be used in the framing of the bases (the panels were still built using SPF lumber). The bases are framed using 76 mm (3") 10d galvanised steel nails (3 nails per joint) and sheathed on both sides with 19 mm (¾") SPF plywood nailed at 75 mm spacing along the perimeter of the frame. Sheathing (refer to Figure 4.6) was only nailed around the perimeter of the frame as there were no joints between sheathing panel edges and standard practice spacing along studs is 300 mm (bases are only 400 mm wide overall thus eliminating the need for a nail that would be 100 mm – or 10 cm – from another nail). Blocking was ultimately not used as it was not required for nailing sheathing edges and further, the cavity spaces were difficult to access to install blocking while the increased density of the Douglas Fir made nailing difficult. It was therefore established that the increased stiffness of the Douglas Fir lumber would suffice without the installation of blocking.

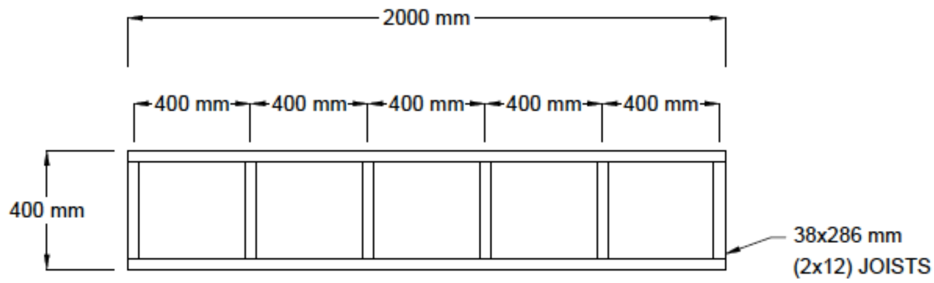


Figure 4.5 – Light-Frame Base

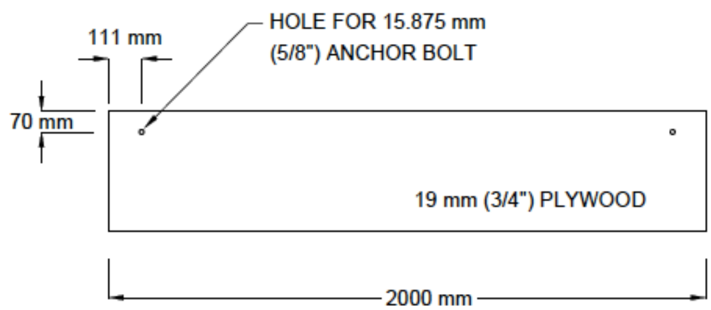


Figure 4.6 – Light-Frame Base Sheathing

4.3.4 Specimen Assembly

To assemble the light-frame test specimen (refer to Figure 4.7), holes are drilled into the wall panel's bottom plate and through the base sheathing for the hold down bolt. Two Simpson Strong tie hold downs are then installed into the light-frame wall panel the secure the panel onto the base. Once mounted into the test frame, the anchor rods will feed through a steel grade beam and be secured down with nuts and washers.

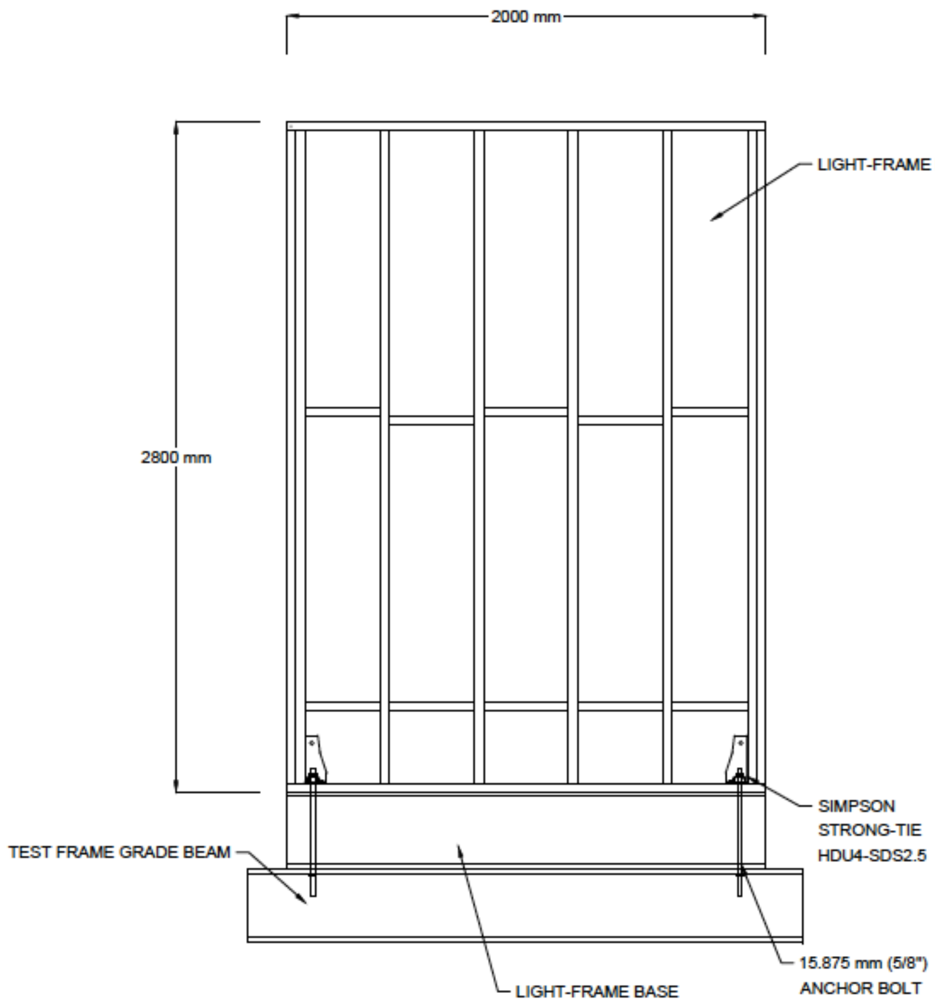


Figure 4.7 – Light-Frame Test Specimen System

4.4 Test Setups and Instrumentation

4.4.1 CLT Test Setup

The test frame setup, shown in Figure 4.9, includes two columns (yellow) on either side of the specimen; one column (right) holds the actuator and load cell and is braced diagonally to the ground on the right side of the setup (out of frame) while the column to the left of the setup has a steel sleeve with the beam on top of the panel to provide out-of-plane support and ensure the panel is properly loaded laterally. The grade beam (yellow) below the panel provides a rigid base to secure the panel and base system down. The CLT base is secured to the grade beam with six anchor rods and enlarged washers on the four outer rods to ensure the base has sufficient area to bear onto when either end is in tension (uplift from the panel rotating). The HSS beam on top of the CLT panel will be loaded by the actuator and transfer the load to the panel through five anchor rods secured through the panel horizontally between two metal plates welded onto the front and back of the HSS beam.

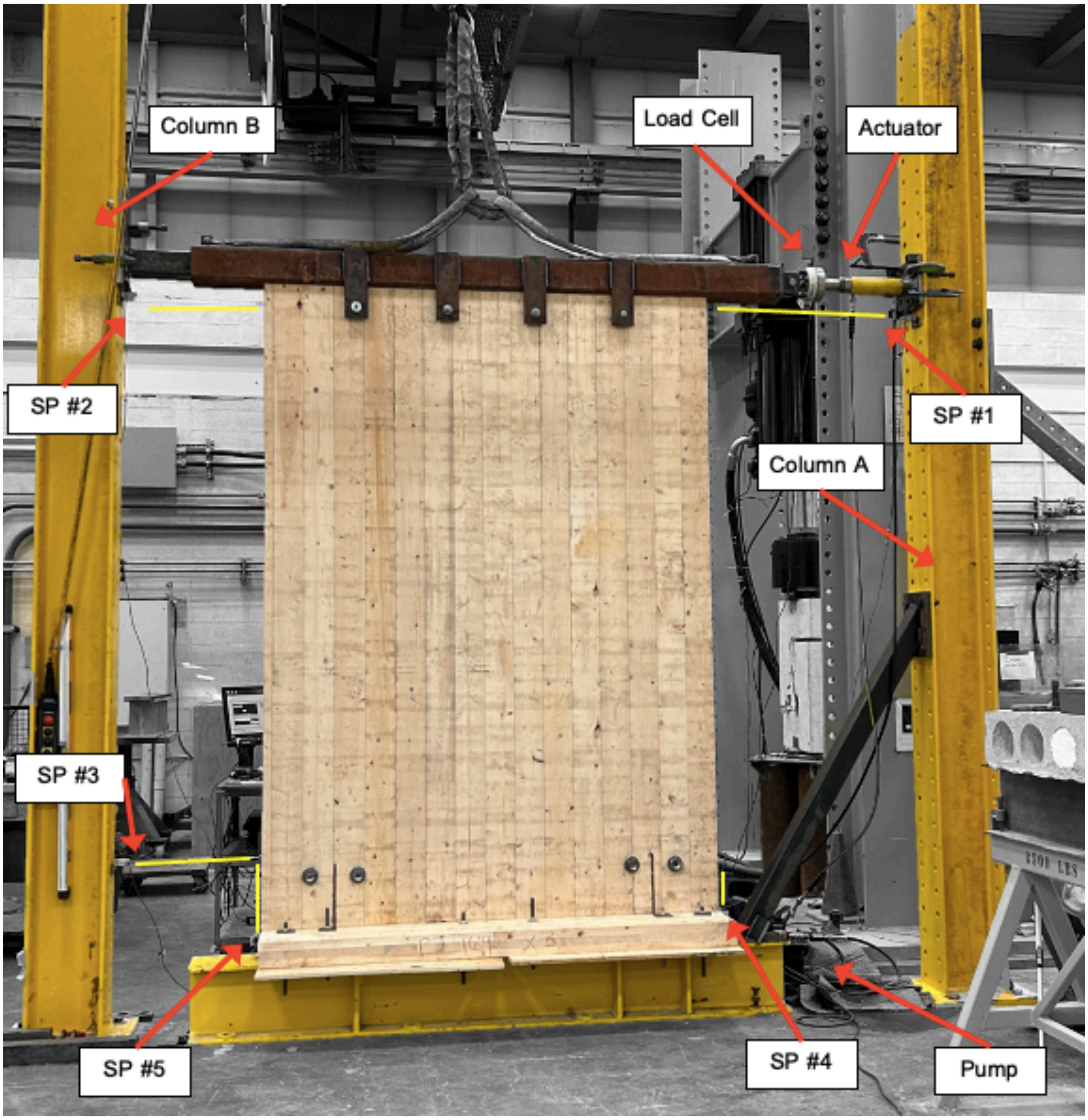


Figure 4.8 – CLT Test Setup

4.4.2 Light-frame Test Setup

The light-frame test frame setup, shown in Figure 4.9, is similar to the CLT test setup. The grade beam (yellow) below the panel provides a rigid base to secure the panel and base system down via the anchor rods through the Simpson-Strongtie hold downs. The HSS beam on top of the light-frame panel will be loaded by the actuator and transfer the load to the panel through five anchor rods drilled in between the panel's studs.

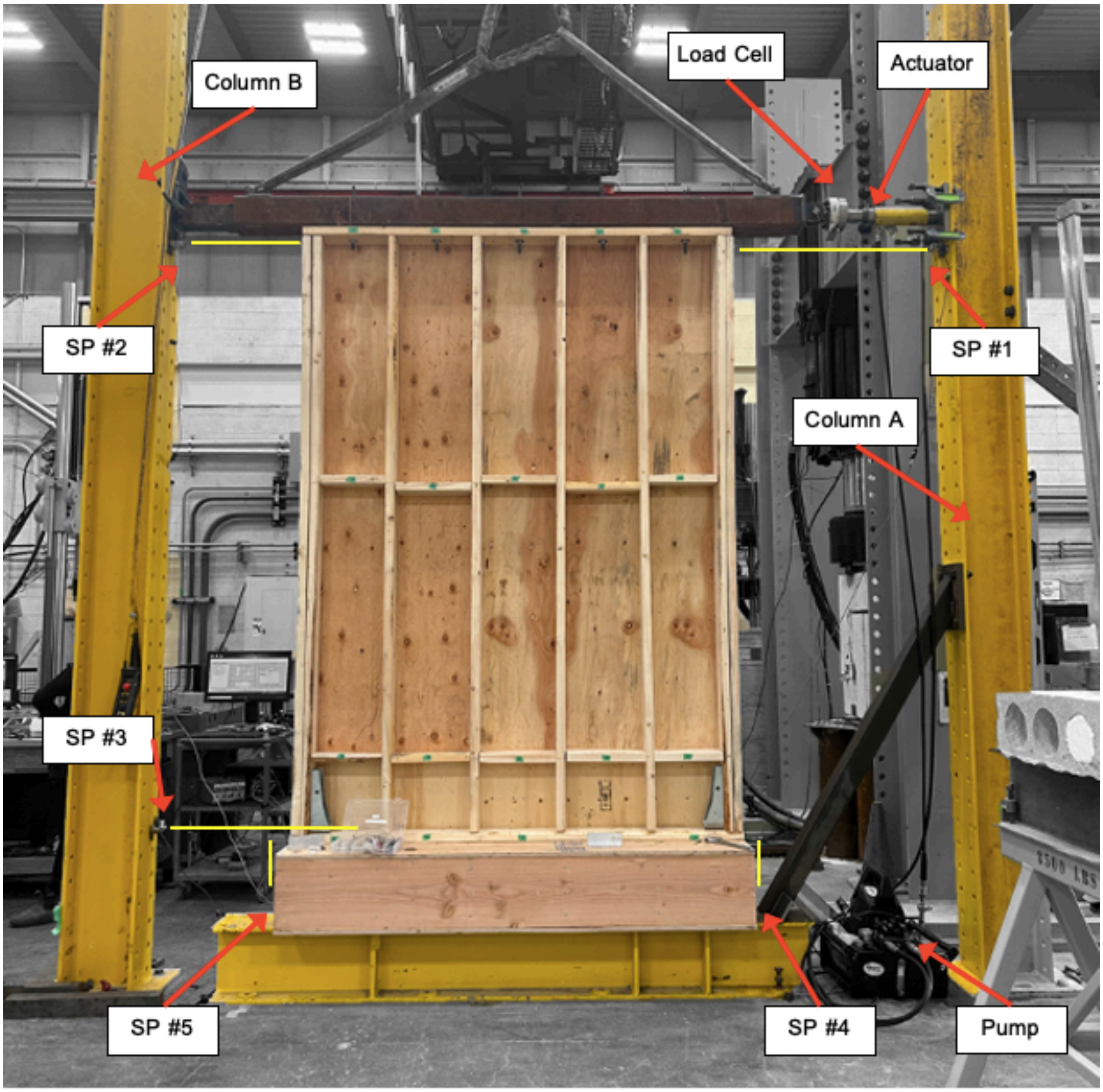


Figure 4.9 – Light-Frame Test Setup

4.4.3 Instrumentation

The CLT and light-frame wall-to-floor assemblies (both for phase 1 and phase 2 testing) are instrumented with a total of one load cell and five string potentiometers (string pots). Figure 4.9 and Figure 4.8 in the previous section identify the locations of the instrumentation on the setup for each panel type. The load cell was placed between the actuator and the HSS beam to collect data from the load being applied along the top of the panel throughout the various load tests. The string pots are placed at specific locations to measure the deflection of the upper right corner of the panel (SP 1), sliding within the test frame (SP 2 and SP 3), and finally any uplift occurring between the panel and base specimens at each of the hold down connections (SP 4 and SP 5).

4.5 Test Procedure

4.5.1 Phase 1 Testing

Phase 1 testing consisted of a monotonic lateral load test that pushed each structural panel type to failure. The load was applied incrementally by manually controlling the electric pump in order to extend the timeline of the full test as the pump lacked pressure and loading rate controls. The specimens (one CLT and one light-frame assembly) were thus loaded in a stepwise manner rather than a controlled rate of loading. Nonetheless, the loading protocol was displacement controlled as the stepwise loading was applied by actuating the system to key displacement values. This process was repeated until the panel reached ultimate failure.

4.5.2 Phase 2 Testing

Phase 2 testing consist of assembling the second wall-to-floor assembly of each system type (CLT and light-frame) and running repeated cyclic load tests (i.e., 3-5 times) up to the design load of each respective panel using the same method to control the displacement of the specimens by loading them in a stepwise manner until the desired specimen displacement was achieved. The following steps consisted of the disassembly of the wall panel from the base, dropping the panel from the height of a truck bed twice (i.e., once on a bottom edge and once on a bottom corner from a height of 1.1-1.2 m based on available lab equipment and safe dropping maneuvers), and finally reassembling the panel to its base to ensure the panel can be reassembled and has not incurred sufficient damage to prevent reassembly and to conduct a final monotonic load test to failure. The latter will establish if and how much the load resistance/capacity of the panel is reduced after the repeated cyclic loads (simulated “service life”) and dropping (simulated transportation/assembly/disassembly processes in which the panels may be damaged). This test phase is summarized in Table 4.2

Table 4.2 – Phase 2 Breakdown

Part of Phase 2	Associated Task(s)
Part 1	Cyclic testing to design load (repeat 3-5 times)
Part 2	Disassemble panel from base
Part 3	Drop from height of truck bed to induce possible damage
Part 4	Reassemble panel on base (if possible)
Part 5	Monotonic test to failure (if possible)

Each cyclic load test follows the breakdown provided in Table 4.3. This consists of incrementally reaching the design load (about 10 kN) in both the positive and negative (forward and backward) directions to simulate a service life of the structure.

Table 4.3 – Cyclic Load Test Breakdown

Step	Minimum # of Cycles	Amplitude (% of Displacement)
1	1	±10
2	2	±20
3	2	±40
4	2	±60
5	2	±80
6	2	±100

Chapter 5 - Experimental Results and Discussion

5.1 General Overview

This chapter presents both the quantitative and qualitative results of the experimental testing on both wall-to-floor assemblies. This includes data collected from the load cell (applied loads), string potentiometers (deflections), and finally the observed damage that occurred throughout each step of testing. The results are then analyzed and discussed.

5.2 CLT Results

5.2.1 Phase 1 Testing

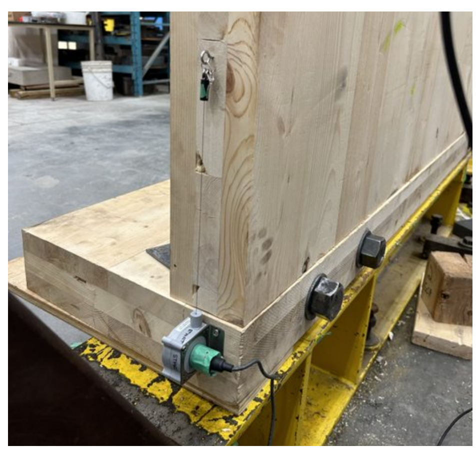
Phase 1 consisted of subjecting the wall-to-floor assembly to monotonic in-plane lateral load until failure was attained in order to establish the global behaviour and to obtain the load-displacement relationship of the assembly. Figure 5.1 presents the panel before testing began; areas of interest included the steel brackets connections acting as the hold downs between the panel and the base and the wood surrounding these connections (both on the panel and base components). It should be noted that the slotting for the bracket was completed using a drill and chisel therefore the slots were not clean cut; however, there were no major imperfections to note before the panel was tested with the exception of additional material being chiseled out of the slot in some locations based on the nature of the material and ensuring that the bracket could be slotted into it. The bolt holes were drilled using a drill thus there were no major pre-existing imperfections to note here.

a) Right side connection slot in panel

b) Left side connection (incl. front bolts through panel)

c) Left side connection (incl. back bolts through base)

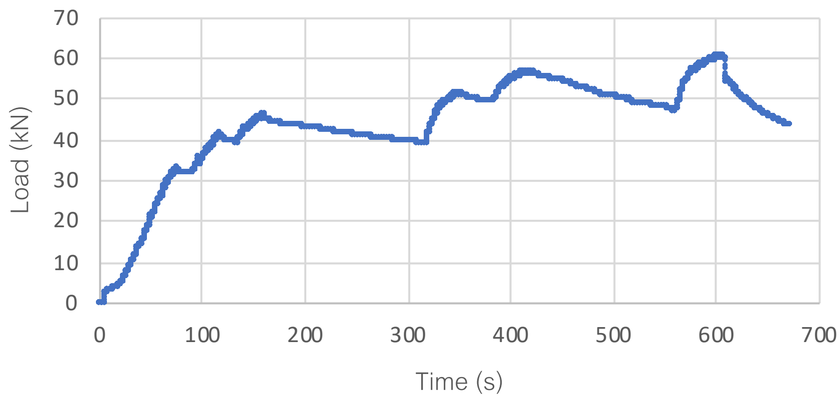
d) Right side connection (incl. front bolts through panel)



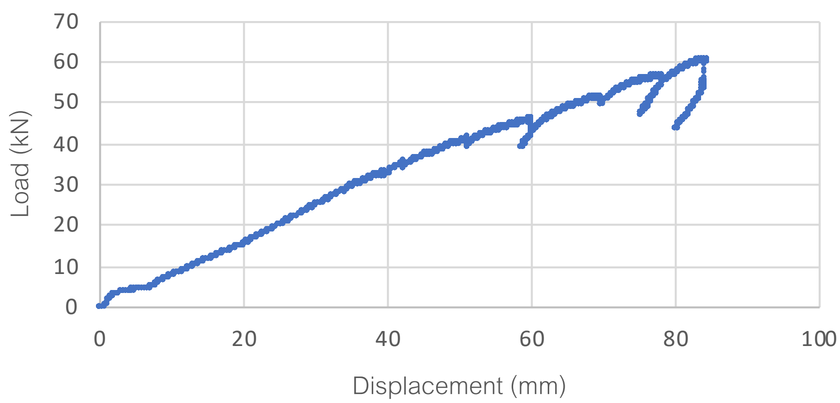
e) Right side connection (incl. back bolts through base)

Figure 5.1 – CLT #1 Before Phase 1 Testing

The monotonic load test results are presented in Figure 5.2. It can be seen from Figure 5.2a that the panel was loaded incrementally as consistently as possible; however, lags in the data points are due to pausing the load to observe any damage to the panel and assess if additional load should be imposed. From Figure 5.2b, it can be seen that CLT #1 attained a maximum load of 60 kN with a corresponding displacement of 84 mm. Furthermore, the panel is observed to behave linearly up to the 40 to 50 kN range where yielding is observed by the lower slope of the load-displacement graph. The initial stiffness prior to and after yielding corresponds to 0.82 kN/mm and 0.71 kN/mm, respectively. Figure 5.3 shows the overall final displaced shape of CLT #1 where it can be seen that there was significant rotation and uplift.



a) Load v. Time



b) Load v. Displacement

Figure 5.2 – CLT #1 Phase 1 Monotonic Test

Figure 5.3 – Final Displaced Shape of CLT #1 After Phase 1

Figure 5.4 shows close-up snap shots of the observed damage at the areas of interests upon completion of phase 1 of testing on the CLT system. In Figure 5.4a, it can be seen that the CLT wall displaced horizontally relative to the base (i.e., sliding). Figures 5.4b and c illustrate the observed uplift from the front and back of the CLT wall-to-floor assembly, respectively. Additionally, it can be seen that the connection securing the CLT floor to grade beam had some uplift as evidenced by the yielding of the custom washers (Figure 5.4b) and uplift between the floor and grade beam (Figure 5.4c). The uplift observed on the right side of the CLT wall-to-floor assembly resulted in crushing of the wood at the bolt locations as shown in Figures 5.4d and e. The left side, where the wall-to-floor specimen was loaded in compression, no significant damage to the wood surrounding bolts was observed as the forces were primarily transferred from the wall bearing against the CLT base.

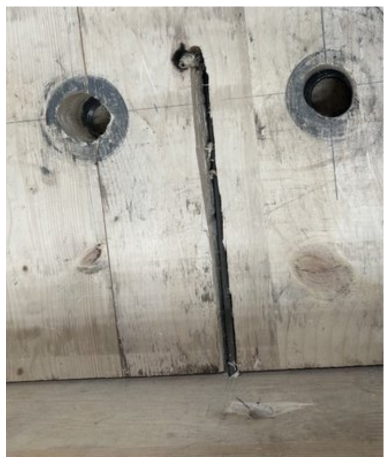
It was noted that the hold down system for the base to the grade beam was affected by the Phase 1 testing; this can be seen in the deformation of the washers on the base in Figure 5.4b and in the indentation of the base at the bottom of Figure 5.4e as well as in the overall uplift of the base from the grade beam in Figure 5.4g. Although the floor-to-grade beam connection system is not within the scope of this research program, it should remain rigid.

a) Left side bottom corner (near connection)

b) Right side connection (front view)

c) Right side connection (rear view)

d) Right-most panel connection



e) Right side connection (bracket removed)

f) Left side connection (bracket removed)

g) Rear view of overall panel uplift

Figure 5.4 – Localized Damage of CLT #1 After Phase 1

5.2.2 Phase 2 Testing

Phase 2 testing consisted of three rounds of cyclic load tests on CLT #2 followed by a drop test, and a monotonic load test to panel failure. The cyclic tests were completed to simulate a service life of the system with each test cycle reaching the design load of the panel. Existing areas of concern were noted and provided in Figure 5.6.

Prior to conducting Phase 2 testing on CLT #2, the uplift between the CLT floor and the grade beam as well as the rotation at the actuator had to be resolved. Figure 5.5 illustrates the revised test setup where larger washers were used while the actuator assembly was therefore modified to include a second pivot point for Phase 2 testing to ensure the panel is being loaded rather than any components within the test frame itself.

a) Modified CLT setup (note additional pivot point between actuator and column, HSS shifted to account for pivot)

b) Left side connection (incl. front bolts through panel; note upgraded washers on base)

Figure 5.5 – CLT #2 Before Phase 2 Testing

a) Rear view of system

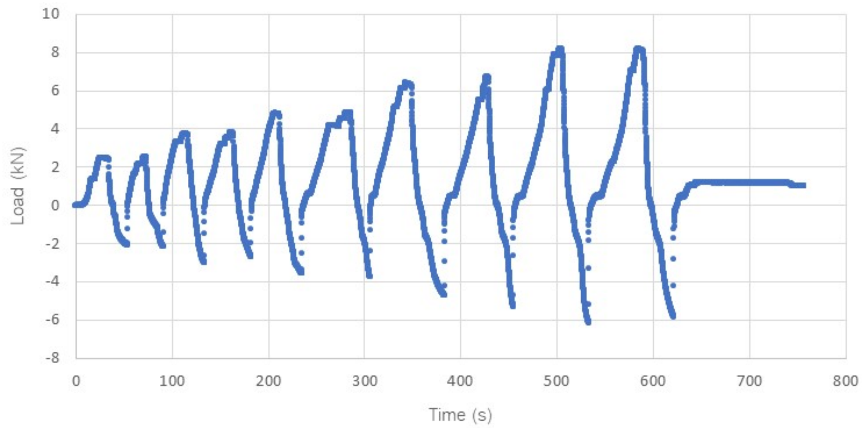
b) Left side connection (incl. back bolts through base)

c) Right side connection (incl. front bolts through panel)

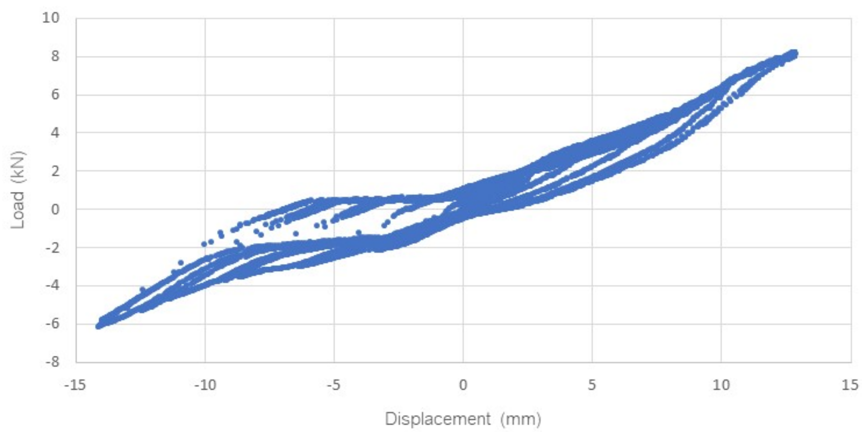
d) Right side connection (incl. back bolts through base)

Figure 5.6 – CLT #2 Before Phase 2 Testing

The first cyclic test was conducted, and the data is presented in Figure 5.7. During testing it was noted up to 60% of the design load, slight cracking noises were heard; however, no visible cracking or crushing was observed. From 60% of the design load and onwards, each bottom corner experienced some visible uplifting when the panel was loaded and therefore uplifting on each side (i.e., cycling). Upon unloading, the CLT wall-to-floor assembly returned to its original position with no residual uplift and with no visible damage. It should also be noted that the various levels of loads reached were slightly less than the target loads (e.g., 100% of design load is 10 kN) as the actuator required manual control of the loading and the data acquisition system had a delay in its readings. As a result of the lag, loading would be paused and the direction of pressure in the pump flipped so as to not overload the panel past the desired increment of the design load.



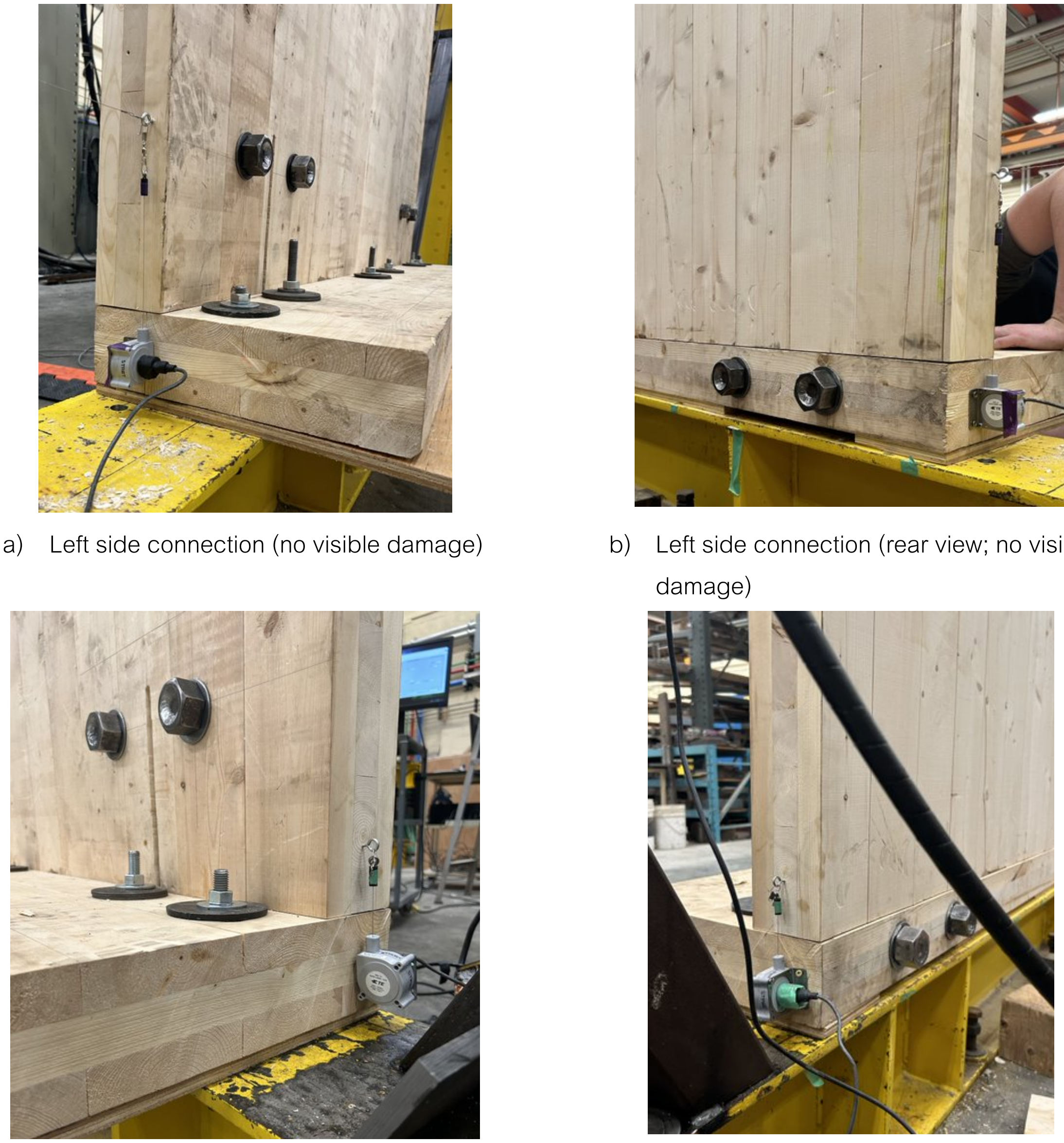
a) Load v. Time



b) Load v. Displacement

Figure 5.7 – CLT #2 Phase 2 Cyclic Test 1

After completing the first cyclic test, areas of interest were documented and provided in Figure 5.8. It can be seen that the system has not experienced any visible damage. The panel remains in place on the base and all connections (i.e., the steel bracket between the panel and base and the anchor connecting the base to the grade beam) remained intact with no visible wood damage (e.g., crushing) surrounding them.



a) Left side connection (no visible damage)

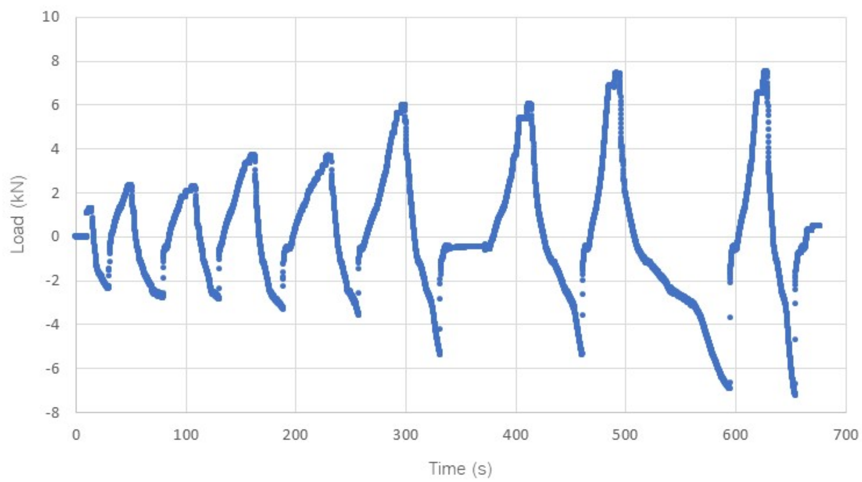
b) Left side connection (rear view; no visible damage)

c) Right side connection (no visible damage)

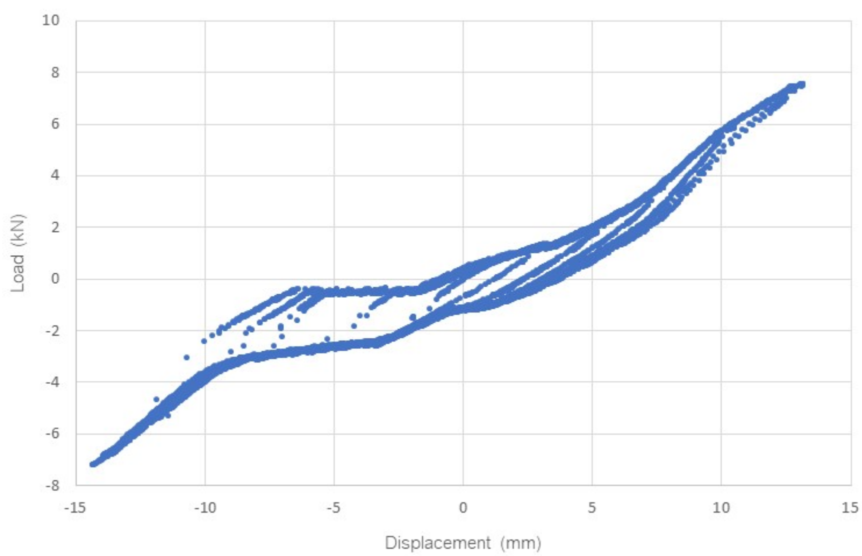
d) Right side connection (rear view; no visible damage)

Figure 5.8 – CLT #2 After Cyclic Test 1

The second cyclic load test was completed, and the results are presented in Figure 5.9. Like the previous cyclic test, some cracking noises were observed once the panel reached the 60% design load cycles onward. Creaking noises were also noted from 40% of design load and onward; however, these were likely due to the system's anchors and threaded loading rods settling against the side of the holes in the panel and base as they were noted just after the loading direction was switching.



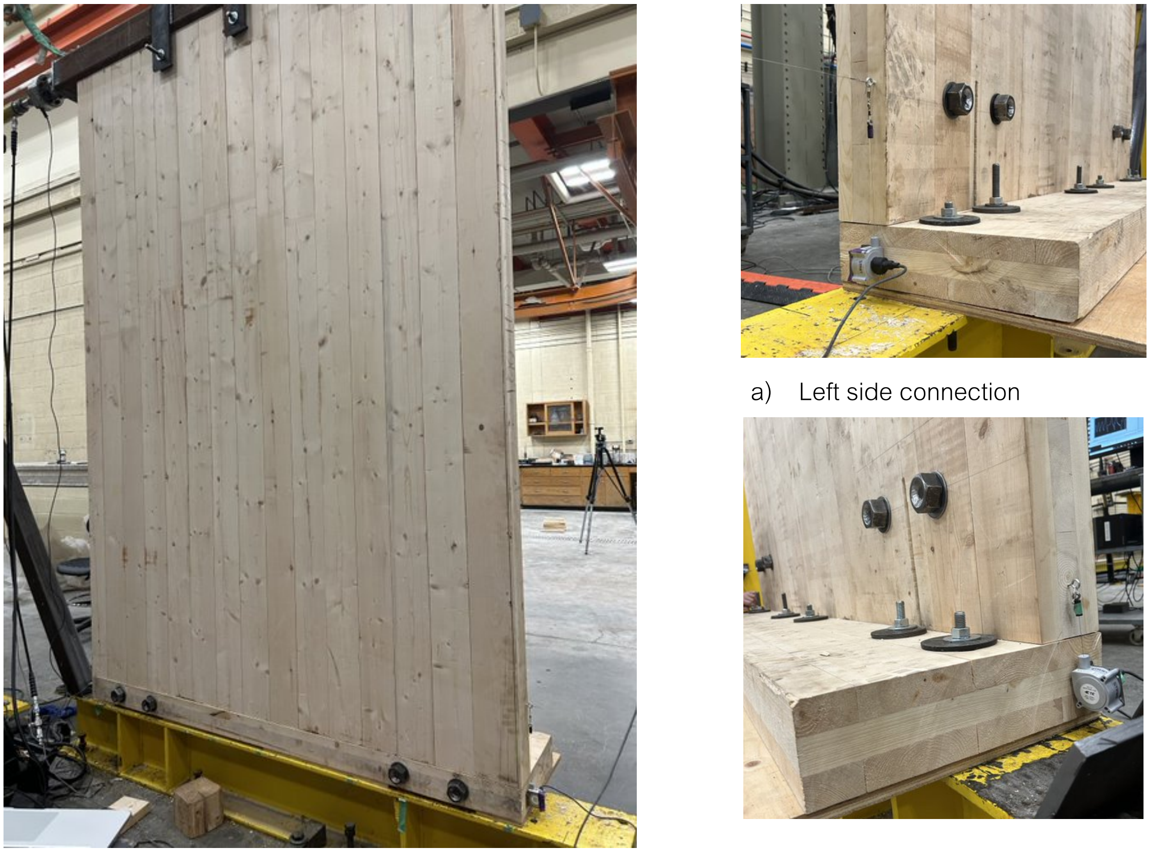
a) Load v. Time



b) Load v. Displacement

Figure 5.9 – CLT #2 Phase 2 Cyclic Test 2

Like the first cyclic test, the second cyclic test did not appear to cause any damage to the panel, base, or wood surrounding the steel bracket connections. The panel remained in the same position on the base with no crushing or other damage visible around the connections/bolts as shown in Figure 5.10).



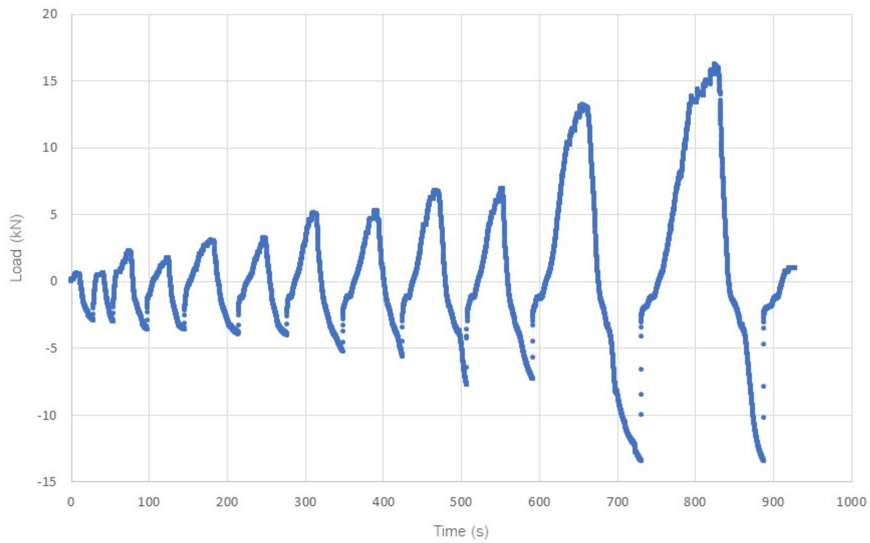
a) Left side connection

b) Back of panel (incl. connections)

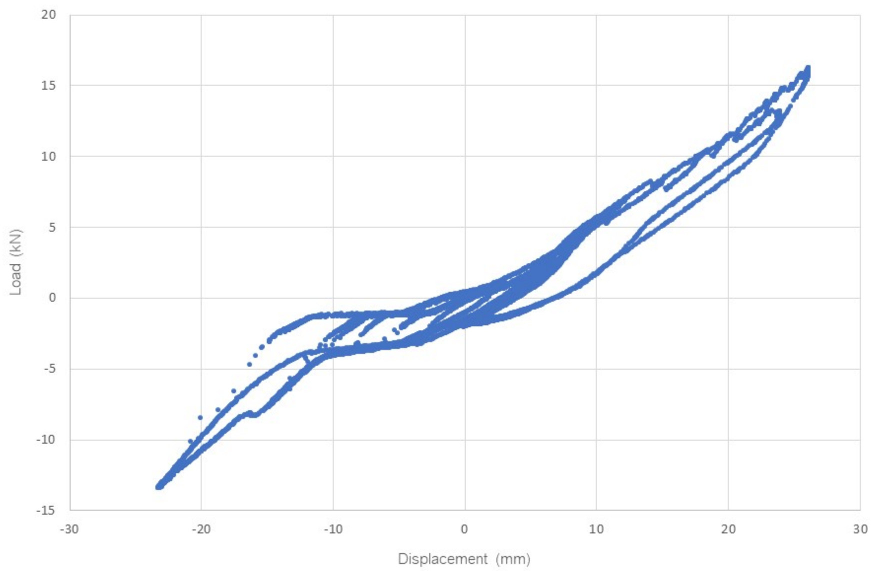
c) Right side connection

Figure 5.10 – CLT #2 After Cyclic Test 2

Due to the lack of damage in the first two cyclic tests on the CLT system, a third cyclic test reaching approximately twice the design load (i.e., 20 kN) was completed. Figure 5.11 presents the final load versus time and displacement results. Throughout the cyclic testing of the CLT specimen, the panel was loaded incrementally based on the reading from string pot 1 as all string pots could be zeroed before testing while the load cell could not be due to software limitations. Thus, during the test, the displayed applied load on the load cell was not the actual load and needed to be zeroed during the analysis. Therefore, the amount of load imposed on the panel was dependent on the string pot 1 data collected in Phase 1 of testing corresponding to the desired load increments. As a result of this and the adjustments made to the test frame setup, the displacement and corresponding loads resulted in lower loads imposed on the panel at the established displacements. Nonetheless, the final cyclic test pushed the panel past it's design load with little damage incurred. During the test, some cracking noises were observed throughout, starting at the 40%-60% design load point in the test, with the most creaking and cracking noises occurring on the final increment (200% design load) which in reality was approximately 16.3 kN (163 % design load).



a) Load v. Time



b) Load v. Displacement

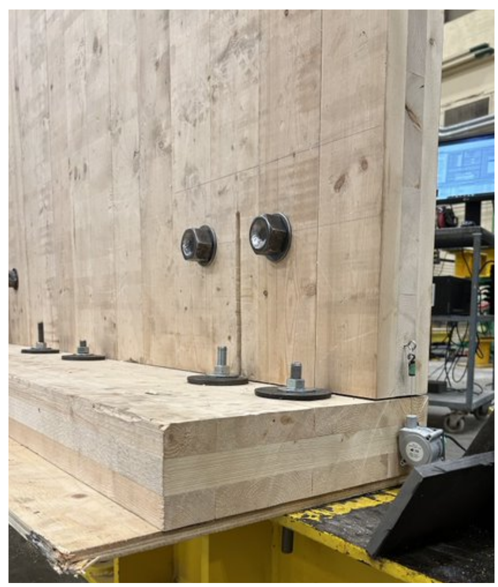
Figure 5.11 – CLT #2 Phase 2 Cyclic Test 3

Upon completing the third cyclic test, the same areas of interest were inspected for any damage, these locations are presented in Figure 5.12. The panel, base, and connection incurred no visible damage from the third cyclic test. After inspection, the front bolts on the panel were unscrewed (with no difficulty) and the panel was removed from the base for drop testing.

a) Back of panel (no visible damage)

b) Left side connection (no visible damage around steel bracket/bolts), upgraded washers were sufficient for test)

c) Left side connection (no visible damage)



d) Right side connection (no visible damage)

Figure 5.12 – CLT #2 After Cyclic Test 3

The drop testing for the CLT panel consisted of two drop tests from a height of approximately 1.1 m; once on the bottom edge of the panel and once on a bottom corner to establish if damage incurred by dropping would affect the ability to reassemble the panel to the base via the steel bracket connection. Figure 5.13 presents photos from both drop tests including the panel's initial position and dropping path.

a) Drop Test #1 Starting Position

b) Drop Test #1 Mid-Drop

c) Drop Test #2 Starting Position

d) Drop Test #2 Mid-Drop

Figure 5.13 – CLT Drop Tests

The damage caused by the drop testing on the CLT panel only included crushing along the edges and corners that were impacted by the drop. Under the weight of the panel itself, the wood crushed upon hitting the pavement (refer to Figure 5.14) however, the damage did not extend past the immediate crushed area. The panel as a whole remained intact with no damage to the overall shape and structure of the panel (i.e., no delaminating of the CLT layers or deformation/warping of the panel or its laminations/individual lumber components).

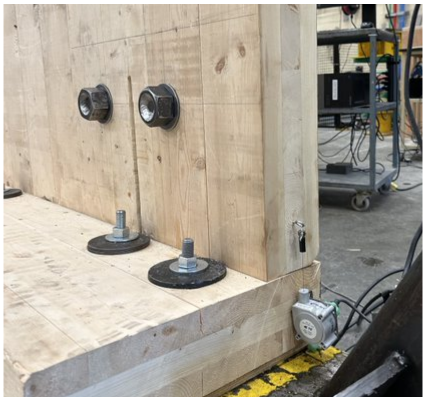
a) Bottom left corner of panel

b) Bottom left corner of panel

c) Bottom right corner of panel

Figure 5.14 – CLT #2 After Drop Test

After the drop testing, the panel was reassembled into the test frame for a final monotonic test. It was noted that there was no difficulty in the reassembly process; no damage incurred through drop testing affected the slotting or bolt holes. The panel was therefore slotted back onto the steel brackets and bolted; the drop test damage; however, was visible in the newly reassembled system (refer to Figure 5.15). The HSS was then reconnected to the actuator and load cell for further testing. Going into the final monotonic test, the panel had no evident damage from the cyclic load testing and only minimal crushing from the drop testing.



a) Right side connection

b) Back of panel (no visible damage except crushing along bottom edge/corners)

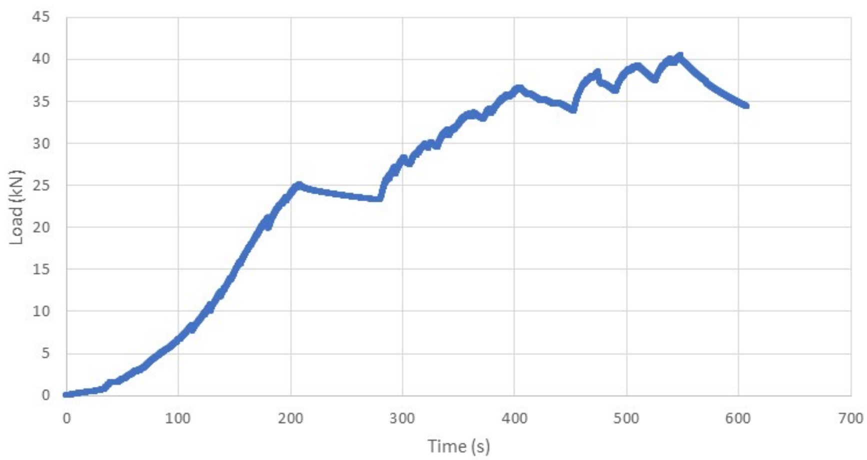
c) Right side connection

d) Left side connection

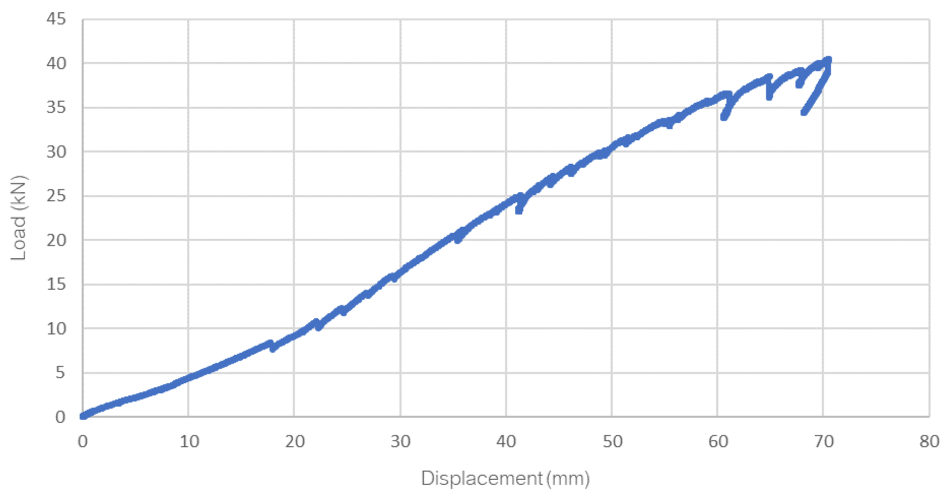
e) Left side connection

Figure 5.15 – CLT #2 Before Monotonic Test

The final Phase 2 test consisted of the CLT panel being monotonically loaded to failure. Like the CLT Phase 1 testing, the panel was manually loaded with pauses in the loading to assess where any damages or points of failure may be occurring on the panel (lags in Figure 5.16 account for these pauses). The results of this test can be found in Figure 5.16; ultimately the panel was loaded to 40.5 kN before failure.



a) Load v. Time



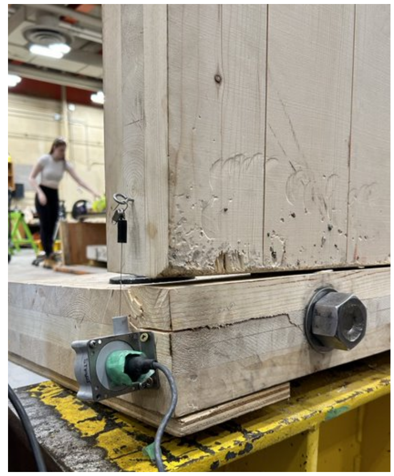
b) Load v. Displacement

Figure 5.16 – CLT #2 Phase 2 Monotonic Test

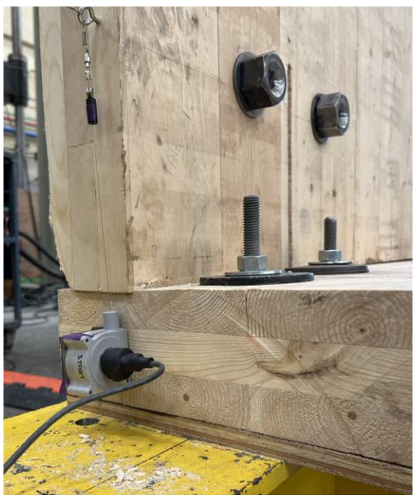
Failure of the system ultimately occurred in the base at the right-most bolt (right side with respect to the front view of the panel) due to uplift of the panel, this failure is shown in Figure 5.17b and c along with additional areas of interest provided in the remaining photos in Figure 5.17. While the bolt hole through the base was the source of failure, the remaining bolt holes in the base and panel remaining widely undamaged with only slight crushing of the bolts into the CLT occurring (visible as threading pattern from bolts was transferred to wood in places. It should also be noted that the damage to the panel caused by the drop testing appears to have had no effect on the mode of failure of the system when subjected to in-plane lateral loads. This damage is visible in Figure 5.17c, d, and e; the damage did not worsen or expand through the monotonic testing indicating that additional drop tests on other edges or corners would most likely have no impact the global failure mode of the system. Figure 5.18 presents the areas of interest on the panel with connections removed.

a) Rotated panel

b) Right side connection uplift (note base crack)



c) Right side connection uplift (note base crack that extends from bolt)

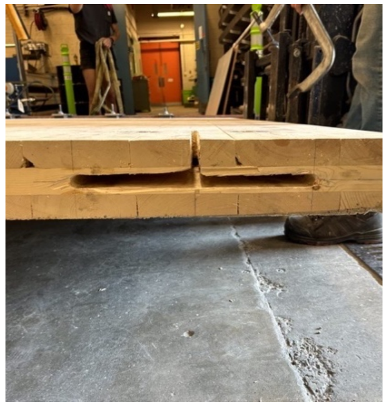


d) Left side connection (note slight crushing on base under panel)

e) Left side connection

Figure 5.17 – CLT #2 After Phase 2 Monotonic Test

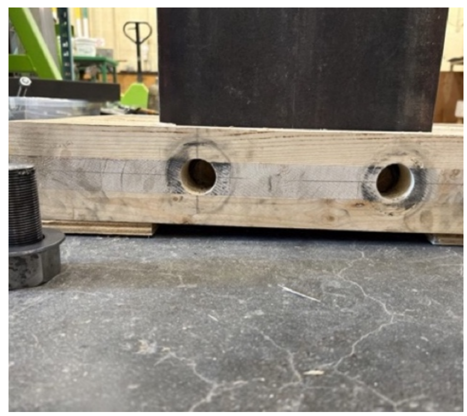
a) Left side connection slot (no visible damage)



b) Right side connection slot (no visible damage)

c) Right side connection with bolts removed

d) Base bolt holes corresponding with left side connection



e) Base bolt holes corresponding with right side connection

f) Hold down bolts (upgraded washers to mitigated uplift between base and grade beam)

g) Right side connection slot (no visible damage)

Figure 5.18 – CLT #2 Disassembled After Phase 2 Monotonic Test

5.3 Light-Frame Results

Each light-frame panel that was tested was labelled with a numbering system according to Figure 5.19 in order to track any damages that were present before testing or that were incurred during testing. Photos of damage were taken throughout testing to track this establish were any damage due to testing was observed and during what point in the testing phase it was a result of.

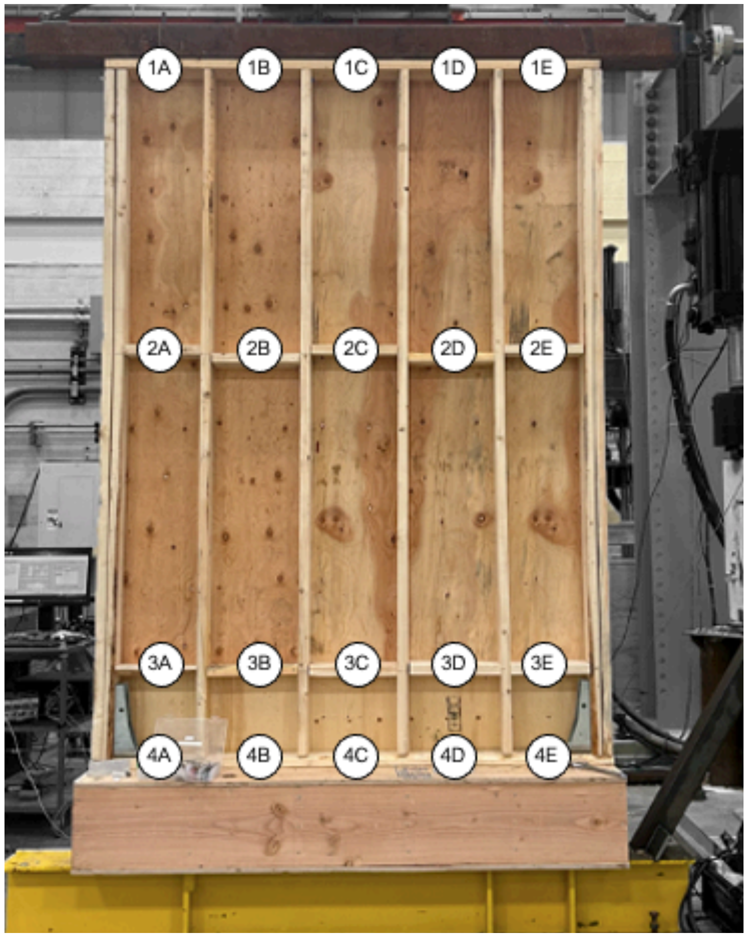


Figure 5.19 – Light-Frame Labelling System

5.3.1 Phase 1 Testing

Phase 1 testing consisted of a monotonic load test to panel failure to obtain an ultimate load for the system. Figure 5.20 presents photos of the panel before testing began; areas of interest included the hold downs and existing damages/defects on the panel.

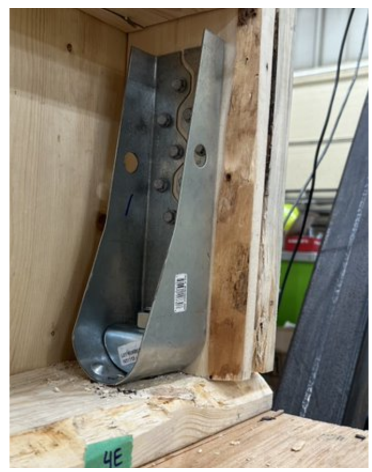
a) Back of panel (note the sheathing layout)

b) Gapping along the double stud due to warped lumber

c) General imperfections: gaps in blocking, protruding nails

d) Left side hold down (located near point 4A)

e) Right side hold down (near 4E); note lumber defects

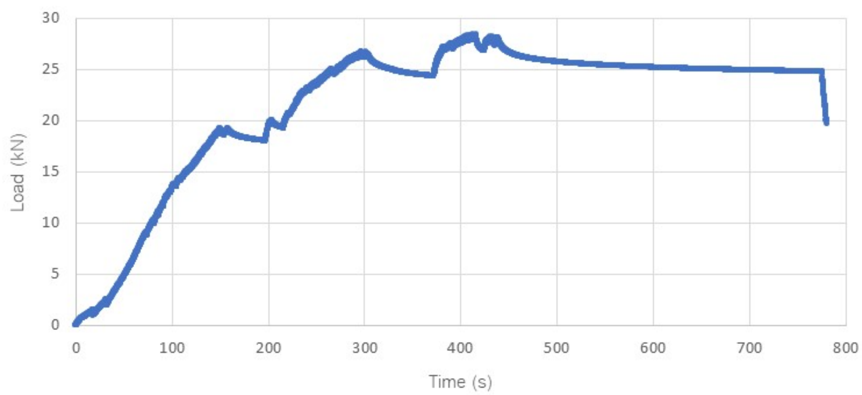


f) Right side hold down; gapping due to chamfered edges

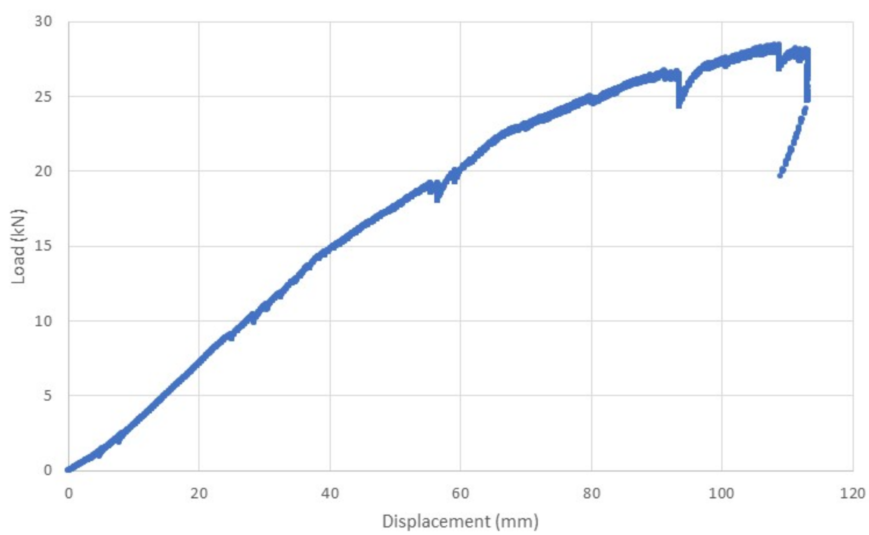
Figure 5.20 – LF #1 Before Phase 1 Testing

As shown in Figure 5.20 the panel had some existing defects/areas of concern due to the number of components that make up a stud wall. These include joints along the sheathing as the standard size of a sheathing panel was not large enough the span the full area of the panel, chamfered corners along the lumber (refer to Figure 5.20e and f) reducing the overlapping surface area of some components and gapping in between the double end studs due to warping in the lumber. Other areas of interest include the hold downs as these connections alone will the uplift loads between the wall panel and base.

The monotonic load test results can be found in Figure 5.21; the panel was loaded incrementally as consistently as possible; however, lags in the data points are due to pausing the load to observe any damage to the panel and assess if additional load should be imposed. Ultimately, the panel reach a maximum load of 28.5 kN and would not take much more load after that; displacement of string pot 1 continued to increase while the applied load began to plateau indicating that the panel was simply deforming at this point and no longer providing additional lateral resistance. When the panel reached maximum loading the displacement of string pot 1 reached approximately 110 mm. At this point the base also experienced some sliding as indicated by the data collected from string pot 3; factoring in this movement by subtracting string pot 3's reading from string pot 1 resulted in an overall displacement of around 110 mm at the panel's upper right corner due to the panel deforming.



a) Load v. Time

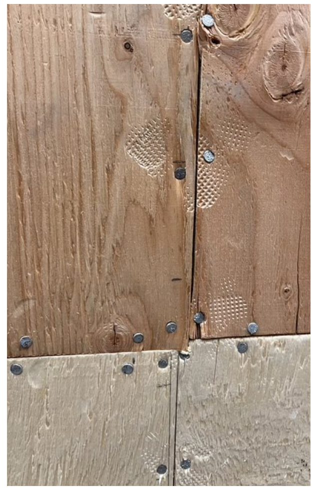


b) Load v. Displacement

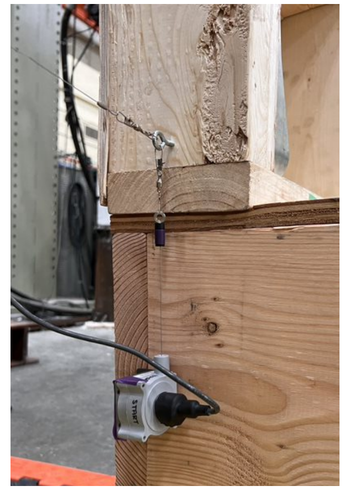
Figure 5.21 – LF #1 Phase 1 Monotonic Test

After conducting the monotonic test for Phase 1; any damage that resulted from the testing was noted and can be found in Figure 5.22. The overall deformation of the panel reached a point that the panel was visibly out of square (Figure 5.22a). Some nails along the sheathing panel joints appear to have pulled inward toward the joints or into the panel as they appear to be pushed into the sheathing further than they could have been hammered (Figure 5.22b). The hold downs and their surrounding components appear to have experienced the majority of the damage, the hold down on the right side of the panel in particular (Figure 5.22d, e, and f). This hold-down experienced significant uplift (over 20 mm based on string pot 4 data) resulting in significant cracking of the bottom plate; complete results collected from the string pot data for all tests can be found in Appendix B. The left side hold down experienced downward force to counteract the uplift on the opposite side resulting in some crushing of the base sheathing from the panel pushing into it (refer to Figure 5.22c). The overall uplift of the panel also resulted in some nail withdrawal at the bottom of the studs on the right side of the panel (refer to Figure 5.22h).

a) Overall deformed panel (before retracting actuator to 0 mm disp.)



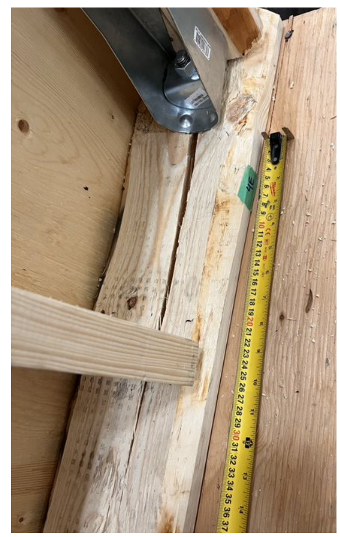
b) Nails along sheathing panel joints pulled in



c) Left side hold down (near 4A) panel crushing base

d) Right side hold down uplift and cracking (4E)

e) Right side hold down uplift (back)



f) Crack in bottom plate

g) Right side hold down (near 4E) uplift (back)

h) Stud nail withdrawal from bottom plate

Figure 5.22 – LF #1 After Phase 1 Testing

5.3.2 Phase 2 Testing

Phase 2 testing consisted of three rounds of cyclic load tests followed by a drop test, and a monotonic load test to panel failure. The cyclic tests were completed to simulate a service life of the system with each test cycle reaching the design load of the panel. Like the previous specimen in Phase 1 of testing, existing damages and areas of concern were noted and provided in Figure 5.23. These included gapping within the blocking and between double studs, protruding nails, and existing cracks in the lumber.

a) Back of panel

b) General imperfections

c) Protruding nails

d) Left side hold down (note crack in bottom plate)

e) Left side hold down (near 4A)

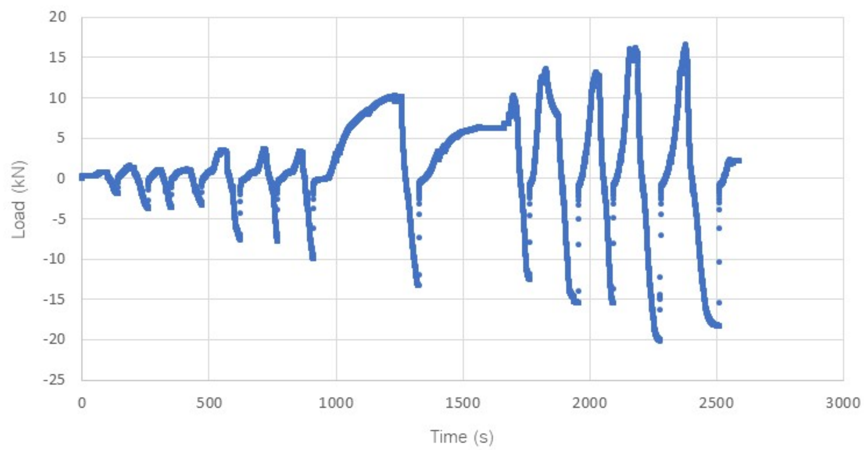
f) Right side hold down (no visible cracks in bottom plate)

g) Right side gold down (slight gap between double end studs)

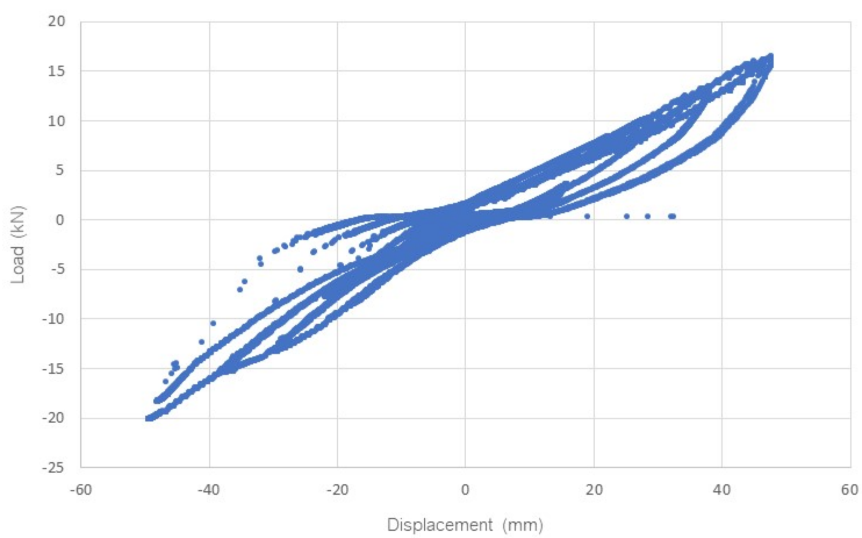
h) Right side hold down (back; no visible damage on bottom plate other than mark on top right)

Figure 5.23 – LF #2 Before Phase 2 Testing

The first cyclic test was conducted, and the data is presented in Figure 5.24. It was noted that cracking noises began once the panel was loaded to 40% of the design load (around 24 mm of string pot 1 displacement). The wall and base system as a whole also began to slide along the grade beam at this point due to the load and size tolerance of the holes through which the hold down anchor rods were fastened. At this point in the test the nuts on the anchor rods were checked and found to be relatively loose and were then tightened up for the remainder of the test. Once the load reached about 60% of the design load the panel began lifting off of the base when each hold down experienced uplift. More cracking noises were noted at 80% and 100% design loads as well as noticeable splitting of the bottom plate at each hold down end starting when the load reached 80%.



a) Load v. Time

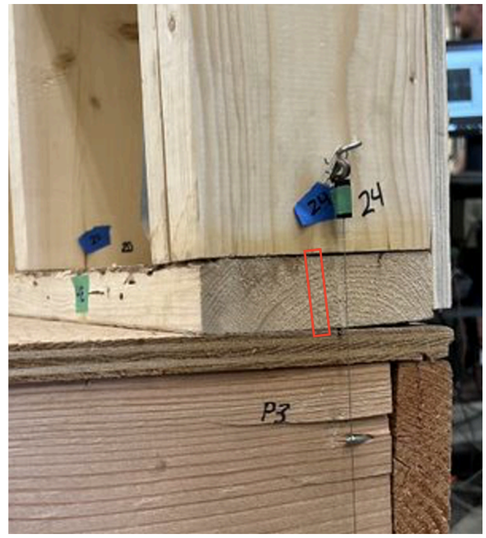


b) Load v. Displacement

Figure 5.24 – LF #2 Phase 2 Cyclic Test 1

Upon test completion, the hold downs were checked and the nut on the left side (Figure 5.25c) was found to be loose again while the right-side hold down (Figure 5.25d) nut was still tight. Figure 5.25 presents the photos from after the first cyclic test. Other damage observations include slight cracking in the bottom plate near the right-side hold down (Figure 5.25b) and an increase in the existing cracking near the left side hold down (Figure 5.25a).

a) Left side hold down (bottom plate end grain)



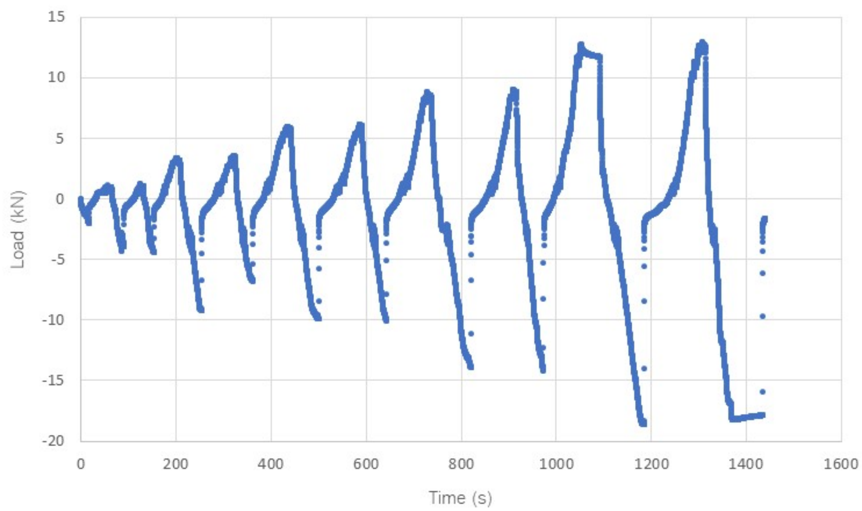
b) Right side hold down (slight cracking in bottom plate – location outlined in red)

c) Left side hold down (easily unscrewed and washer easily removed)

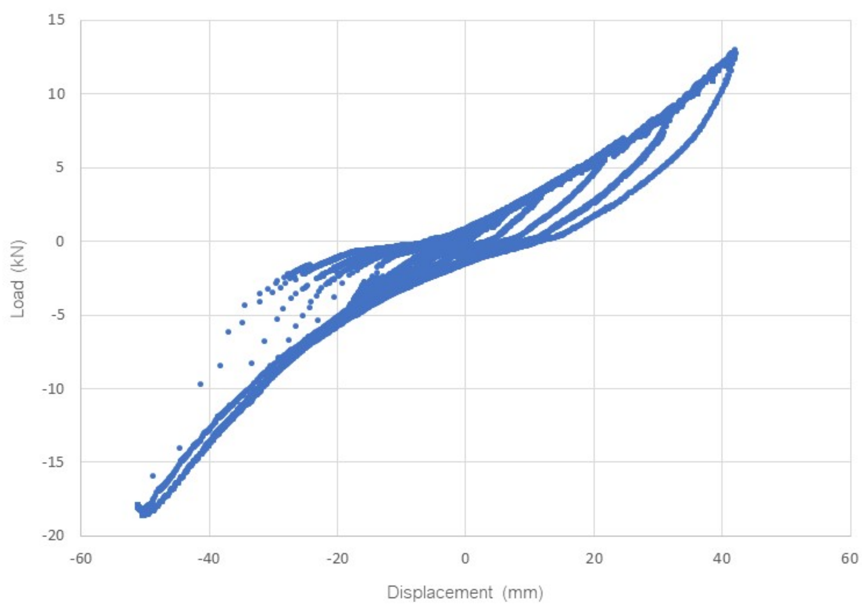
d) Right side hold down (tight to unscrew, curved washer difficult to remove – jammed in hold down)

Figure 5.25 – LF #2 After Cyclic Test 1

The second cyclic test was conducted, and the data is presented in Figure 5.26. At 40% of the design load the base (and panel) began sliding along the grade beam within the tolerance of the anchor bolt holes in the wood. More sliding was observed at around 80% of the design load as well as some cracking noises. Upon reaching 100% of the design load, more cracking noises were noted as well as more sliding back and forth along the grade beam.



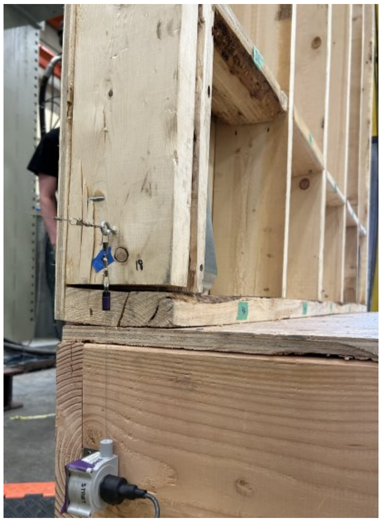
a) Load v. Time



b) Load v. Displacement

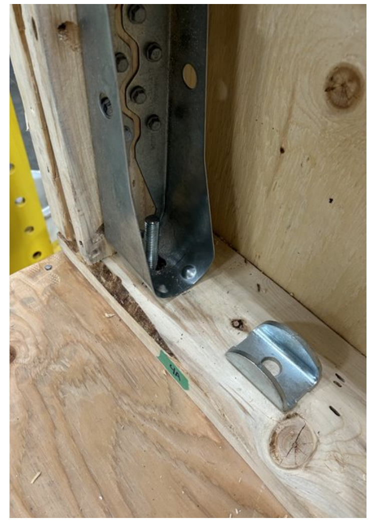
Figure 5.26 – LF #2 Phase 2 Cyclic Test 2

Observations made upon the completion of the second cyclic test were similar to those found after the first and can be found in Figure 5.27; the right side hold down was tighter with its washer more difficult to remove while the left side hold down was less effected and easily disassembled. Cracking further progressed in the bottom plate near both hold downs and some uplift can be seen between the end studs and bottom plate near the left side hold downs (Figure 5.27a).



a) Left side hold down (stud lifting & crack in bottom plate worsening)

b) Left side hold down (no damage development on inside of hold down)



c) Left side hold down (easy to remove nut and curved washer)

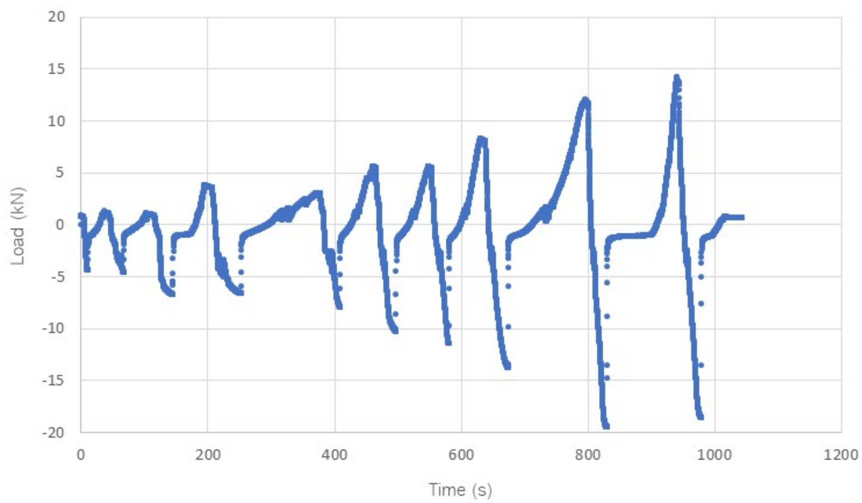
d) Right side hold down (no significant growth of crack from cyclic test 1)

e) Right side hold down (washer snug in hold down)

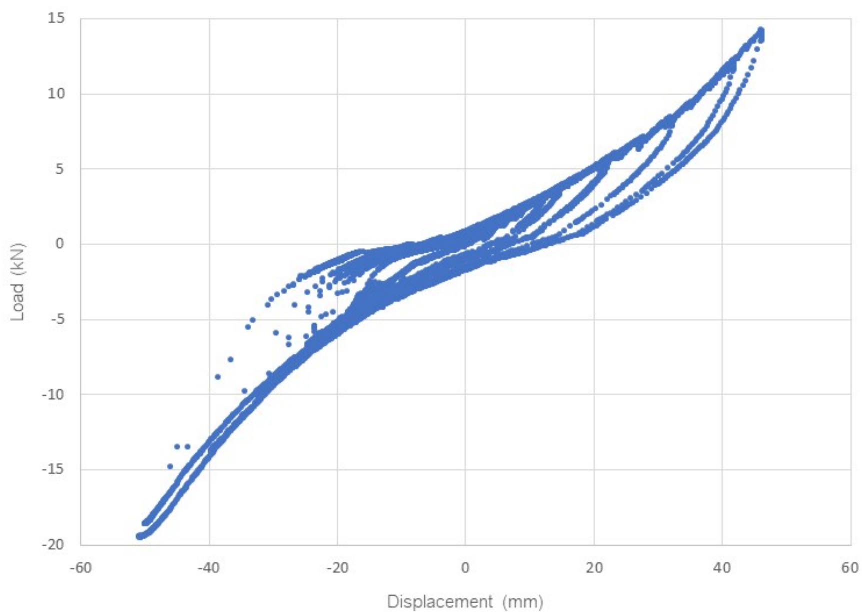
f) Right side hold down (hold down warper, washer to be hammered back into place)

Figure 5.27 – LF #2 After Cyclic Test 2

The third and final cyclic test was completed similar to the first two cyclic tests; the results are presented in Figure 5.28. Similar observations were made throughout the final test as the previous cycles of testing including cracking and sliding along the grade beam.



a) Load v. Time



b) Load v. Displacement

Figure 5.28 – LF #2 Phase 2 Cyclic Test 3

Figure 5.29 presents both hold downs and the state of the bottom plates and studs after the final round of cyclic testing was completed. Other areas highlighted after previous test cycles did not experience any other significant changes or further damage; the cracking in the bottom plate near both hold downs was amplified by the final load cycling.

a) Right side hold down (no significant additional damage since previous cyclic test)

b) Crack slightly more visible/apparent

Figure 5.29 – LF #2 After Cyclic Test 3

Once all three cyclic tests were completed, the panel was disassembled by removing the nuts and washers on the hold down anchor rods and lifting the panel (with the hold downs still screwed into the panel) and removing it from the base. The panel was then propped up on a forklift and a table at the approximate height of a truck bed (just over 1 m; limited by max table height) as shown in Figure 5.30; the panel was then dropped from this setup to simulate it being dropped off of a truck bed when being unloaded on a site. The panel was dropped twice; once on an edge (Figure 5.30a and b) and once on the corner corresponding to the right-side hold down (Figure 5.30c and d) to achieve any possible worst-case damage that might affect its lateral resistance capability in further loading conditions.

a) Drop Test #1 Starting Position

b) Drop Test #1 Mid-Drop

c) Drop Test #2 Starting Position

d) Drop Test #2 Mid-Drop

Figure 5.30 – Light-Frame Drop Tests

After both drop tests, the panel remained intact and in good enough condition to be reassembled. The only notable damages included wood crushing of the corner that hit the ground first along with some nail withdrawal through the sheathing and between the stud and bottom plate members; these can all be seen in Figure 5.31.

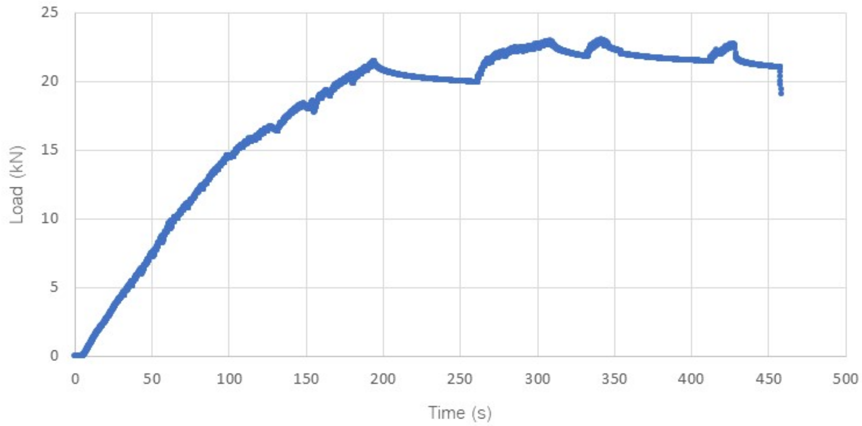
a) Right hold down corner damage

b) Studs pulling out of bottom plate

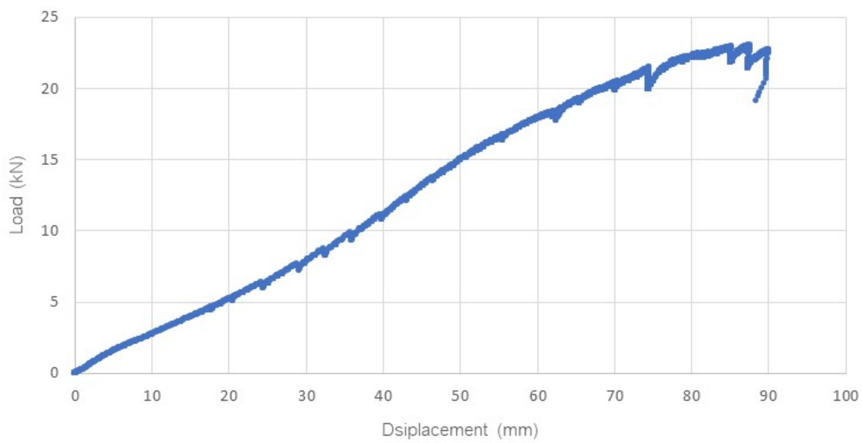
c) Nails pulling out of sheathing along studs

Figure 5.31 – LF #2 After Drop Test

After the drop tests were completed, the panel was reassembled onto the base with the anchor rods securing the panel and base system through its hold downs. In order for the rods to be inserted, slight hammering was required due to damage causing misalignments between the wall and base. Once reinstalled in the test frame, the panel was monotonically loaded, as the previous light-frame panel was in phase 1 of testing, to failure. The monotonic test results, shown in Figure 5.32, concluded that after enduring cyclic loads and being dropped, the maximum load that the panel could laterally resist was approximately 23 kN. This value is to be compared with the 28.5 kN load that was reached in Phase 1 of testing; after enduring cyclic load tests up to only its design load as well as drop tests from a height of approximately 1 m, the light-frame panel system had a 5.5 kN reduction in resistance capacity.



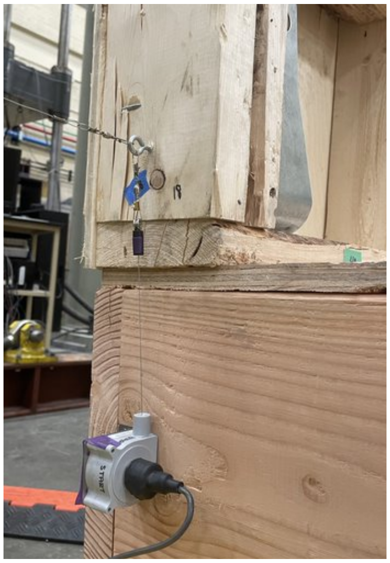
a) Load v. Time



b) Load v. Displacement

Figure 5.32 – LF #2 Phase 2 Monotonic Test

Figure 5.33 presents the locations of visible damage after the final monotonic test for Phase 2. Cracking occurred at the hold downs like in previous testing (Phase 1); in addition to this, the panel appears to have shifted on the base (Figure 5.33b) and the sheathing seems to have shifted causing nails to pull inward and toward the joints in the sheathing (Figure 5.33c and d).



a) Left side hold down (panel sliding off of base)

b) Offset between panel and base (~1.5-2 cm)

c) Intersection of plywood sheathing (not aligned)

d) Nails ripping through sheathing toward joint & gap between sheets

e) Right side hold down (bottom plate splitting due to uplift)

f) Right side hold down (bottom plate cracking at end grain & nail withdrawal between studs and bottom plate)

Figure 5.33 – LF #2 After Phase 2 Monotonic Test

5.4 Analysis & Discussion

5.4.1 General

The primary purpose behind the experimental tests was to evaluate the feasibility and durability of the CLT wall-to-floor structural system when subjected to loads representative of in-service conditions. The steel bracket (BC) fabricated and implemented for testing was designed to resist the in-plane lateral forces using only four large steel bolts at each end of the panel. The decision of limiting the number of fasteners stemmed from the need to have a connection system that is designed for disassembly and reassembly. Traditionally, the common practice in the design of wood connections and lateral systems is to use smaller diameter fasteners in a larger quantity to promote yielding of the fasteners (i.e., energy dissipation) and to distribute stresses across the surface. Since the prototype design for the relocatable panelized modular building relies on multiple reuses, it is important to compare the behaviour of the proposed CLT wall-to-floor connection to a similar wall-to-floor connection using conventional light-frame wood with traditional hold-downs to establish a comparative baseline in terms of behaviour, durability, and ease of disassembly and reassembly in addition to overall feasibility of CLT versus light-frame.

A total of two wall-to-floor (CLT #1 and LF #1) systems were tested when subjected to monotonic loading to establish the baseline behaviour in addition to inform the loading protocols to be used for the other two walls subjected to loads representative of in-service conditions. These loading protocols included in-plane cyclic loading, verification of the ease of disassembly and reassembly, drop tests to represent accidental panel drops that may occur during transportation and/or installation, and finally a monotonic test to failure. The latter was conducted to evaluate the ultimate capacity of the walls in order to determine if the durability of the wall-to-floor assemblies had been affected by the in-service loading.

5.4.2 CLT Discussion

The design of the novel CLT wall-to-floor connection moves away from traditional design guidelines provided in the Engineering Design in Wood standard (CSA O86) (Canadian Standard Association, 2019)) where the lateral strength of the wall relies on smaller diameter fasteners (e.g., lag screws, self-tapping screws, rivets) used in conjunction with angle brackets and traditional hold-downs. The proposed design relies on four large steel bolts to transfer the forces in bearing at each end. Further, for platform CLT shear walls, the CSA O86 (Canadian Standard Association, 2019) recommends a height-to-width ratio ranging from 2:1 to 4:1 to promote a rocking and sliding behaviour. Ratios below and above these limits have been observed to predominantly behave in a manner where sliding and bending are the predominant behaviours, respectively. Whereas the panels had a 1.4:1 height to width ratio, both sliding and rocking were observed in both tests which resulted in some wood crushing at the fastener locations (Figure 5.4). While the uplift was significant in CLT #1 and was attributed to the washers holding the floor-to-grade beam connection not being sufficiently large (Figure 5.4), upon fixing for CLT #2, there was still significant uplift observed during the monotonic test (Figure 5.17).

During the cyclic loading representative of in-service conditions, no visible damage was observed for CLT #2 with the wall-to-floor assembly returning to its original position upon unloading (Figure 5.8, Figure 5.10, Figure 5.12). The novel connection detailing allowed for a smooth disassembly and reassembly process after the third cyclic test prior to the wall panel being dropped. In the drop tests, the CLT was observed to experience some damage at the corners (Figure 5.13, Figure 5.14); however, it did not affect the reassembly process. While damage was evident following the reassembly (Figure 5.15), it did not significantly affect the global behaviour of the wall during the monotonic test. The loading protocol (i.e., cyclic, drop) aimed to be representative of in-service use scenarios and aimed to measure if the structural behaviour would be adversely affected by a lifecycle of disassembly, relocation, and reassembly. Figure 5.34 shows the two load-displacement graphs for CLT #1 and CLT #2 under monotonic loading where it can be seen that CLT #2 had a lower initial stiffness and ultimate capacity.

Ultimately, a strength reduction of about 20% can be observed when comparing the maximum load achieved (40 kN) during phase 2 with the phase 1 load corresponding to the same deflection (50 kN) as the maximum phase 2 load. These loads also indicate a reduction in stiffness of approximately 27.5%. Whereas the reduction in resistance and stiffness appears to be significant, the cylinder of the actuator and load cell in CLT #1 were observed to have significant rotation such that the load was no longer applied horizontally (Figure *4.8*, Figure 5.3) thus resulting in an artificially higher load due to the force now having horizontal and vertical components. This was rectified by adding a second pin connection between the end of the actuator and reaction column for CLT #2 (Figure 5.17a). It is expected that the reported reduction factors in terms of resistance and stiffness would be less had two identical setups had been employed.

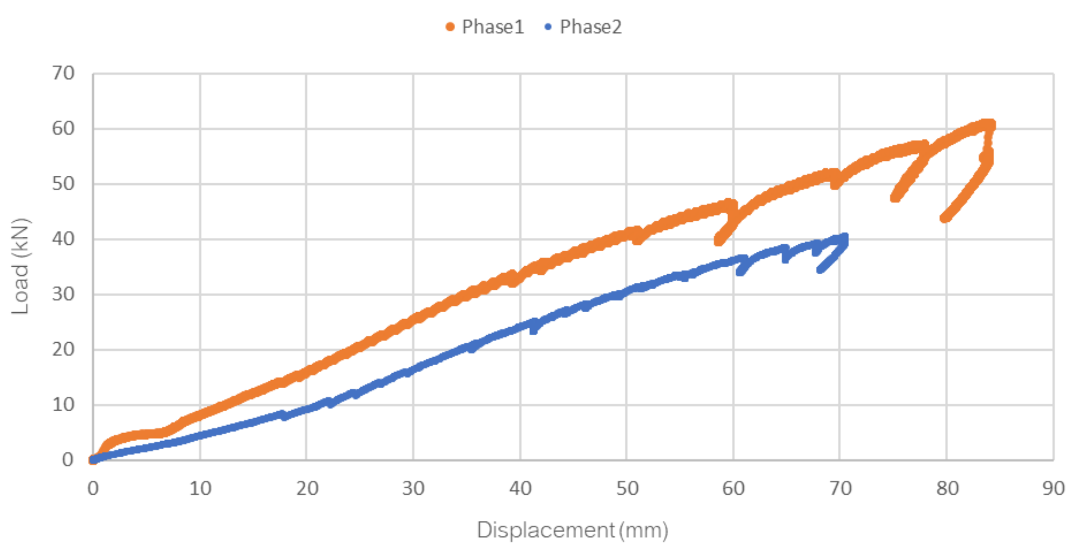


Figure 5.34 – Phase 1 and Phase 2 Monotonic Load Test Curves for CLT

5.4.3 Light-Frame Discussion

To evaluate the potential of the CLT wall-to-floor connection system, a light-frame wall-to-floor panel assembly was designed according to the CSA O86 (Canadian Standard Association, 2019) with a slight modification to accommodate for the aim of being able to disassemble and reassemble it. Therefore, the wall was designed such that the nails connecting the sheathing to the stud would not yield, and the wall panel was only attached to the floor panel using two hold-downs at either end of the wall panel. This created a similar system to the one conceived for the CLT wall-to-floor assembly, allowing for easy light-frame wall-to-floor connection disassembly and reassembly.

Based on the tests, no nail yielding or head pull-through occurred, and the over-turning forces were transferred to the base through conventional the light-frame wood wall's hold-downs. However, failure of the bottom plate was observed in both the monotonic and cyclic tests (Figure 5.22, Figure 5.33) as the forces transferred exceeded their capacity. Although the hold-downs were able to transfer the load through the floor panel and ultimately into the grade beam, it was observed that the disassembly and reassembly process did not go as smoothly as for the CLT wall-to-floor assembly. The washer in the hold-down assembly was observed to get stuck and needed to be hammered back into place (Figure 5.27). Following the drop tests, the wall panel experienced wood crushing at the corner as well as some nail withdrawal through the sheathing and between the stud and bottom plate elements (Figure 5.31). Slight hammering was required for the wall to be reassembled.

Following the reassembly, LF #2 was loaded monotonically to failure where an overall drop in strength of 20% was observed, or 12% if comparing the maximum load reached in phase 2 (23 kN) along with the phase 1 load corresponding to the same displacement (26 kN) as well as a reduction in stiffness of 23%. Figure 5.35 shows the two load-displacement curves for the light-frame wall-to-floor assembly. Whereas this drop in resistance appears to be significant, it should be noted that the cyclic test of LF #2 was exceeded by about 50% due to human error. Cracking and damage were observed to occur between 40 and 60% of the cyclical maximum load of 15 kN coinciding with the design load of 10 kN. Thus, it is hypothesized that the loss in ultimate capacity would be reduced by half had the light-frame wall-to-floor panel been loaded within the design load. Nonetheless, the current approach has shown to achieve promising results with less raw materials when compared to the CLT panels.

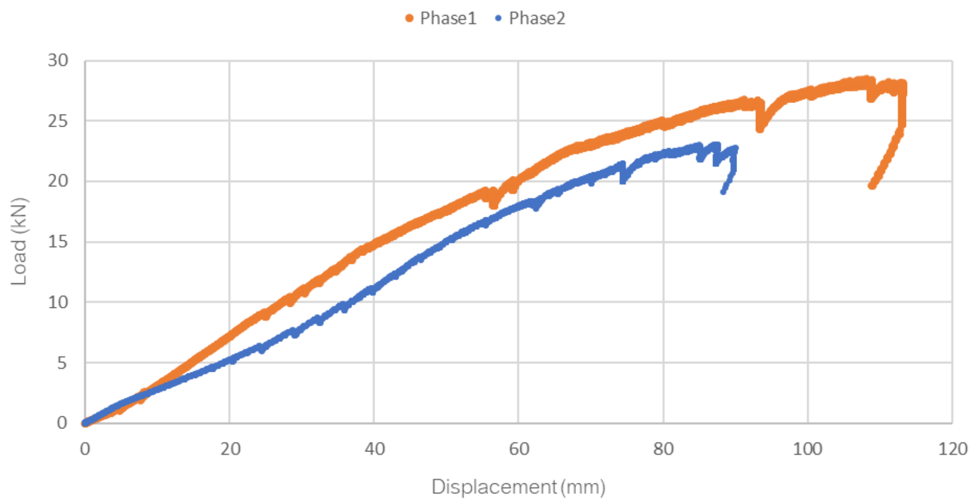


Figure 5.35 – Phase 1 and Phase 2 Monotonic Load Test Curves for Light-Frame

5.4.4 Summary

In summary, both systems performed well with no catastrophic failures during the cyclical tests as well as the final monotonic tests. The CLT wall-to-floor assembly appears to exhibit more significant strength and stiffness loss than the light-frame assembly, which was in part attributed to a deficiency in the test setup, but it was reported to being easier to disassemble and reassemble. On the other hand, despite requiring the replacement of the bottom plate and using less raw materials, the light-frame wall-to-floor assembly showed adequate behaviour but the disassembly and reassembly were not as convenient due to requiring some light force application to put everything into place.

Chapter 6 – Life Cycle Assessment

6.1 General Overview

While the CLT system in comparison to an equivalent light-frame system may prove to be more structurally durable, the added embodied carbon emissions associated with mass timber must be taken into account. As discussed in Chapter 2, mass timber products present a more environmentally sustainable alternative building material when compared to concrete and steel however, much more material, processing, and manufacturing are required to produce mass timber than dimensional lumber. Thus, if a project can be accomplished using light-frame construction (i.e., light-frame is structurally sufficient); then the traditional method should be employed.

This investigation of CLT over light-frame construction for a modular building surrounds the possible reuse of the building, thus a preliminary life cycle assessment (LCA) assessing the feasibility of a CLT system over a light-frame is completed considering various cases in which a CLT system can withstand more reuse cycles than the equivalent light-frame system. Varying these parameters allows for conclusions to be made regarding the scenario in which a CLT system would be more sustainable, in the sense of embodied carbon emissions, over a light frame system and further, what parameters are necessary for this scenario to exist. Data and observations from experimental testing (Chapter 5) will qualitatively guide the set of parameters used for the analysis.

The scope of the high-level LCA includes cradle to gate modules as well as some end-of-life phases as these modules are associated with the embodied carbon emissions. The LCA is accomplished by using Environmental Product Declarations (EPDs) for the individual materials and products in each prototype. The prototypes being compared include configurations A, B, and C, presented in Chapter 3 (with areas varying from 24 m^2 to 40 m^2) in both the CLT structural system and the equivalent light-frame system, meaning that the systems were both designed to withstand the same design load previously calculated and were both detailed to achieve similar enclosure performances (i.e., similar RSI values). The prototypes referenced in the LCAs include the floor and wall panels and the connections in between; roof panels are omitted from the analysis due to the similarities between the SIPs used in the CLT design and the equivalent light-frame structural roof system and further, the foundation system omitted from the prototype design is also omitted from the LCA. The CLT prototype input for the LCA includes the connections and insulation outlined Chapter 3 while the light-frame equivalent includes the connections outlined in Chapter 4 (experimental testing specimen) as well as a cavity and continuous rigid insulation that will be detailed in Section 6.2.2. The outputs from the LCA are then transferred to a model in which different reuse scenarios will be laid out to establish if reuse plays a role in concluding why the CLT modular building might be more feasible than its light-frame equivalent.

6.2 LCA

This section provides the preliminary life cycle assessment of the CLT and equivalent light-frame prototype design specifically considering embodied carbon emissions from LCA modules A1-A3, the production stage, as well as partial C2 (waste transport), C3 (waste processing), and C4 (waste disposal) modules, from the end-of-life stage. These modules are selected based on information available in material environmental product declarations (EPDs) and embodied carbon emissions intensive processes. Omitted from the LCA are modules A4-A5 (transport to site and construction and installation processes), B1- B5 (in-use stage), and C1 (deconstruction and demolition). Modules B1-B5 are associated with operational emissions that are dependent on building systems which are not the focus of this study while modules A4, A5 and C1 are omitted due to the extent of the factors that would affect the emissions results that are beyond this study. Both modular systems are the same size but have significantly different weights (e.g., a mass timber structure is likely 2-3 times heavier than an equivalent light-frame) and would therefore require different equipment specifications (e.g., fuel consumption).

The prototype inputs for the LCA comparison include the floor and wall panel structural systems and insulation, as these are different for each structural system type. All other assembly layers, such as roof and building envelope, would be consistent in both system types and are therefore not considered in the analysis. Further, the roof SIPs use consistent materials with a light-frame equivalent; 2x12s (two members spaced at 1m for SIPs versus one member spaced at 400 mm for light-frame), 12” of rigid insulation versus cavity and rigid insulation for light-frame, and plywood sheathing and are therefore expected to yield similar embodied carbon outputs in an LCA. The roof panels are therefore not considered in the analysis.

6.2.1 CLT Prototype Inputs

Similar to the light frame prototype inputs; the CLT prototype material requirements for the three configurations are considered. This includes the structural component (CLT and connectors) and major insulating components (XPS insulation). The structure accounted for is 105 mm thick (3-ply) CLT with RSI-3.62 and RSI-5.72 XPS insulation (for walls and floors, respectively) spanning the same surface area as the CLT structure along with two steel bracket connections per wall panel. Table 6.1 provides a summary of material requirements by prototype configuration.

Table 6.1 – CLT Prototype Material Requirements

Material	Configuration A	Configuration B	Configuration C
CLT	7.85 m^3	9.74 m^3	11.63 m^3
Steel Brackets	0.25 t	0.30 t	0.35 t
XPS Insulation	322.86 m^2 (for total RSI)	404.82 m^2 (for total RSI)	486.78 m^2 (for total RSI)

6.2.2 Light-Frame Prototype Inputs

The light-frame prototype inputs consider the three configurations of the prototype previously outlined in Chapter 3. These include a 4 m by 6 m, 4 m by 8 m, and 4 m by 10 m building. The light frame material requirements account for the structural components (wood framing, sheathing, nails, and hold downs) as well as major insulating components (XPS and batt insulation). The total volume of softwood lumber required accounts for the studs, joists blocking, bottom, and top plates, etc. (i.e., all wood members of a light-frame wall or floor panel). For comparison purposes, RSI-1.3 (50 mm) and RSI-0.7 (25 mm) XPS rigid insulation are used for the wall and floor assemblies, respectively. Further, RSI-3.8 (140 mm) and RSI-5.3 (195 mm) of cavity batt insulation are used in the wall and floor assemblies, respectively, in addition the exterior rigid insulation used, to achieve a similar RSI-value to that of the CLT assembly discussed in Chapter 3. A summary of material requirements for all three prototype configurations can be found in Table 6.2. It should be notes that since the EPDs publish the global warming potential (GWP) of insulations based on the cross section of material providing an RSI of 1, it was necessary to factor up to the actual area of material used. The values estimated for steel nails and hold downs are based on the assumption of nailing spaced at 75 mm, similar to the construction of the test panels in Chapter 4, two hold-downs per panel, and weights of approximately 85 nails per pound (8d and 10d nails) and 7849 kg/m^3 (490 lb/ft^3) for the steel plates (bracket connection) (Fastek, 2023)

Table 6.2 – Light frame Prototype Material Requirements

Material	Configuration A	Configuration B	Configuration C
Softwood Lumber	3.47 m^3	4.60 m^3	5.71 m^3
Softwood Plywood	1.58 m^3	2.01 m^3	2.44 m^3
Steel Nails	0.030 t	0.038 t	0.044 t
Steel Hold Downs	0.022 t	0.026 t	0.031 t
Batt. Insulation	279.94 m^2 (for total RSI)	346.61 m^2 (for total RSI)	413.28 m^2 (for total RSI)
XPS Insulation	68.82 m^2 (for total RSI)	86.64 m^2 (for total RSI)	104.46 m^2 (for total RSI)

6.2.3 Environmental Product Declaration (EPD) Inputs

Environmental Product Declarations (EPDs) were collected for the various materials used in the wall and floor assemblies of the CLT and comparable light frame modular prototypes. All wood products EPD were sourced from either the Canadian Wood Council or Canadian-based manufactures (i.e., CLT manufacturer) and the remaining EPDS for steel connection materials were sourced based on availability of information on the EPD and not necessarily on only Canadian-manufactured products (i.e., EPDs that presented breakdowns by module rather than a total module A-D emissions value). Table 6.3 presents the CO_2 equivalent for global warming potential (GWP) per unit of material used. These values consider LCA modules A1-A3 of the material, or the 'production stage' as well as modules C2-C4 which includes waste transport, processing, and disposal processes. The disposal process considered in all EPDs for wood-based products (e.g., lumber, plywood, CLT) is landfill for 100% of the material as this yields a worst-case scenario.

Table 6.3 – Global Warming Potential (GWP) Carbon Dioxide Equivalent (CO2e) by Material EPD

Material	Modules A1-A3 GWP [kg CO_2e]	Module C2 GWP [kg CO_2e]	Module C3 GWP [kg CO_2e]	Module C4 GWP [kg CO_2e]	Unit of Material
CLT[1]	122	-	-	124.5	Per m^3
Softwood Lumber[2]	994	-	-	135.33	Per m^3
Softwood Plywood[3]	63.12	-	-	138.42	Per m^3
Steel Nails[4]	219.32	-	1.34	-	Per metric ton
Cold-Formed Steel[5]	2380	-		-	Per metric ton
Heavy Steel Plates[6]	2600	-	1.99	-	Per metric ton
XPS Insulation[7]	6.92	0.003	-	0.014	Per m^2 of RSI-1 thickness
Batt. Insulation[8]	0.464	0.009	-	-	Per m^2 of RSI-1 thickness

[1](Element5, 2022) [3](AWC & CWC, 2020b) [5](Clark Dietrich, 2021) [7](Owens Corning, 2021)

[2](AWC & CWC, 2020a) [4](ArcelorMittal Brasil, 2019) [6](Arcelormittal Europe, 2020) [8](Owens Corning, 2018)

6.2.4 LCA Results

Using the GWP values from material EPDs, outlined in Table 6.3, along with the material quantities by system and prototype configuration in Table 6.2 and Table 6.1, LCA modules A1-A3 (material production stage) and C2-C4 (end-of-life stage) were calculated using an Excel spreadsheet and summarized in Table 6.4. These modules represent the embodied emissions associated with each component's raw material sourcing, transportation, and manufacturing processes as well as waste transport, processing, and disposal processes.

Table 6.4 – Life Cycle Assessment Breakdown by System and Configuration

Configuration	System	Modules A1-A3 [kg CO_2e]	Module C2 [kg CO_2e]	Module C3 [kg CO_2e]	Module C4 [kg CO_2e]	Total [kg CO_2e]
A	CLT	3846	1.05	0.50	982	4830
	LF	1253	2.64	0.04	689	1945
B	CLT	4775	1.32	0.60	1219	5996
	LF	1590	3.27	0.05	901	2494
C	CLT	5703	1.59	0.70	1455	7161
	LF	1927	3.91	0.06	1112	3043

The majority of each prototype's embodied carbon stems from the production stage modules (A1-A3) as well as the waste disposal module (C4) while modules C2 and C3 make up much smaller portions of the total embodied carbon. Evidently, due to the larger amount of raw material (lumber) used to manufacture CLT over light-frame materials as well as its more carbon- and energy-intensive manufacturing process, the CLT prototype for each configuration produces nearly two and a half times the amount of embodied carbon emissions as their equivalent light-frame configurations.

Biogenic carbon, or carbon sequestered and stored in wood, was not accounted for in the total embodied emissions of each system type. While it is acknowledged that wood stores carbon, the focus of the study is on the embodied carbon emitted for each system; factoring in the biogenic carbon would inevitably provide a significant advantage to the CLT system results purely based on the amount of wood used to produce the CLT over the amount of wood required in a light-frame structure and thus its ability to store a significantly larger amount of carbon than the wood required in the light-frame structure. Further, the stored carbon may not remain stored in the wood product permanently depending on its end-of-life disposal; in certain scenarios the carbon may be re-emitted into the environment.

6.3 LCA Reuse Model

6.3.1 Model Parameters

The parameters of the LCA reuse model include limitations on the number of reuse cycles for each structural system type as well as additional materials included in each reuse cycle to account for any losses in the disassembly and transportation process.

The limitations on the number of reuses for each system type are set to two to three times for the light frame system and five to six times for the CLT system. Based on the experimental testing, the light-frame panel experienced a significant reduction in capacity (over 5 kN of lateral resistance); while the panel was however, overloaded in the cyclic testing, it is assumed that when cyclically loaded to design capacity and acquiring damage from transportation and disassembly, the panel would nonetheless experience a reduction in capacity. Thus, while the system can be somewhat over-designed to account for a reduction in capacity after a use, it cannot be overdesigned such that it can be reused several times without its capacity falling below the design load. In comparison, the CLT panel behaviour was relatively consistent in both Phase 1 of testing and the monotonic test of Phase 2 up until 40 kN at which the panel tested in Phase 1 began experiencing crushing while the panel tested in Phase 2 (after a "service life") experienced failure in the base. Thus, the panel consistently withstood approximately four times its design load before failure. The CLT is therefore assigned a higher reuse limit. The model will investigate two different cases using the aforementioned reuse limitations; Case 1 considering two light-frame reuses and five CLT reuses and Case 2 considering three light-frame reuses and six CLT reuses.

In each reuse cycle (i.e., meaning the panel set has already been used at least once), a percentage of the initial material volume is added to account for the replacement of any material losses or damages that may have occurred during the previous use cycle or during the disassembly and transportation processes before reaching the new site. The parameter set for the CLT structural system is 2.5% for Case 1 as it only accounts for any potential lost connections that require replacement, for Case 2 the parameter is increased to 5% as added reuses are assumed to increase the likelihood of lost components. The parameter set for the light-frame system for Case 1 is 10%. This accounts for the replacement of damaged (e.g., bottom or top plate) or lost components. For Case 2 the parameter is increased to 20% as added reuses of the system may require the replacement of damaged hold downs (as seen in testing) or nails in addition to the Case 1 component replacement assumptions.

It should be noted while the parameters consider material replacement and limitations on the number of uses that each structural system can withstand, the LCA model does not factor it the duration of each "use". It is acknowledged that this would significantly affect the two parameters that have been considered in model; if one use of the building is 5 years versus 50 years, 50 years would significantly reduce the number of uses the building could withstand due to the long duration of its service life while likely increasing the material replacement factor (i.e., more components would need replacing after a long service life). The purpose of this study, however, is for comparison purposes between the two system types to establish the general feasibility of CLT in the proposed application.

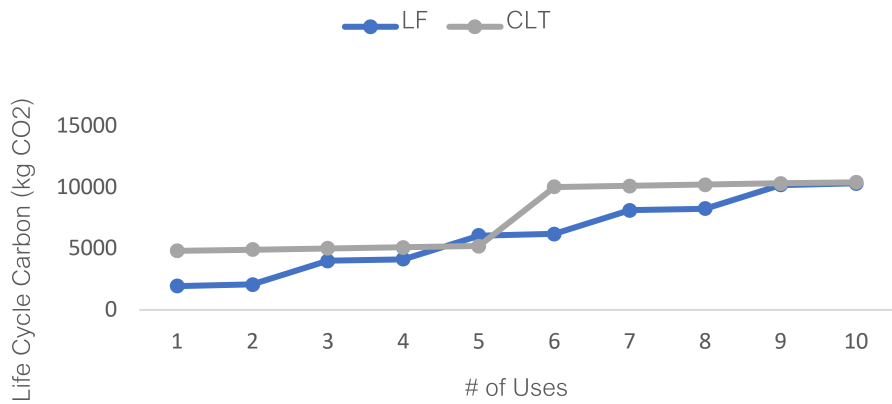
6.3.2 Reuse Model & Results

In order to investigate how the embodied carbon of each structural system type fluctuates based on varying parameters and number of uses, a model was developed to take the parameters as well as the anticipated number of reuses as inputs and output the total embodied emissions for each scenario. This is accomplished by pulling the relevant LCA module emissions of both systems for each LCA module; Figure 6.1 presents a sample of the tool used to generate the models; this presents the breakdown of Case 2 with four anticipated use cycles of configuration A of the prototype.

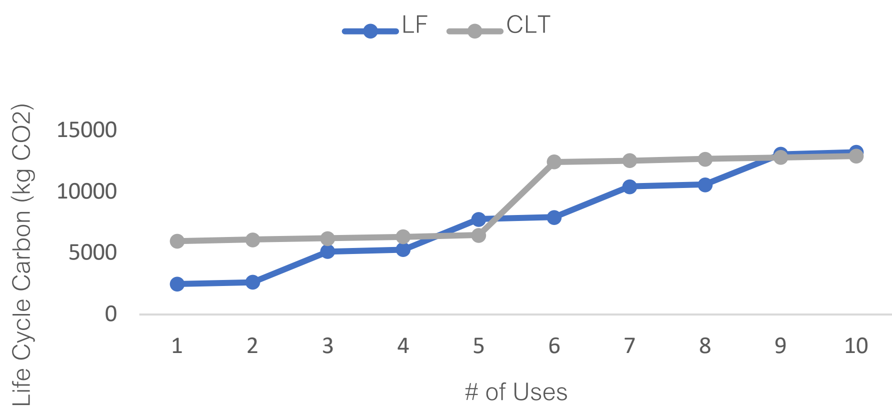
Use	Life Cycle Stage		LF	CLT
1	A1-A3	Construction materials	1253	3846
	A4	Transportation to site	-	-
	A5	Construction/installation process	-	-
	C1	EoL - Deconstruction/demolition	-	-
	C2	EoL - Waste transportation	-	-
	C3	EoL - Waste processing	-	-
	C4	EoL - Waste disposal	-	-
	D	External impacts	0	0
	Total:		1253	3846
	Cumulative Total:		1253	3846
2	A1-A3	Construction materials	251	192
	A4	Transportation to site	-	-
	A5	Construction/installation process	-	-
	C1	EoL - Deconstruction/demolition	-	-
	C2	EoL - Waste transportation	-	-
	C3	EoL - Waste processing	-	-
	C4	EoL - Waste disposal	-	-
	D	External impacts	0	0
	Total:		251	192
	Cumulative Total:		1503	4039
3	A1-A3	Construction materials	251	192
	A4	Transportation to site	-	-
	A5	Construction/installation process	-	-
	C1	EoL - Deconstruction/demolition	-	-
	C2	EoL - Waste transportation	3	-
	C3	EoL - Waste processing	0	-
	C4	EoL - Waste disposal	689	-
	D	External impacts	0	0
	Total:		942	192
	Cumulative Total:		2446	4231
4	A1-A3	Construction materials	1253	192
	A4	Transportation to site	-	-
	A5	Construction/installation process	-	-
	C1	EoL - Deconstruction/demolition	-	-
	C2	EoL - Waste transportation	3	1
	C3	EoL - Waste processing	0	1
	C4	EoL - Waste disposal	689	982
	D	External impacts	0	0
	Total:		1945	1176
	Cumulative Total:		**4391**	**5407**

Figure 6.1 – LCA Breakdown (in kg CO_2e) for Case 2 and Four Uses of Configuration A

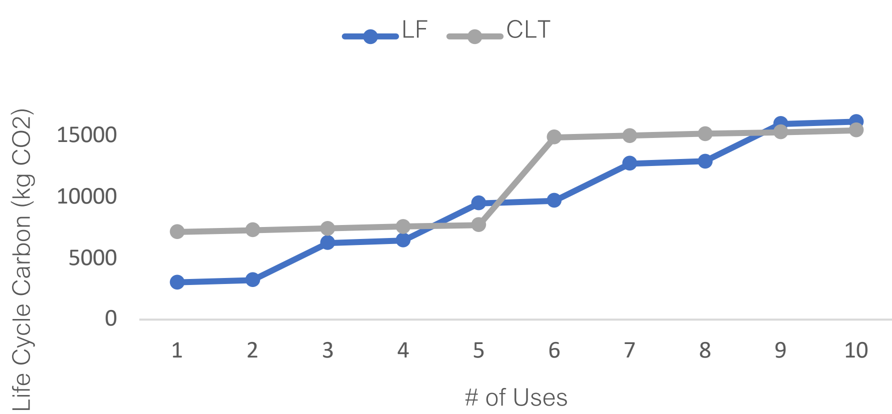
The tool used to create the emissions breakdown for the scenario presented in Figure 6.1 was used to generate models for both Case 1 and Case 2 considering one through ten uses for each configuration of the prototype. These models (per prototype configuration) can be found in Figure 6.2 and Figure 6.3. A complete breakdown of the carbon emissions accounted for by case and configuration can be found in Appendix C.



a) Configuration A



b) Configuration B

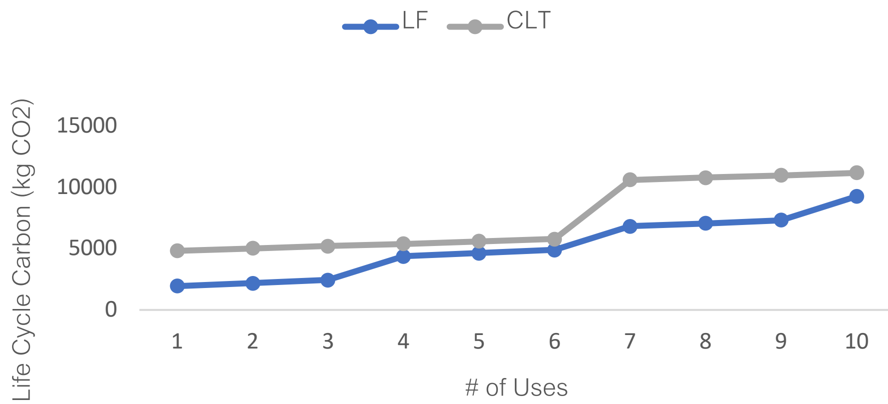


c) Configuration C

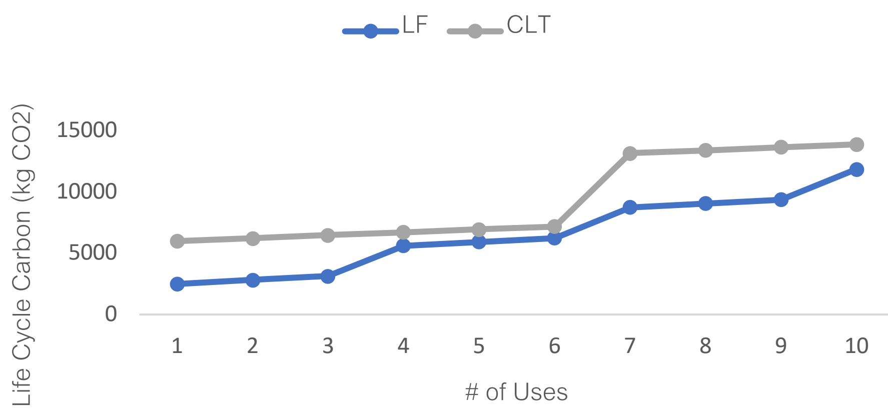
Figure 6.2 – Case 1 Model

As previously noted, Case 1 considers the following parameters: two- and five-reuse limits and 10% and 2.5% material replacement for light-frame and CLT, respectively. Figure 6.2 ((a), (b), and (c)) indicate that the carbon emissions graphs intersect at certain points relative to the number of uses (x-axis) but the system with the lower overall embodied carbon emissions (y-axis) fluctuates. It should also be noted that varying the configuration does not drastically affect the comparison. Thus, for any size configuration, the number of anticipated uses dictates which system produces an overall lower cumulative amount of embodied carbon emissions; the light-frame system, however, is more frequently the lower carbon emitting system.

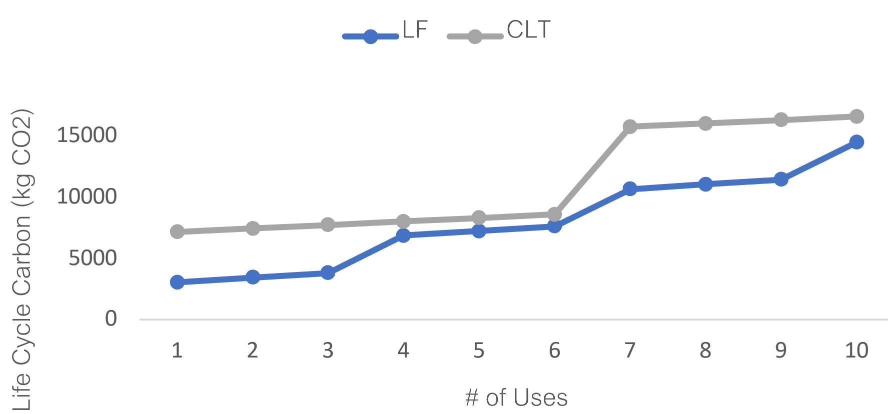
Case 2, presented in Figure 6.3, considers the following parameters: three- and six-reuse limits and 20% and 5% material replacement for light-frame and CLT, respectively. With the parameters set for this case, the light-frame and CLT results begin to converge but at no point do the CLT embodied carbon emissions fall below the light-frame. When the light-frame system can withstand an additional use, even considering a much higher material replacement percentage for each reuse, this still brings the cumulative emissions to a lower value than that of the CLT system.



a) Configuration A



b) Configuration B



c) Configuration C

Figure 6.3 – Case 2 Model

6.4 Summary of LCA Observations

The LCA and case models show that in general, CLT is a reasonable material to consider in the application of a small-scale relocatable building. While varying the cases, and therefore the parameters of reuse and material replacement, the cumulative emissions for each system did not vary greatly. It is evident that the limit on the number of reuses per system and the total anticipated number of uses of the building dictate which system emits less embodied carbon in Case 1 (i.e., when 5 reuses are required and the CLT can withstand exactly 5 reuses, the CLT system becomes the better choice). However, as the results were relatively close in both cases (graphs generally close together) there will not be a clear ideal choice of system type over the other by varying the parameters however, the light-frame system in most scenarios explored in the two cases is the lower carbon emitting system. It should, however, be noted that the parameters that were set in the cases explored in this study were assumptions guided by the limited literature reviewed and experimental testing conducted and as previously mentioned, do not factor in the anticipated duration of each use or the overall service life. Additional research and experimental investigation would be required to gain more insight into how the different structural systems withstand repeated assembly and use and how varying the expected duration of each use might affect the reuse limit and material replacement parameters.

Chapter 7 – Conclusions and Future Recommendations

7.1 Summary and Conclusions

A detailed literature review on the topics of modular construction and the state of the industry in Ontario, the circular economy and its overlap with the wood industry, wood as a building material and its use in modular construction identified a need for relocatable panelized modular structures made from sustainable materials and designed for disassembly and reassembly to meet the various housing demands across Ontario. A prototype design was completed; this included a full panel set, a complete structural design of the panels and overall assembled structure, a preliminary building enclosure to be prefabricated on each panel, and finally a set of conceptualized connections designed for ease of disassembly and reassembly.

In order to assess the feasibility of the prototype, experimental testing and a life cycle assessment were conducted. The aim for the experimental testing was to assess the structural behaviour, potential for disassembly and reassembly, as well as the overall durability of the proposed CLT assembly relative to an equivalent light-frame alternative in order to determine whether the use of the CLT panel is truly necessary for added durability over conventional light-frame solutions. The portion of the prototype design selected for testing included a wall-to-floor assembly that was expected to experience the greatest lateral load within the prototype. One specimen of each system type was tested to failure under monotonic in-plane loading while a second was cyclically tested to simulate a worst-case service life scenario that included drop tests to simulate damage incurred through transportation or reassembly processes, and finally tested to failure under monotonic in-plane loading to determine the residual strength.

The CLT wall-to-floor system withstood the design load for multiple cycles without significant visual damage throughout the assembly and despite some damage at the corners of the panels induced by the drop tests, it reached a lower ultimate capacity than the wall tested under monotonic loading only. The novel connection design proved to be successful in the disassembly and reassembly process. The predominant failure mode was the crushing of wood at the fastener locations as well as at the ends of the wall panel.

The light-frame wall-to-floor assembly was adapted from conventional lateral detailing to prevent any yielding of the fasteners in the sheathing-to-stud connections, thereby relying solely on traditional hold-downs, in order to meet the requirements of disassembly and reassembly. While the light-frame wall-to-floor assembly performed reasonably well, damage to the bottom plate was observed and the need for a light application of force was required for the disassembly and reassembly process. Although a loss of 20% of ultimate capacity was observed in the light-frame wall-to-floor assembly under cyclic versus monotonic loading was observed, it is hypothesized that the loss in strength would be reduced to about 10% as it was loaded to nearly 200% of its capacity.

Upon conducting the life cycle assessment for both the CLT and light-frame equivalent, it was found that the CLT prototype emits approximately two times as much embodied carbon as the light-frame. The LCA results were then input into a reuse model with additional parameters including limits on the number of reuses per structural system type and material replacement percentages for reuse to obtain a comparison of the two structural systems under varying anticipated numbers of reuses. It was concluded through running two different cases in this model that when a CLT system can withstand two or three more reuses than the light-frame system, both alternatives have relatively similar cumulative embodied carbon emissions. While this is subject to change based on the input parameters, the parameters would need to change significantly to have a significant effect on the outcome (i.e., to conclude a clear choice on which system emits less embodied carbon in all cases or scenarios). Thus, the preliminary LCA that was conducted concludes that the use of CLT in the proposed application is in fact feasible from an embodied emissions standpoint.

7.2 Recommendations for Future Work

Based on the research described in this thesis, it is recommended to:

- Refine and optimize the building prototype components and detailing based on the findings herein
- Conduct further experimental testing investigating the reassembly and disassembly scenario to further reflect on-site conditions and potential situations
- Refine the design detailing of Connection BC
- Explore how including openings in the wall panels affects structural behavior and LCA results
- Further explore the foundation system and connections between the foundation and the structure
- Complete a more comprehensive life cycle assessment including service life durations as a parameter/factor in the assessment as well as factoring in additional LCA modules (i.e., modules A4-A5, B1-B5, and C1), and other environmental impacts on the building.

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Appendix A: Design Calculations

A.1 External Load Calculations

A.1.1 Snow Load

Snow:

$I_s = 1.0$
$S_s = 3.6$ (Chapleau)
$S_r = 0.4$
$C_b = 0.8$
$C_w = 1.0$
$C_s = 1.0$ (roof slope < 30°)

$S = I_s [S_s(C_b C_w C_s C_a) + S_r]$
$= 3.28\ kPa$ (68.5 psf)
$\Rightarrow 3.28\ kPa \times \frac{4.88m}{2} \times 2m$
$= 16\ kN \Rightarrow 8\ kN/m$

$C_b = 0.8$ for $l_c \le \frac{70}{C_w^2}$ $\Rightarrow 2w - \frac{w^2}{l} \le \frac{70}{C_w^2}$ $\Rightarrow 2(4) - \frac{4^2}{6} \le \frac{70}{1.0^2}$ ✓

A.1.2 Wind Load

Wind:

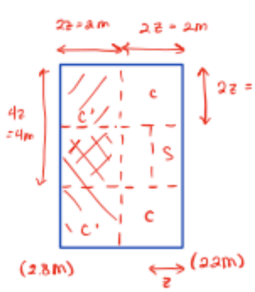
$I_w = 1.0$
$q = 0.55$ (Kincardine / Port Elgin / Goderich)
$C_e = 0.9$
$C_t = 1.0$ (conservative)

(a) $C_e = \left(\frac{h}{10}\right)^{0.2} > 0.9$
$= \left(\frac{2.2}{10}\right)^{0.2} > 0.9$
$= 0.74 \ngtr 0.9 \Rightarrow C_e = 0.9$ ← use larger value

(b) $C_e = 0.7\left(\frac{h}{12}\right)^{0.3} > 0.7$
$= 0.7\left(\frac{2.2}{12}\right)^{0.3} > 0.7$
$= 0.42 \ngtr 0.7 \Rightarrow C_e = 0.7$

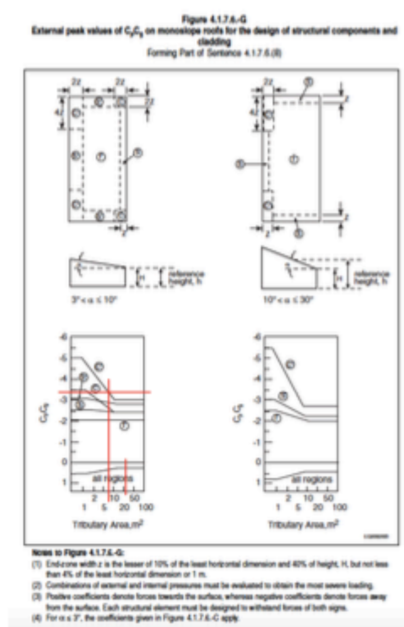
$C_pC_g \rightarrow$

$z = \min(0.1 \times 4m,\ 0.4 \times 2.2) \ge 1$
$\therefore,\ z = 1m$



		$-C_pC_g$
$c' = 8m^2$	→	-3.4 (governs)
$c = 4m^2$	→	-3
$s = 2m^2$	→	-2.6
$r = 2m^2$	→	-2
$s' = 0m^2$	→	–

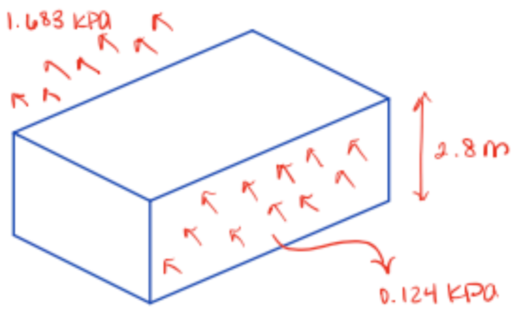
all regions $= 24m^2 \rightarrow$ $+C_pC_g$ 0.25

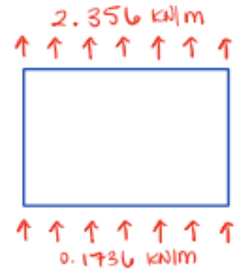


Notes to Figure 4.1.7.6-G:
(1) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.
(2) Combinations of external and internal pressures must be evaluated to obtain the most severe loading.
(3) Positive coefficients denote forces towards the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand forces of both signs.
(4) For α ≤ 3°, the coefficients given in Figure 4.1.7.6-C apply.

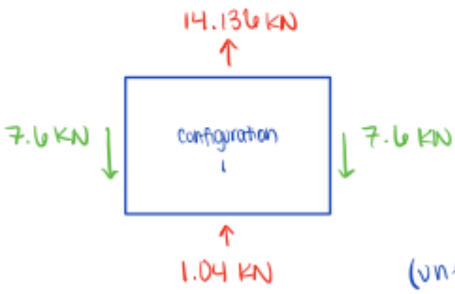
$p = I_w q C_e C_t C_p C_g = 1.0(0.55)(0.9)(1.0)(3.4) = 1.683\ kPa$
$p' = 1.0(0.55)(0.9)(1.0)(0.25) = 0.124\ kPa$

⇒ for Diaphragm / Shearwall design



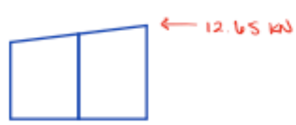


Roof / floor loading : (plan view)

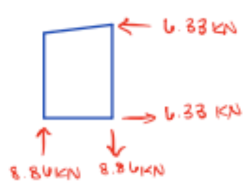


(unfactored)

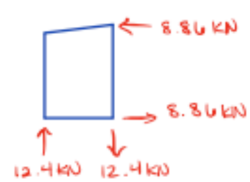
Wall Panel overturning Loads : (unfactored)



unfactored wind :



factored wind :



A.1.3 Seismic Load

Seismic Loading: → for Orleans, Ottawa for worst case values.

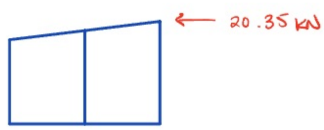
Minimum Lateral Earthquake Design Force : $V_S = S(T_a) M_v I_E W / (R_d R_o)$

$I_E = 1.0$
$S_a(0.2) = 0.474$
$S_a(0.5) = 0.252$
$S_a(1.0) = 0.124$
} based on location
$M_v = 1.0$
$W = 74$ kN
$R_d = 1.0$
$R_o = 1.0$

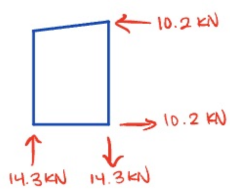
→ bracing type: shearwalls
→ lateral resisting system type: walls, wall frame systems

$V_S = 40.7$ kN

Wall Panel Overturning Loads: (unfactored)



Per panel: (unfactored + factored since factor of 1.0 for earthquake)

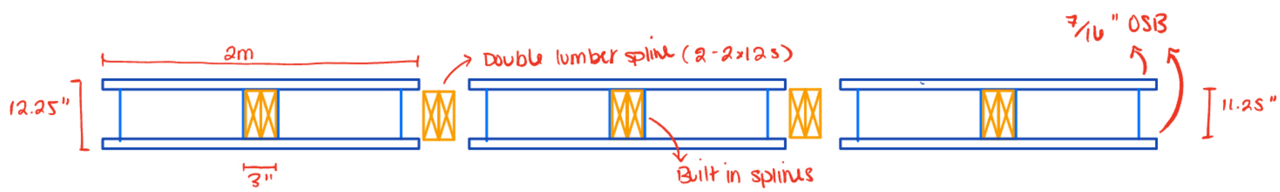


A.2 Structure Self-Weight Load

A.2.1 Roof Panel Weight

Self-weight calculated per major component of SIP: lumber spline, OSB sheathing, EPS insulation.

Roof Panels (Thermapan 4' × 16' 12.25" - Double Lumber spline)



WDM for SPF   4 - 2×12's

Lumber → $420\ kg/m^3 \times 0.038m \times 0.286m \times 4.87m \times 4 = 88.9\ kg$

OSB → $6.83\ kg/m^2$ (7/16")

$= 6.83\ \frac{kg}{m^2} \times 2m \times 4.87m \times 2$ layers

$= 133.05\ kg$

EPS → $45.65\ \frac{kg}{m^3} \times 0.286m \times 4.87m \times 1.85m = 117.6\ kg$

Total → $88.9\ kg + 133.05\ kg + 117.6\ kg$

$= 339.6\ kg$ / panel

$= 3.33\ kN$ / panel

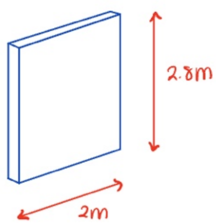
⇒ $\frac{3.33\ kN}{2} = 1.66\ kN$

$\frac{1.66\ kN}{2m} = 0.83\ kN/m$ (dead load imposed along each wall panel)

A.2.2 Wall Panel Weight

Largest wall panel in set selected for heaviest loading consideration (2m by 2.8m panel):

CLT:

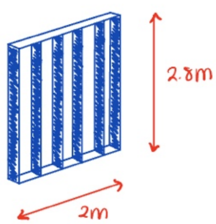


3-ply (105mm) CLT panel

(weight of CLT)

$470\ \frac{kg}{m^3} \times 2m \times 2.8m \times 0.105m = 276.4\ kg\ /\ panel$

Light frame:



SPF 2×6 stud wall

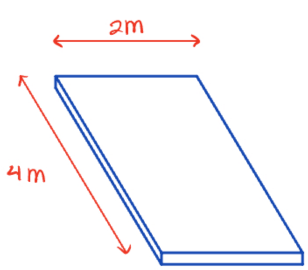
(value for 2×6 from WDM)

$110.1\ N/m^2 \times 2m \times 2.8m = 616.56\ N = 0.62\ kN\ /\ panel$

(or 62.9 kg/ panel)

A.2.3 Floor Panel Weight

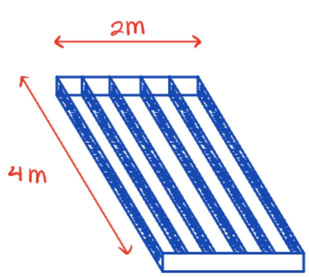
CLT:



3-ply (105 mm) CLT

$470\ kg/m^3 \times 2m \times 4m \times 0.105m = 394.8\ kg\ /panel$

Light frame:



SPF 2×12 floor panel

$220.2\ N/m^2 \times 2m \times 4m = 1.76\ kN\ /panel$

(or 179.8 kg / panel)

A.3 Prototype Structural Member Design

A.3.1 Roof Panel Design

$$Applied\ loads = Dead\ load +\ Snow\ load =\ 0.2\ kPa +\ 3.28\ kPa = 3.48\ kPa =\ 72.7\ psf$$
$$(Thermpan\ Allowable\ Design\ Loads\ based\ on\ ASD\ load\ combination: D + S)$$

$$Design\ load =\ 98\ psf$$
$$(per\ Thermapan\ Design\ Table\ and\ {}^{1}/_{180}\ deflection\ limit\ according\ to\ CSA\ O86)$$

Transverse Allowable Design Load (PSF)
Spline Type: Double Lumber Spline NLGA SPF No. 2 or Better Grade

SIP Thickness (inches)	Deflection Limit	Panel Length (feet)				
		8	10	12	14	16
4.5	L/480	38	27	19		
	L/360	51	37	26		
	L/240	76	56	39		
	L/180	91	74	52		
6.5	L/480	83	50	38	25	17
	L/360	110	68	51	33	23
	L/240	164	106	77	51	35
	L/180	164	121	80	67	47
8.25	L/480	94	75	63	45	28
	L/360	128	102	85	60	38
	L/240	178	142	119	89	56
	L/180	178	142	119	91	73
10.25	L/480	106	85	71	61	46
	L/360	142	113	94	81	62
	L/240	191	153	127	109	93
	L/180	196	153	127	109	98
12.25	L/480	106	85	71	61	46
	L/360	142	113	94	81	62
	L/240	191	153	127	109	93
	L/180	191	153	127	109	98

Thermapan Design Table

A.3.2 Wall Panel Design

3-ply (105mm) CLT Wall Panel:

Applied Loads:

Snow (S) = $3.28\text{ kPa} \times \frac{4.87\text{m}}{2} = 8\text{ kN/m}$

Dead (D) = $0.83\text{ kN/m} + \left(0.2\text{ kPa} \times \frac{4.87\text{m}}{2}\right) = 1.32\text{ kN/m}$

self-weight of SIPs

ceiling fixtures on interior of SIPs

Wind (W) = 1.81 kPa

LC #1 : $1.25D + 1.5S + 0.4W$

$P_{f1} = 1.25(1.32) + 1.5(8) = 13.65\text{ kN/m}$

$W_{f1} = 0.4W = 0.72\text{ kPa}$

LC # 2 : $1.25D + 0.5S + 1.4W$

$P_{f2} = 1.25(1.32) + 0.5(8) = 5.65\text{ kN/m}$

$W_{f2} = 1.4(1.81) = 2.5\text{ kPa}$

Design Loads : (from WDM for 3m E2 3-ply CLT)

$P_r = 65.2\text{ kN/m}$

$W_r = 27.3\text{ kPa}$

Design checks:

$P_r > P_{f1} = 13.65\text{ kN/m},\ P_{f2} = 5.65\text{ kN/m}$ ✓

$W_r > W_{f1} = 0.72\text{ kPa},\ W_{f2} = 2.5\text{ kPa}$ ✓

$V_r = 1.15 \times 39.7\text{ kN/m} = 45.7\text{ kN/m}$

$V_f = \frac{2.5\text{ kPa} \times 3\text{m}}{2} = 3.75\text{ kN/m} < V_r$ ✓

$$\Delta = \frac{5(0.75 \times 1.81\text{ kPa})(3000\text{mm})^4}{384(958 \times 10^9)} + \frac{2.68(3000\text{mm})^2}{8(7.98 \times 10^6)} = 1.87\text{mm} < \frac{L}{180} = 16.7\text{mm}\ ✓$$

2x6 Stud Wall Panel:

⇒ same applied loads as CLT calc

LC # 1: $P_{f1} = 13.65\ kN/m \times 0.4m = 5.46\ kN$ (per stud)
$W_{f1} = 0.72\ kPa \times 0.4m = 0.288\ kN/m$ (per stud)

LC # 2: $P_{f2} = 5.65\ kN/m \times 0.4\ m = 2.26\ kN$ (per stud)
$W_{f2} = 2.5\ kPa \times 0.4\ m = 1\ kN/m$ (per stud)

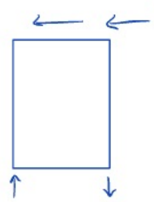
Design loads: (from WDM for 3m SPF No.1/No.2 38 × 140 mm stud wall)

$P_r = 8.35\ kN$ (per stud)
$W_r = 2.19\ kN$ (per stud)

Design checks:

$P_r > P_{f1} = 5.46\ kN$, $P_{f2} = 2.26\ kN$
$W_r > W_{f1} = 0.288\ kN/m$, $W_{f2} = 1\ kN/m$

Shearwall Design: (sheathing + nailing)



$V_{fs} > \frac{10.2kN}{2m} = 5.1\ kN/m$

2.5" (3.25mm ϕ) nails spaced @ 75 mm on 12.5mm SPF plywood sheathing:
$V_{rs} = 10.1\ kN/m >> V_{fs}$

⇒ HDU4 hold downs at each end (up to 14.6 kN tension)

applied load

$T_R = 14.6\ kN \Rightarrow \frac{14.6\ kN}{2.9\ mm} = \frac{14.3\ kN}{x} \Rightarrow x = 2.84\ mm$

defl. of HDU4 at allowable load

Deflection : (of blocked shearwall)

$$\Delta_{sw} = \frac{2VH_s^3}{3EAL_s} + \frac{VH_s}{B_v} + 0.0025\,H_s\,e_n + \frac{H_s}{L_s}d_a = 8.4\text{mm}$$

$$\left\{\begin{array}{l} V = 5.1\text{ N/mm} \\ H_s = 2800\text{ mm} \\ E = 9500\text{ MPa} \\ A = 2(5320\text{ mm}^2) \\ L_s = 2000\text{ mm} \\ B_v = 5700\text{ N/mm} \\ d_a = 2.84\text{ mm} \end{array}\right. \qquad e_n = \left[\frac{0.013\,Vs}{d_F^2}\right]^2 \Rightarrow \begin{array}{l} s = 75\text{mm} \\ d_F = 3.25\text{mm} \end{array} \qquad e_n = 0.2216$$

Δ_{max} for seismic drift $= 0.025\,h_s$ (per 4.1.8.13 NBCC 2015)

$$= 0.025(2800)$$
$$= 70\text{ mm} > \Delta_{sw} = 8.4\text{mm} \ \checkmark$$

$\Rightarrow$ wind check : $\Delta_{sw} = 4.84\text{mm}$

$$\left\{\begin{array}{l} V = 3.165\text{ N/mm} \\ d_a = 1.76\text{ mm} \end{array}\right\}$$

Δ_{max} for service wind drift $= \frac{h_s}{500} = \frac{2800}{500} = 5.6\text{mm} > 4.84\text{ mm} \ \checkmark$

(per 4.1.3.5 NBCC 2015)

A.3.3 Floor Panel Design

3-ply (105mm) CLT Floor Panel:

Applied Loads :

Dead (D) = 1.5kPa (assumption for partitions, etc.)

Live (L) = 2.4 kPa

$W_f = 1.25D + 1.5L = 1.25(1.5) + 1.5(2.4) = 5.475 \text{ kPa}$

$M_f = \dfrac{W_f L^2}{8} = \dfrac{\left(5.5\frac{kN}{m^2} \times 2m\right)(4m)^2}{8} = 21.9 \text{ kN}\cdot\text{m}/\text{m}$

$V_f = \dfrac{W_f L}{2} = \dfrac{\left(5.5\frac{kN}{m^2} \times 2m\right)(4m)}{2} = 21.9 \text{ kN}/\text{m}$

Design Loads : (from WDM for E2 3-ply CLT)

$M_r = 32.4 \text{ kN}\cdot\text{m}/\text{m}$
$V_r = 39.7 \text{ kN}/\text{m}$

Design Checks :

$M_r > M_f = 21.9 \text{ kN}\cdot\text{m}/\text{m}$ ✓
$V_r > V_f = 21.9 \text{ kN}/\text{m}$ ✓

Serviceability : (from WDM for E2 3-ply CLT, D = 1.5 kPa, L = 2.4 kPa)

Maximum span = 4.12 m > 4m ✓

2x12 Floor Panel:

Applied Loads:

Dead (D) = 1.5 kPa (assumption for partitions, etc.)

Live (L) = 2.4 kPa

$W_f = 1.25D + 1.5L = 1.25(1.5) + 1.5(2.4) = 5.475 \text{ kPa} \times 0.4\text{m} = 2.19 \text{ kN/m}$

$M_f = \frac{W_f L^2}{8} = \frac{(2.19 \text{ kN/m})(4\text{m})^2}{8} = 4.38 \text{ kN}\cdot\text{m}$

$V_f = \frac{W_f L}{2} = \frac{(2.19 \text{ kN/m})(4\text{m})}{2} = 4.38 \text{ kN}$

Design Loads: (from WDM SPF No.1/No.2 38×286mm joists)

$M_r = 7.7 \text{ kN}\cdot\text{m}$
$V_r = 13.7 \text{ kN}$

Design Checks:

$M_r > M_f = 4.38 \text{ kN}\cdot\text{m}$ ✓
$V_r > V_f = 4.38 \text{ kN}$ ✓

Serviceability: (from WDM SPF single span joists without topping, 38×286mm spaced @ 400 mm o/c)

Vibration-controlled span = 4.87 m < 4m ✓

A.4 Prototype Connection Design

A.4.1 Applied Loads

Loads on floor - wall connections:

↱ weight of wall panel

Dead = 1.32 kN/m + 1.35 kN/m = 2.67 kN/m = 2.67 kN per connector

↳ weight of roof panel (incl. ceiling finishes / fixtures)

Snow = 8 kN/m = 8 kN per connector

Wind = 8.86 kN (uplift, downward)

↳ unfactored

Earthquake = 14.3 kN (uplift, downward)

Load combinations:

① $1.4D = 1.4(2.67 \text{ kN}) = 3.74 \text{ kN}$

3A $1.25D + 1.5S + 0.4W = 1.25(2.67) + 1.5(8) + 0.4(8.86) = 18.9 \text{ kN}$ (down)

3b $0.9D + 1.5S + 0.4W = 0.9(2.67) + 1.5(8) + 0.4(-8.86) = 10.9 \text{ kN}$

4A $1.25D + 1.4W + 0.5S = 1.25(2.67) + 1.4(8.86) + 0.5(8) = 19.7 \text{ kN}$

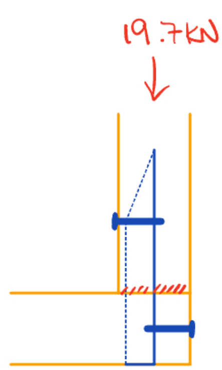
4B $0.9D + 1.4W = 0.9(2.67) + 1.4(-8.86) = -10 \text{ kN}$ (uplift)

5A $1.0D + 1.0E = 1.0(2.67) + 1.0(14.3) = 16.97 \text{ kN}$ (down)

5B $1.0D + 1.0E = 1.0(2.67) + 1.0(-14.3) = -11.63 \text{ kN}$ (uplift)

A.4.2 CLT Design Checks

① Bearing (CLT) : $Q_r = \phi F_{cp} A_b K_b K_{zcp}$



$\phi = 0.8$

$F_{cp} = f_{cp}(K_D K_{Scp} K_T) = 5.3 \text{ MPa}(1.15 \times 1.0 \times 1.0)$

$A_b = 105 \text{ mm} \times 2000 \text{ mm}$

$K_b = 1.0$

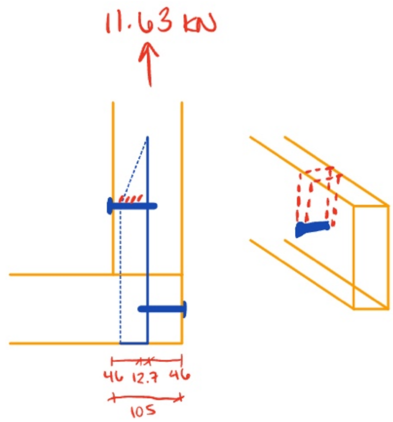
$K_{zcp} = 1.0$

$Q_r = 0.8(5.3 \text{ MPa})(1.15)(210000 \text{ mm}^2)$

$Q_r = 1024 \text{ kN}$

$Q_r \gg 19.7 \text{ kN}$ ✓

② Bearing (against bolts): $Q_r = \phi F_{cp} A_b K_B K_{Zcp}$



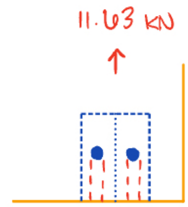
$A_b = 2 \times (46\,mm \times 33\,mm)$

$Q_r = (0.8)(5.3\,MPa)(1.15)(3036\,mm^2)(1.0)(1.0)$

$Q_r = 14.8\,kN$

$Q_r > 11.63\,kN$ ✓

③ Row Shear



$PR_{r1} = \phi_w PR_{min} n_R$

↳ $PR_{min} = 1.2 f_v (K_D K_{sv} K_T) K_{ls} t\, n_c\, a_{cri}$

$\phi_w = 0.7$

$f_v = 1.5\,MPa$

$K_D = 1.15$

$K_{ls} = 0.65$ (side member)

$t = 46\,mm$

$n_c = 1$

$a_{cri} = \min(S_R, a_L) = 200\,mm$

$PR_{min} = 1.2(1.5\,MPa)(1.15)(0.65)(46\,mm)(1)(200\,mm)$

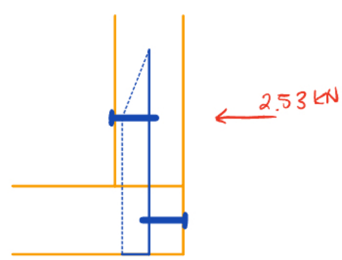
$= 12.38\,kN$

$PR_{r1} = 0.7(12.38\,kN)(2)$

$PR_{r1} = 17.332\,kN$

$PR_{r1} > 11.63\,kN$ ✓

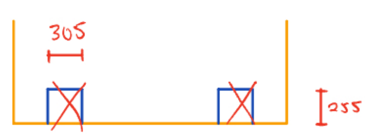
④ Flatwise Shear (punch-out) : $V_r = \phi F_s \cdot \frac{2}{3} A_g$



$\phi = 0.9$

$F_s = f_s(K_D K_H K_{sv} K_T) = 0.5\,MPa\,(1.15)$

$A_g = (305 \times 46) + 2(260 \times 46)$



$V_r = 0.9(0.5\,MPa)(1.15) \cdot \frac{2}{3}(37950\,mm^2)$

$V_r = 13.09\,kN$

$V_r > 2.53\,kN$ ✓

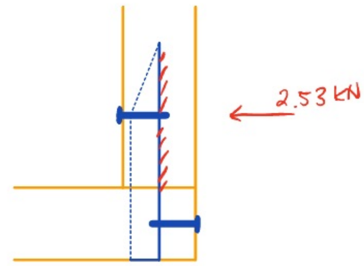
⑤ Bearing (CLT on plate) : $Q_r = \phi F_{cp} A_b K_B K_{Zcp}$

$A_b = 260 \times 305$

$Q_r = (0.8)(5.3 MPa)(1.15)(79300 mm^2)$

$Q_r = 386.7 kN$

$Q_r >> 2.53 kN$ ✓



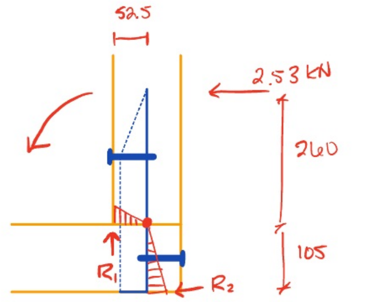
⑥ Bearing (CLT rotation) : $Q_r = \phi F_{cp} A_b K_B K_{Zcp}$

$A_b = 46 \times 305$

$Q_r = (0.8)(5.3 MPa)(1.15)(14030 mm^2)$

$Q_r = 68.4 kN$

$Q_r > 12.65 kN$ ✓



$R_1 = \frac{2.53 kN \times 0.26 m}{0.052 m}$

$R_1 = 12.65 kN$

$R_2 = \frac{2.53 \times 0.26}{0.105}$

$R_2 = 6.26 kN$

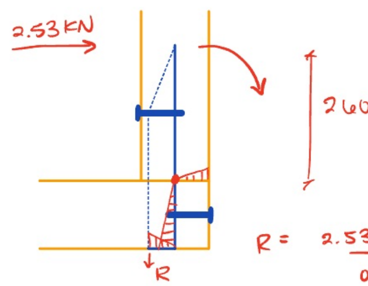
⑦ Bearing (CLT on foot) : $Q_r = \phi F_{cp} A_b K_B K_{Zcp}$

$A_b = 46 \times 305$

$Q_r = (0.8)(5.3 MPa)(1.15)(14030 mm^2)$

$Q_r = 68.4 kN$

$Q_r > 17.6 kN$ ✓



$R = \frac{2.53 kN \times 0.365 m}{0.0525 m}$

$R = 17.6 kN$

⑧ Shear in base : $V_r = \phi F_s \cdot \frac{2}{3} A_g$

$\phi = 0.9$

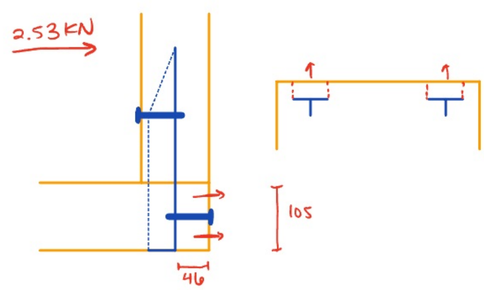
$F_s = f_s (K_D K_H K_{SV} K_T) = 0.5 MPa (1.15)$

$A_g = (48 \times 105)(2)$

$V_r = 0.9 (0.5 MPa)(1.15) \cdot \frac{2}{3} (10080 mm^2)$

$V_r = 3.47 kN$

$V_r > 2.53 kN$ ✓

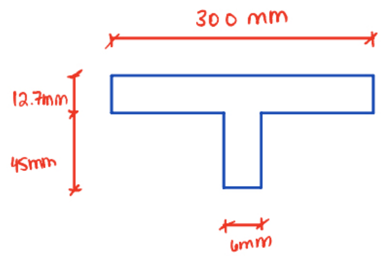


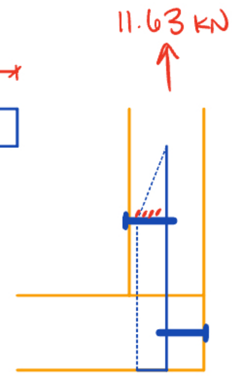
A.4.3 Steel Design Checks

① Plate in tension: $T_r = \phi A_s F_y$

$A_s = (12.7 \times 305) + (6 \times 45)$
$= 4143.5\,mm^2$
$\phi = 0.9$
$F_y = 300\ MPa$

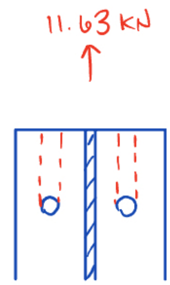
$T_r = 0.9(4143.5\,mm^2)(300\ MPa)$
$T_r = 1118.7\ kN$
$T_r >> 11.63\ kN$ ✓





② Bearing (bolts on plate): $B_r = 3\phi_{br} n t d F_u$

$\phi_{br} = 0.8$
$n = 2$ (# bolts)
$t = 12.7\ mm$
$d = 31.75\ mm$
$F_u = 830\ MPa$



$B_r = 3(0.8)(2)(12.7mm)(31.75mm)(830\ MPa)$
$B_r = 1606.4\ kN$
$B_r >> 11.63\ kN$ ✓

③ Shear failure of bolts: $V_r = 0.6\,\phi_{ar} A_{ar} F_u$

anchor rod (or bolt)

$\phi = 0.8$
$A = \pi\left(\frac{31.75}{2}\right)^2 \times 2$

$F_u = 830\ MPa$

$V_r = 0.6(0.8)(\pi)\left(\frac{31.75}{2}\right)^2(2)(830\,MPa)$

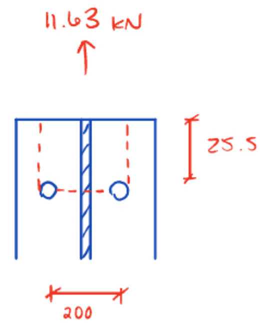
$V_r = 630.85\ kN$
$V_r >> 11.63\ kN$ ✓

④ Block shear : $T_r = \phi_u \left[\underset{\text{tension area}}{U_t A_n F_u} + \underset{\text{shear area}}{0.6 A_{gv} \left(\frac{F_y + F_u}{2} \right)} \right]$

$U_t = 1.0$

$A_n = (200 - 33)(12.7\,mm)$

$A_{gv} = \left[25.5 + \left(\frac{33}{2} \right) \right] (12.7\,mm)(2)$

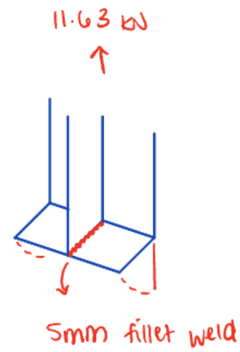


$T_r = 0.75 \left[1.0 (2121\,mm^2)(830\,MPa) + 0.6 (1066.8\,mm^2) \left(\frac{300 + 830}{2} \right) \right]$

$T_r = 875.7\,kN$

$T_r \gg 11.63\,kN$ ✓

⑤ Weld capacity (below stiffener) : $T_r = \underset{\uparrow\ T3\text{-}246}{\phi_w A_n F_u} < \phi A_g F_y$



5mm fillet weld @ 90° ⇒ 1.17 kN/mm × 39.45mm

= 46.16 kN × 2 welds

≫ 11.63 kN ✓

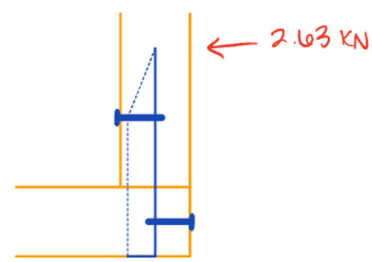
⑥ Bending in stiffener :

→ Section classes (min. t requirements) :

Flange class : $\frac{bel}{t} \leq \frac{200}{\sqrt{F_y}}$ (Table 2)

$\frac{152\,mm}{12.7\,mm} \leq \frac{200}{\sqrt{300}}$

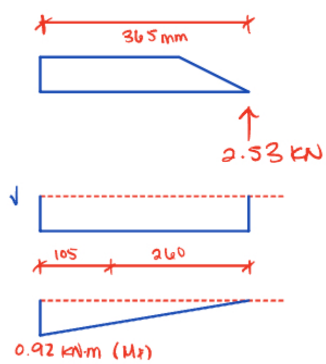
$11.9 \leq 11.55$ → would reduce plate width but for fabrication purposes a 12" (305mm) plate is required.



Fin class : $\frac{bel}{t} \leq \frac{340}{\sqrt{F_y}}$ (Table 2)

$\frac{45\,mm}{6\,mm} \leq \frac{340}{\sqrt{300}}$

$7.5 \leq 19.6$ ✓



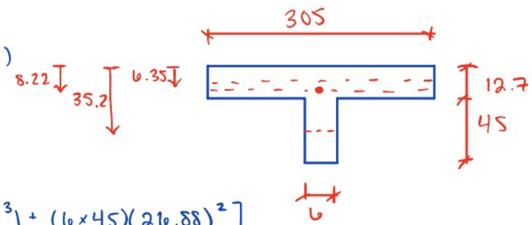
Class 3 ⇒ $M_r = \phi S F_y$

$S = \frac{I}{y}$

$F_y = 300\,MPa$

$$\bar{y} = \frac{\Sigma y'A}{\Sigma A} = \frac{(12.7 \times 305)(6.35) + (6 \times 45)(35.2)}{(12.7 \times 305) + (6 \times 45)}$$

$$\bar{y} = 8.22 \text{ mm}$$



$$I = \Sigma I_x + Ad^2$$

$$= \left[\left(\frac{305 \times 12.7^3}{12}\right) + (260 \times 12.7)(1.97)^2\right] + \left[\left(\frac{6 \times 45^3}{12}\right) + (6 \times 45)(26.88)^2\right]$$

$$= 52063.1 + 12814.7 + 45562.5 + 195084.3$$

$$= 0.305 \times 10^6 \text{ mm}^4$$

$$S = \frac{I}{y} = \frac{0.305 \times 10^6}{49.38} = 6187.2 \text{ mm}^3$$

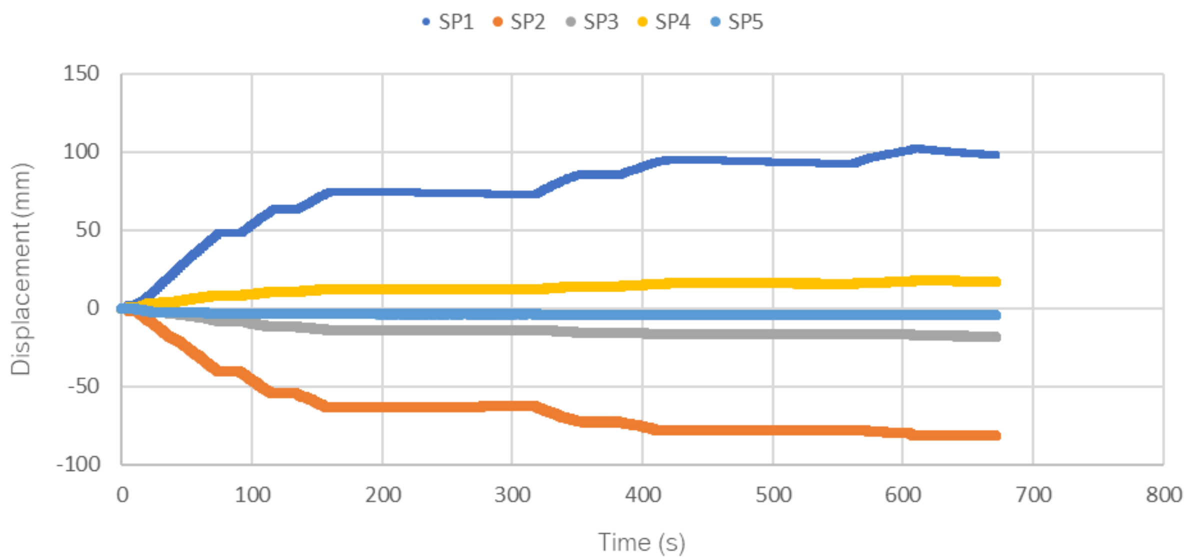
$$M_r = (0.9)(6187.2 \text{ mm}^3)(300 \text{ MPa})$$

$$M_r = 1.67 \text{ kN·m}$$

$$M_r > M_f = 0.92 \text{ kN·m} \checkmark$$

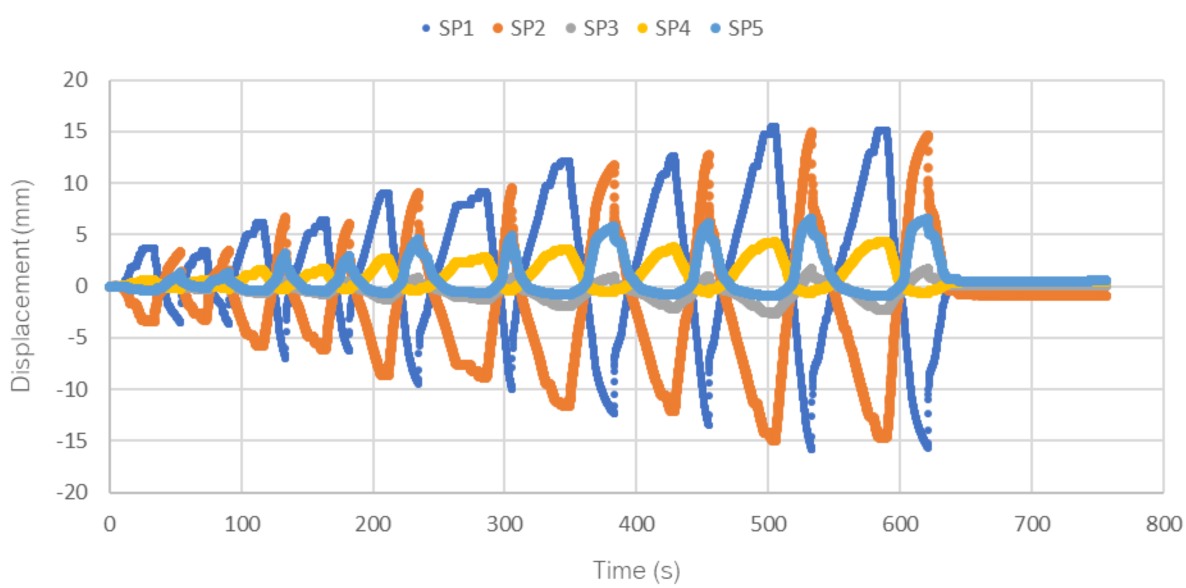
Appendix B: Experimental Results: String Pot (Displacement) Data

B.1 CLT Phase 1

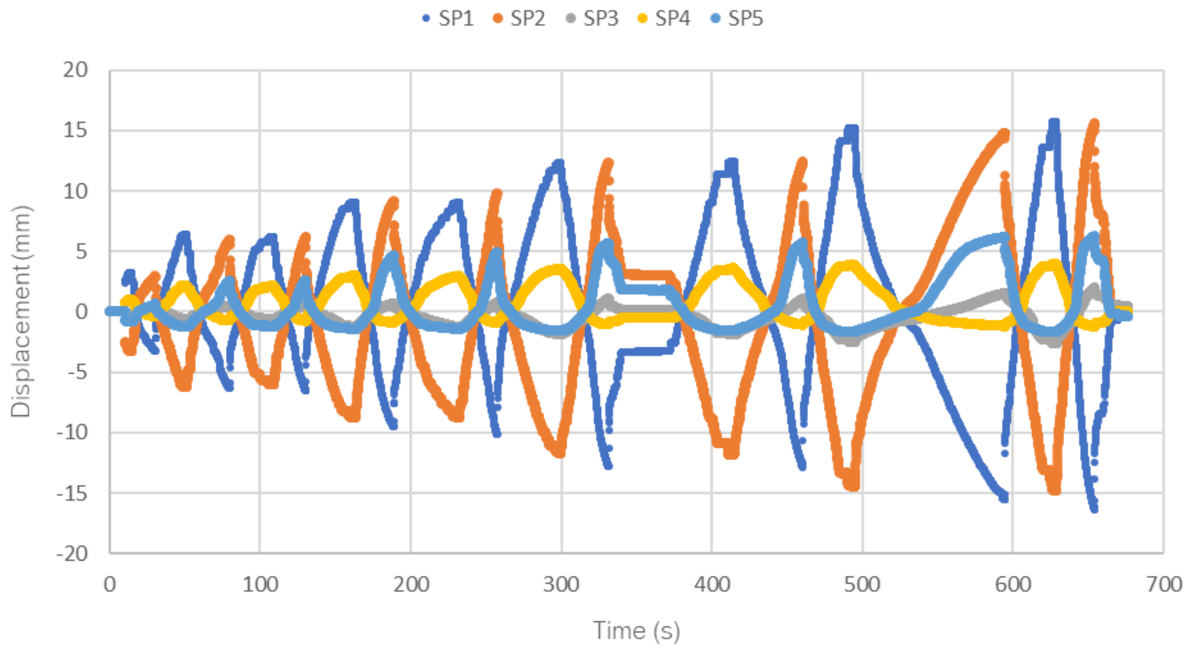


B.2 CLT Phase 2

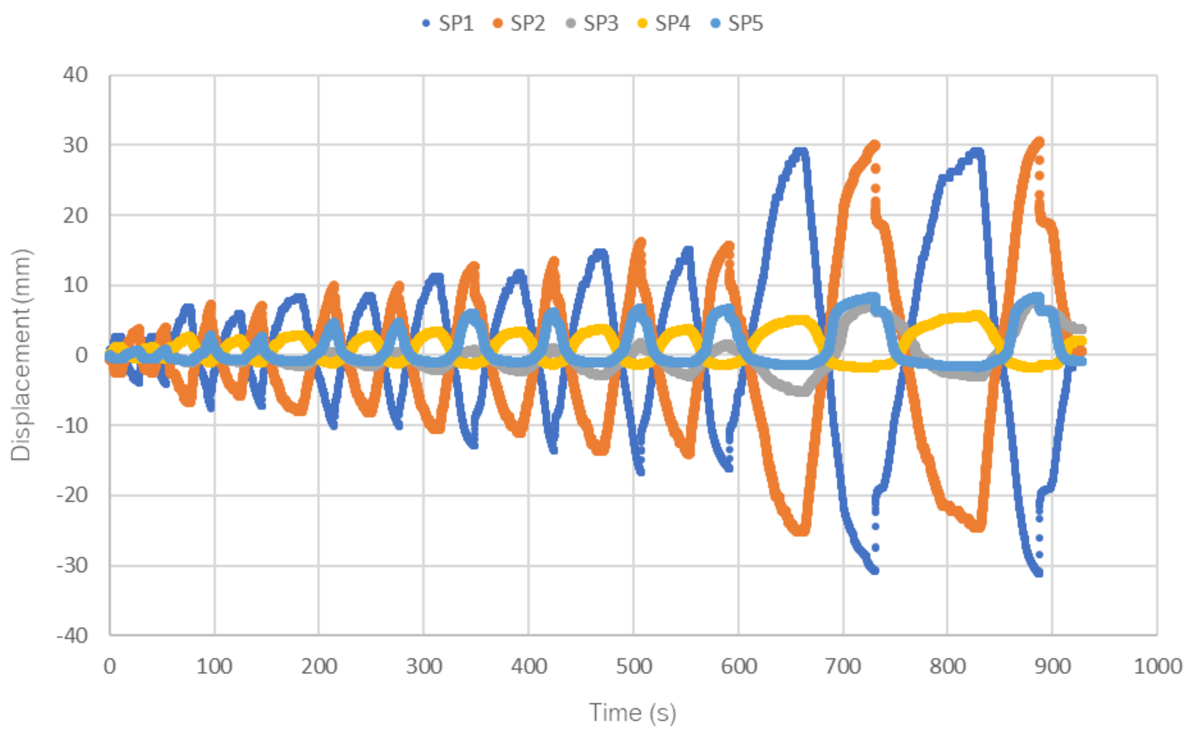
B.2.1 Cyclic Test 1



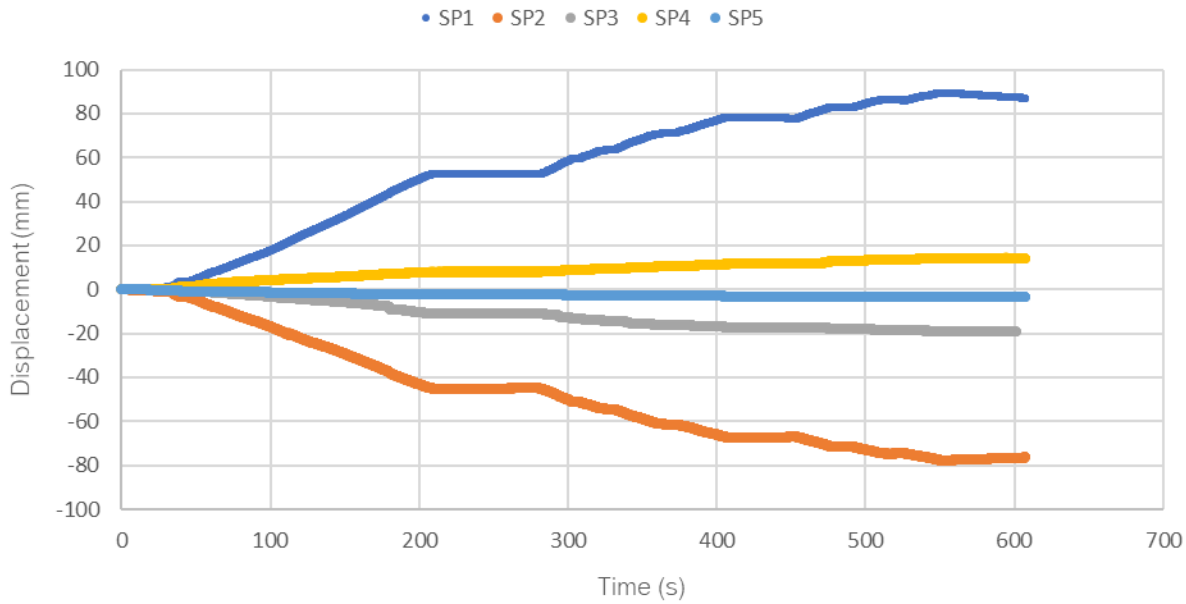
B.2.2 Cyclic Test 2



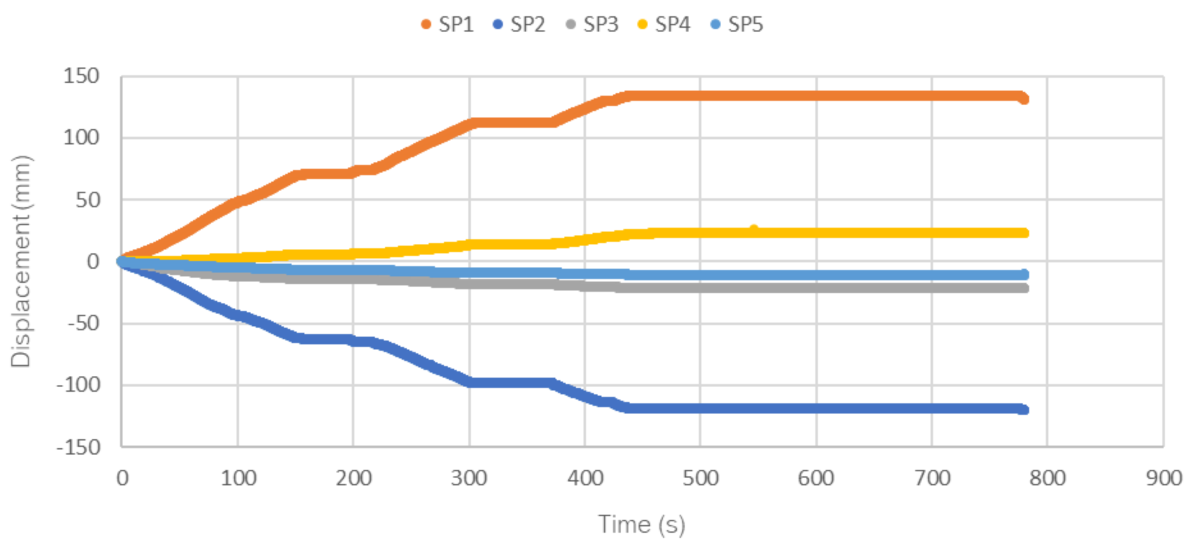
B.2.3 Cyclic Test 3



B.2.4 Monotonic Test

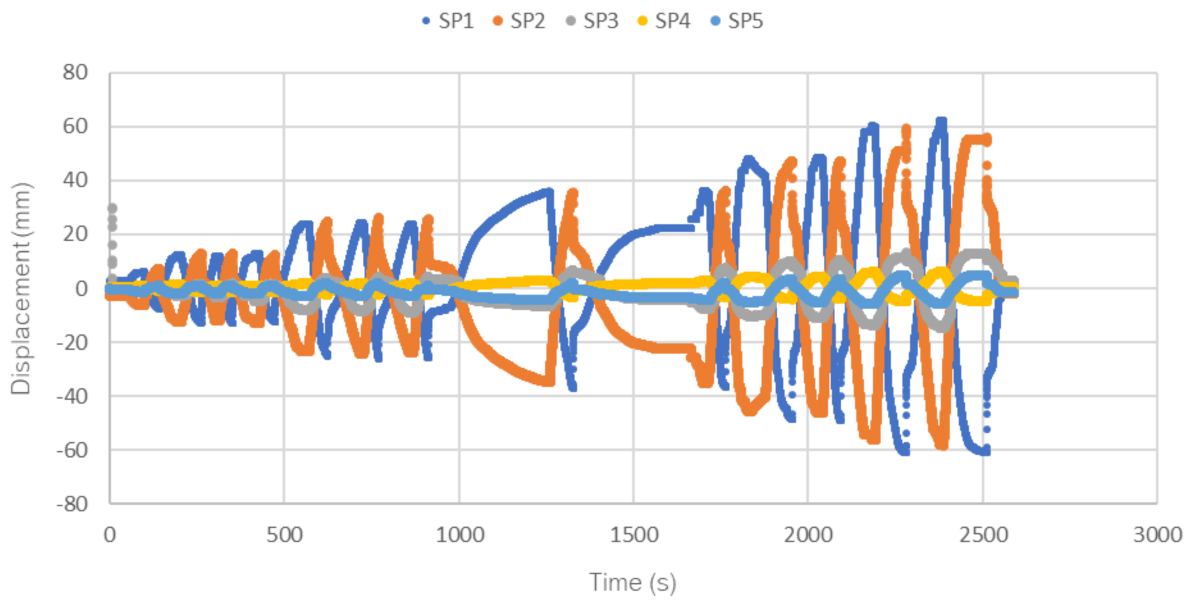


B.3 Light-Frame Phase 1

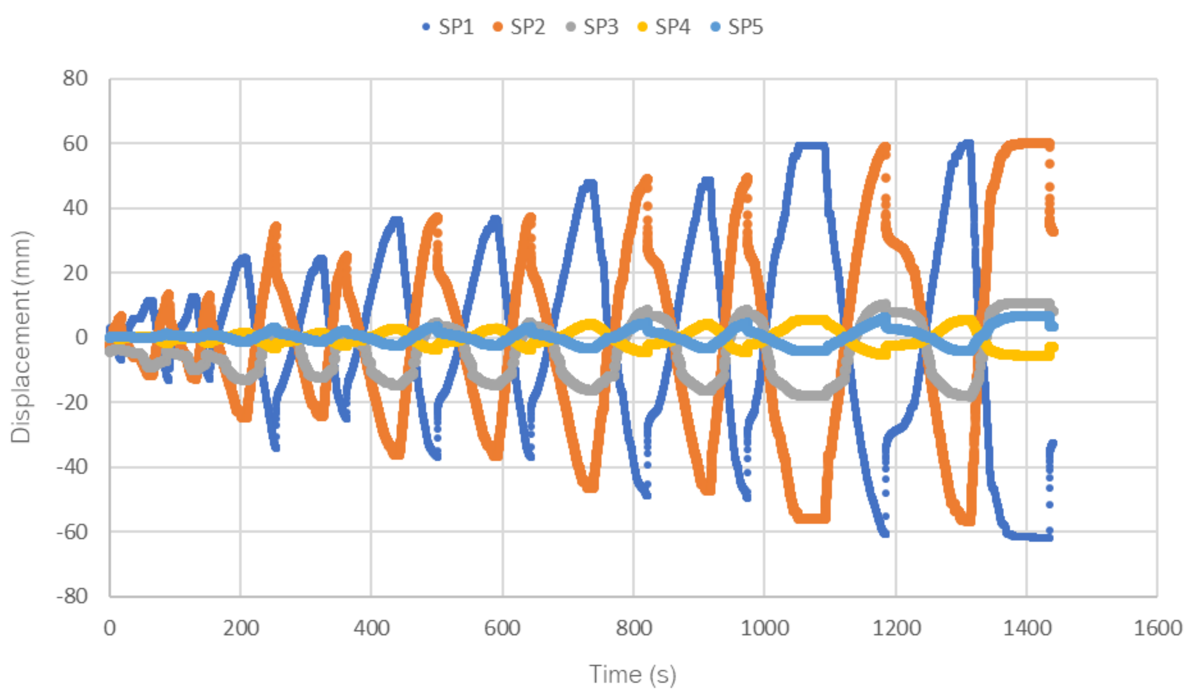


B.4 Light-Frame Phase 2

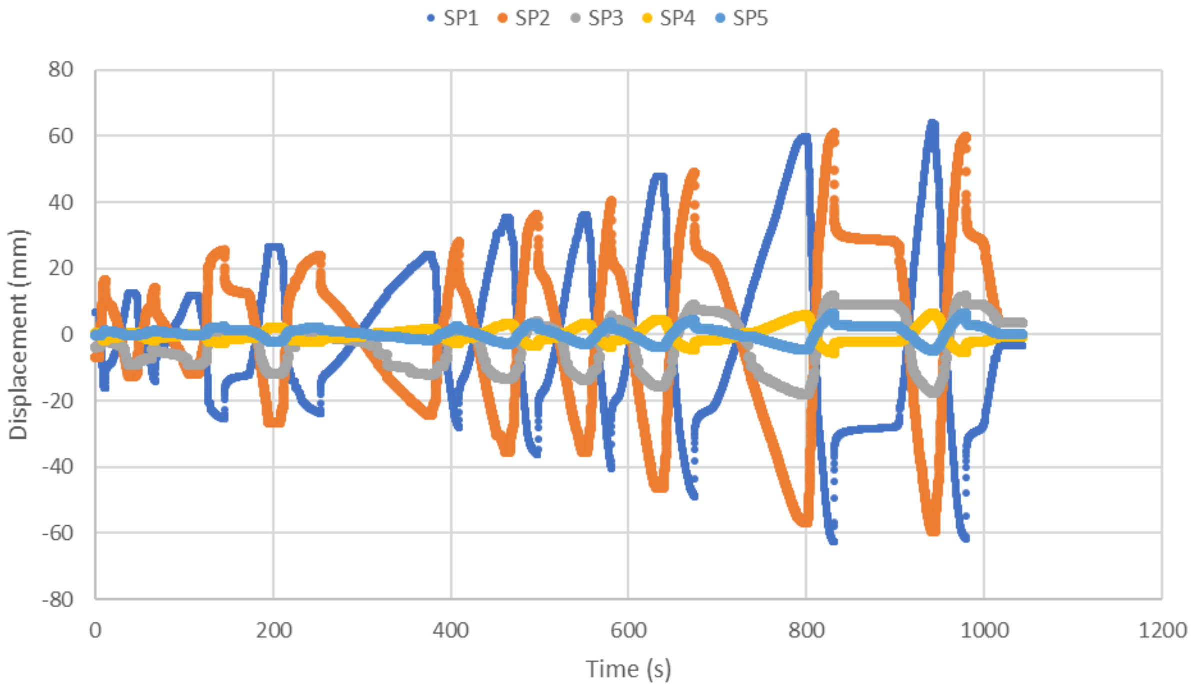
B.4.1 Cyclic Test 1



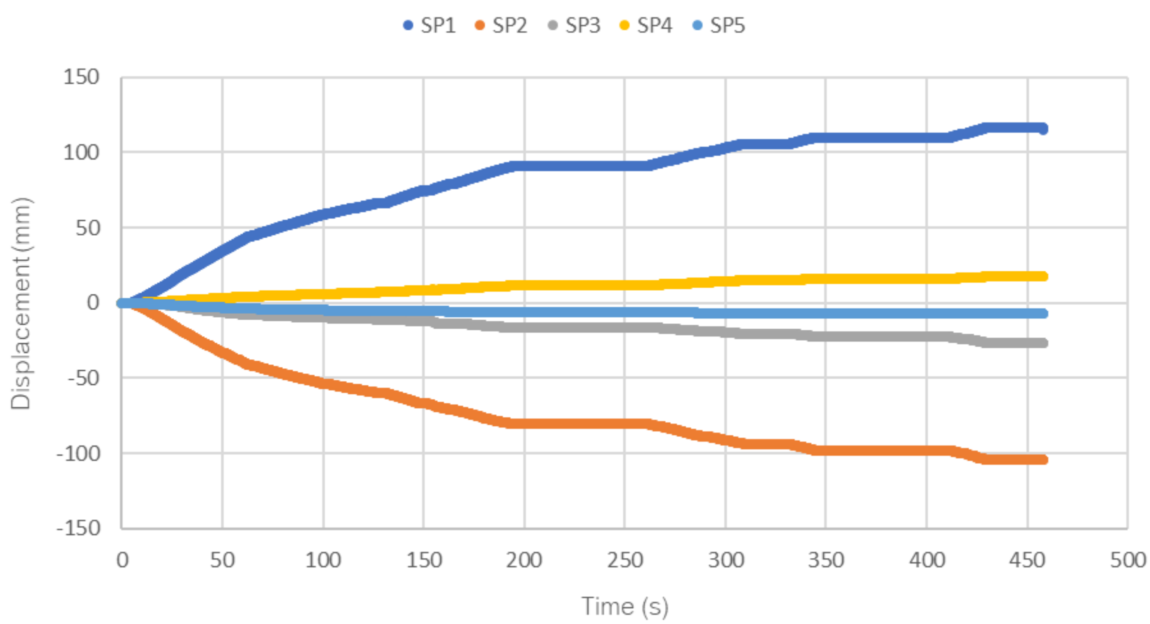
B.4.2 Cyclic Test 2



B.4.3 Cyclic Test 3



B.4.4 Monotonic Test



Appendix C: Life Cycle Assessment

C.1 LCA Tool Carbon Emissions Breakdown

C.1.1 Case 1

Inputs	
Number of Uses	10
Light Frame Limit	2
CLT Limit	5
LF material replacement	10.00%
CLT material replacement	2.50%

			Configuration A		Configuration B		Configuration C	
Use		Life Cycle Stage	LF	CLT	LF	CLT	LF	CLT
1	A1-A3	Construction materials	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
		Cumulative Total:	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
2	A1-A3	Construction materials	125.2853	96.1573	159.0379	119.3706	192.7002	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	817.1386	96.1573	1063.0131	119.3706	1308.7962	142.5839
		Cumulative Total:	2069.9915	3942.4504	2653.3921	4894.1953	3235.7986	5845.9401
3	A1-A3	Construction materials	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
		Cumulative Total:	3322.8445	4038.6078	4243.7710	5013.5659	5162.8009	5988.5240
4	A1-A3	Construction materials	125.2853	96.1573	159.0379	119.3706	192.7002	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000		0.0000	0.0000
		Total:	817.1386	96.1573	1063.0131	119.3706	1308.7962	142.5839
		Cumulative Total:	**4139.9831**	**4134.7651**	5306.7841	5132.9365	6471.5971	6131.1079
5	A1-A3	Construction materials	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	1.0525	-	3.9072	-	1.5869
	C3	EoL - Waste processing	-	0.5005	-	0.6006	-	0.7007
	C4	EoL - Waste disposal	-	982.3430	-	1218.7954	-	1455.2479
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	1080.0533	1590.3790	1342.6738	1927.0024	1600.1194
		Cumulative Total:	5392.8360	5214.8184	6897.1631	6475.6103	8398.5995	7731.2273
6	A1-A3	Construction materials	125.2853	3846.2931	159.0379	4774.8246	192.7002	5703.3562
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	817.1386	3846.2931	1063.0131	4774.8246	1308.7962	5703.3562
		Cumulative Total:	6209.9746	9061.1116	7960.1762	11250.4350	9707.3957	13434.5834
7	A1-A3	Construction materials	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
		Cumulative Total:	7462.8276	9157.2689	9550.5551	11369.8056	11634.3980	13577.1673
8	A1-A3	Construction materials	125.2853	96.1573	159.0379	119.3706	192.7002	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	817.1386	96.1573	1063.0131	119.3706	1308.7962	142.5839
		Cumulative Total:	8279.9661	9253.4262	10613.5682	11489.1762	12943.1942	13719.7512
9	A1-A3	Construction materials	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	96.1573	1590.3790	119.3706	1927.0024	142.5839
		Cumulative Total:	9532.8191	9349.5835	12203.9472	11608.5468	14870.1966	13862.3351
10	A1-A3	Construction materials	125.2853	96.1573	159.0379	119.3706	192.7002	142.5839
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	1.0525	3.2737	3.9072	3.9072	1.5869
	C3	EoL - Waste processing	0.0402	0.5005	0.0505	0.6006	0.0596	0.7007
	C4	EoL - Waste disposal	689.1729	982.3430	900.6510	1218.7954	1112.1291	1455.2479
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	817.1386	1080.0533	1063.0131	1342.6738	1308.7962	1600.1194
		Cumulative Total:	10349.9577	10429.6369	13266.9603	12951.2206	16178.9928	15462.4545

C.1.2 Case 2

Inputs	
Number of Uses	10
Light Frame Limit	3
CLT Limit	6
LF material replacement	20.00%
CLT material replacement	5.00%

			Configuration A		Configuration B		Configuration C	
Use		Life Cycle Stage	LF	CLT	LF	CLT	LF	CLT
1	A1-A3	Construction materials	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
		Cumulative Total:	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
2	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
		Cumulative Total:	1503.4236	4038.6078	1908.4547	5013.5659	2312.4029	5988.5240
3	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	942.4239	192.3147	1222.0510	238.7412	1501.4964	285.1678
		Cumulative Total:	2445.8474	4230.9224	3130.5057	5252.3071	3813.8993	6273.6918
4	A1-A3	Construction materials	1252.8530	192.3147	1590.3790	238.7412	1927.0024	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000		0.0000	0.0000
		Total:	1252.8530	192.3147	1590.3790	238.7412	1927.0024	285.1678
		Cumulative Total:	**3698.7004**	**4423.2371**	4720.8847	5491.0483	5740.9017	6558.8596
5	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
		Cumulative Total:	3949.2710	4615.5517	5038.9605	5729.7896	6126.3021	6844.0274
6	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	1.0525	3.2737	3.9072	3.9072	1.5869
	C3	EoL - Waste processing	0.0402	0.5005	0.0505	0.6006	0.0596	0.7007
	C4	EoL - Waste disposal	689.1729	982.3430	900.6510	1218.7954	1112.1291	1455.2479
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	942.4239	1176.2107	1222.0510	1462.0444	1501.4964	1742.7033
		Cumulative Total:	4891.6948	5791.7624	6261.0115	7191.8340	7627.7985	8586.7307
7	A1-A3	Construction materials	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1252.8530	3846.2931	1590.3790	4774.8246	1927.0024	5703.3562
		Cumulative Total:	6144.5478	9638.0555	7851.3904	11966.6587	9554.8009	14290.0868
8	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	-	-	-	-	-	-
	C3	EoL - Waste processing	-	-	-	-	-	-
	C4	EoL - Waste disposal	-	-	-	-	-	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
		Cumulative Total:	6395.1184	9830.3702	8169.4662	12205.3999	9940.2014	14575.2546
9	A1-A3	Construction materials	250.5706	192.3147	318.0758	238.7412	385.4005	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	-	3.2737	-	3.9072	-
	C3	EoL - Waste processing	0.0402	-	0.0505	-	0.0596	-
	C4	EoL - Waste disposal	689.1729	-	900.6510	-	1112.1291	-
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	942.4239	192.3147	1222.0510	238.7412	1501.4964	285.1678
		Cumulative Total:	7337.5423	10022.6848	9391.5172	12444.1411	11441.6978	14860.4225
10	A1-A3	Construction materials	1252.8530	192.3147	1590.3790	238.7412	1927.0024	285.1678
	A4	Transportation to site	-	-	-	-	-	-
	A5	Construction/installation process	-	-	-	-	-	-
	C1	EoL - Deconstruction/demolition	-	-	-	-	-	-
	C2	EoL - Waste transportation	2.6402	1.0525	3.2737	3.9072	3.9072	1.5869
	C3	EoL - Waste processing	0.0402	0.5005	0.0505	0.6006	0.0596	0.7007
	C4	EoL - Waste disposal	689.1729	982.3430	900.6510	1218.7954	1112.1291	1455.2479
	D	External impacts	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		Total:	1944.7062	1176.2107	2494.3542	1462.0444	3043.0983	1742.7033
		Cumulative Total:	9282.2485	11198.8955	11885.8714	13906.1856	14484.7961	16603.1257

C.2 Global Warming Potential (kg CO2) per Case, Configuration, and Number of Uses

Case 1			Case 2		
Configuration 1			Configuration 1		
# of Uses	LF	CLT	# of Uses	LF	CLT
1	1945	4830	1	1945	4830
2	2070	4926	2	2195	5023
3	4015	5023	3	2446	5215
4	4140	5119	4	4391	5407
5	6085	5215	5	4641	5599
6	6210	10045	6	4892	5792
7	8155	10141	7	6836	10622
8	8280	10237	8	7087	10814
9	10225	10333	9	7338	11007
10	10350	10430	10	9282	11199
Configuration 2			Configuration 2		
# of Uses	LF	CLT	# of Uses	LF	CLT
1	2494	5998	1	2494	5998
2	2653	6117	2	2812	6237
3	5148	6237	3	3131	6476
4	5307	6356	4	5625	6714
5	7801	6476	5	5943	6953
6	7960	12474	6	6261	7192
7	10455	12593	7	8755	13190
8	10614	12712	8	9073	13429
9	13108	12832	9	9392	13667
10	13267	12951	10	11886	13906
Configuration 3			Configuration 3		
# of Uses	LF	CLT	# of Uses	LF	CLT
1	3043	7161	1	3043	7161
2	3236	7303	2	3428	7446
3	6279	7446	3	3814	7731
4	6472	7589	4	6857	8016
5	9515	7731	5	7242	8302
6	9707	14892	6	7628	8587
7	12750	15035	7	10671	15748
8	12943	15177	8	11056	16033
9	15986	15320	9	11442	16318
10	16179	15462	10	14485	16603