Evaluating the Non-linear behaviour of a Timber-Steel Moment Resisting Connections

by

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ABSTRACT

Timber moment resisting connections have gained considerable interest in structural design due to the numerous advantages offered by timber as a lightweight, renewable, sustainable, and aesthetically pleasing material. This research focuses on investigating the feasibility and potential benefits of hybrid timber-steel moment connections in enhancing the seismic performance and ductility of timber structures. The objective is to evaluate the response modification factors of the hybrid timber-steel moment-resisting frames to see if this type of moment connection has the ability to perform like steel moment-resisting frames in lateral loadings. The process by which the studied frames were designed was focused on preventing damage to timber elements by inducing inelastic deformations exclusively in the steel beams, while the remaining parts of the frame retain their elasticity. Nonlinear static analysis is employed to evaluate the force modification factors and nonlinear behavior of the selected structures.

In this study, a total of 18 frames with different span lengths, numbers of stories, and seismicity levels were analyzed to comprehensively investigate their seismic performance. The frames were designed to represent a range of practical configurations commonly found in timber structures. The span lengths of 4, 6, and 8 meters were considered. The number of stories were 2, 4, and 6, and the frames

were located in Montreal, QC, and Vancouver, BC, which are known for having varying seismic conditions. By considering a diverse set of frames, this study tried to provide a comprehensive understanding of the behavior and performance of different timber frame structures under seismic loading, taking into account the effects of span length, number of stories, and regional seismic conditions.

The results of the analysis offer a preliminary understanding of the seismic performance and potential advantages of steel-timber moment connection frames. However, it should be noted that further research is needed to conduct full-scale experimental tests to validate the proposed connections and gather more accurate data. The findings from this study have the potential to contribute to the development of new seismic provisions for moment connection timber frame systems, advancing the field of timber structural design and offering potential design schemes that increase ductility and performance in timber moment resisting connections.

ACKNOWLEDGEMENTS

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1. Introduction

1.1 General Introduction

The use of timber moment resisting connections in structural design has garnered interest due to the advantages of timber as a light, renewable, sustainable, and aesthetically pleasing material. Timber moment resisting frames consist of vertical and horizontal timber members, connected by moment-resisting connections with sufficient strength, stiffness and ductility to resist lateral forces, such as those caused by wind or earthquakes. Timber moment resisting frames are commonly used in low-to-medium rise buildings as alternative to steel and concrete frames where spaces are required to be unobstructed by shearwalls and braced frames. Figure 1-1 provides examples of typically ways moment resisting connections have been achieved. These examples primarily aim to engage wood in tension and compression parallel to grain in order to develop the moment in the joint.

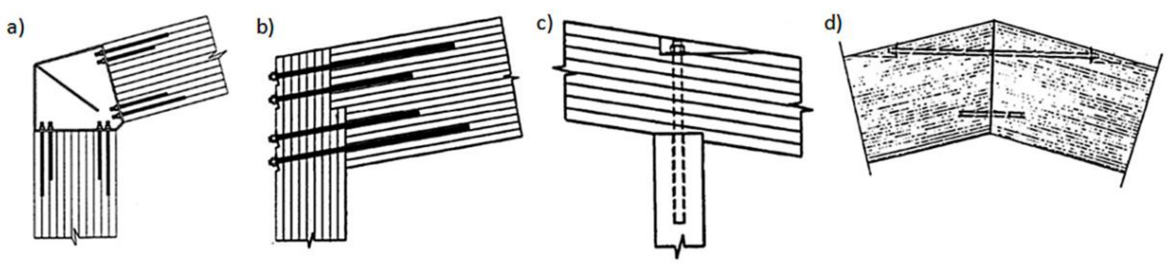

Figure 1- 1 (a) Connection with stiff corner and bar insets; (b) Frames connection with glued in rods; (c) Beam to column connection; (d) Ridge connection with glued rods (Lesko, 2016)

Timber moment resisting frames rely on various types of moment connections to provide stability and resist lateral forces. One of the most common types of moment connections used in timber frames is the beam-to-column connection, which involves connecting the end of a horizontal timber beam to a vertical timber column. This connection is typically achieved through the use of bolts and steel plates. Another type of moment connection used in timber frames is the connection with stiff corner and bar insets. This type of connection involves reinforcing the corners of the frame with additional timber members, which are connected using bolts or steel plates. Bar insets are also used to increase the stiffness of the connection, preventing the frame from collapsing under lateral loads.

Frame connection with glued-in rods is another type of moment connection, which involves inserting steel rods or timber dowels into pre-drilled holes in the timber members and bonding them in place with epoxy or other adhesives. This type of connection provides high levels of stiffness and strength, making it suitable for use in taller buildings. Ridge connection with glued rods is commonly used in roof ridge beams. This connection involves gluing steel rods or timber dowels into the end grain of the timber beam, which provides significant resistance to bending moments.

The issue commonly encountered with the aforementioned types of moment connections is that they may not provide a high enough level of moment connection, or the ductility of the system may not be sufficiently high. Other innovative moment connections that rely primarily on steel sections while over-designing the timber component has shown positive performance especially pertaining to the ductility of the system. The current project investigates such hybrid connection, which combines timber and steel, with the primary aim to prevent damage to brittle timber elements that may undermine the seismic performance of the building. The frame resists lateral loads in a manner that induces all inelastic deformations exclusively in the steel links situated at both ends of the beams, while the other parts of the frame retain their elasticity. An example of such system is depicted in Figure 1-2. The behaviour of this connection type will be investigated in this study in order to provide guidance on the link between the connection- and system level ductility and energy dissipation that would allow moment connections frames to be utilized in timber buildings.

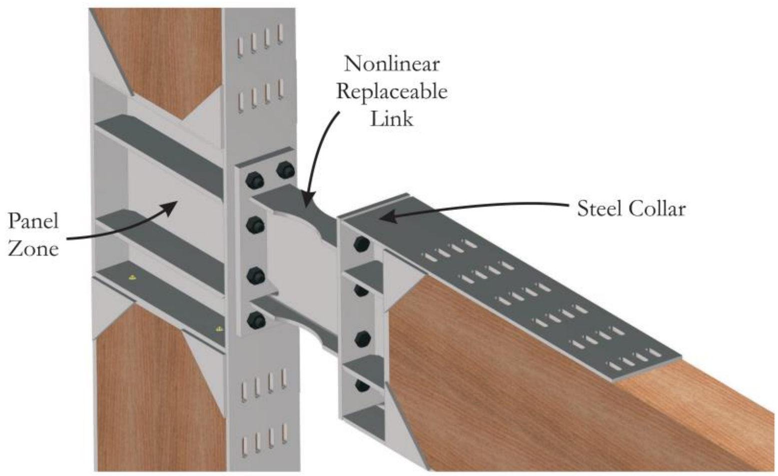

Figure 1- 2 a general view of studied connection (Gohlich, 2015)

As research continues to advance in this area, it is likely that new design schemes will emerge that can further optimize the safety, reliability, ductility, and performance of timber moment resisting connections.

1.2 Research Needs

The lack of design detailing in the timber design standard, coupled with lower force modification factors found in the national building code compared to other structural systems, highlights the need for more comprehensive studies on timber moment resisting connections. To address these needs and develop a structural system that exhibits better seismic behavior than existing systems, the research in the current study aims to investigate the feasibility of using an experimentally investigated timber-steel moment connection system. The work involved in this study can be considered as a first step towards complete investigation of the behaviour of moment connection systems involving energy dissipation exclusively in a steel component designed to yield and dissipate the required energy.

Previous research has demonstrated the potential of hybrid timber-steel connections to improve the hysteretic behavior of timber systems. Study by Gohlich et al. (2015) have explored the use of hybrid timber-steel connections on small-scale specimens. Despite advancements in this field, the work has primarily focused on the behaviour of the connections experimentally and further investigation is needed to develop connections for timber frames that can improve seismic performance and offer better force modification factors than current systems. The current study builds upon previous research by focusing on the force modification factors of hybrid timber-steel connections that can offer both high strength and ductility for seismic applications.

The potential advantages of these types of connections includes favorable nonlinear behavior and sequence of yielding and failure modes that could lead to a significant improvement in the seismic performance of timber structures. With further research and thorough evaluation of this system, it could become a reliable seismic moment-resisting connection with established behavior and performance, comparable to that of steel moment-resisting frames.

1.3 Research Objectives

The overarching goal of this study is to investigate the nonlinear behaviour and evaluate the force modification factors of steel-timber moment connection frames. This is done through a series of nonlinear static (pushover) analyses conducted on different archetypes with varying numbers of stories, span lengths and numbers, and located in different seismic zones. It is anticipated that the results of the nonlinear static analysis and the force-displacement curves generated from these analyses will demonstrate favorable ductility and overstrength behavior, which could provide a seismic performance that is superior to frame systems with traditional moment-connection frames (see Fig. 1-1) and approaching those of steel moment-resisting connections.

1.4 Methodology

To comprehensively evaluate the seismic performance of a structure, it is necessary to investigate its nonlinear behavior and ductility. The nonlinear behavior of a structure describes its response under loading conditions beyond the linear elastic range, and ductility is a measure of the structure's ability to deform under loads while maintaining its integrity. Therefore, to evaluate the seismic performance of a structure, a set of structures with varying geometrical and seismic characteristics are initially selected to investigate the nonlinear behavior and ductility of the system under consideration. The selection of these structures was based on the requirements of the Federal Emergency Management Agency (FEMA) P695 (2009) guidelines, which provides a framework for the seismic performance assessment of buildings.

To perform a detailed investigation of the nonlinear behavior and ductility of the timber-steel connection, nonlinear static analysis is conducted on the selected structures. Nonlinear static analysis is a powerful analytical tool to predict the structural response of the system under loading conditions beyond the linear elastic range. This analysis is based on the principles of the static force method, which assumes that the structure behaves in a nonlinear manner but remains in equilibrium at all stages of the loading process. The results of the nonlinear static analysis are used to plot the force-displacement curves, which provide a detailed understanding of the structural response under extreme loading conditions. The response modification factors are then extracted from the force-displacement curves, which enable the structural engineers to assess the seismic performance of the system in terms of strength, deformation capacity, and energy dissipation.

By analyzing the obtained results, the effect of various factors on the nonlinear behavior of the system under study is also investigated. These factors include the geometrical characteristics of the archetypes and the seismicity of the site where the structure is located. Investigating the effect of these factors on the nonlinear behavior of the system enables structural engineers to identify the critical factors that govern the seismic performance of the structure. This information can be used to design new structures with better seismic performance.

1.5 Scope

One of the main limitations of our research on the studied timber-steel moment resisting connection is the lack of experimental verification. While the current study utilized nonlinear static analysis to investigate the feasibility of the proposed connection, experimental tests would be required to validate the accuracy and reliability of the results. Although modeling abilities have been previously verified with laboratory tests, it should be noted that these tests were conducted on a small scale and focused solely on the connection rather than the entire structure. Therefore, future research could focus on conducting full-scale experimental tests on the proposed connection and the entire timber-steel moment-resisting structure to provide more accurate and reliable data.

1.6 Thesis Structure

The structure of this thesis is designed to provide a comprehensive and logical progression of the research study on hybrid timber moment connections. Each chapter serves a specific purpose and contributes to the overall understanding and analysis of the topic. The following is a summary of the thesis structure:

Chapter 1: Introduction In this chapter, an overview of the research topic is presented, highlighting the significance, objectives, and scope of the study. The motivation behind investigating hybrid timber moment connections and the research methodology employed are discussed.

Chapter 2: Literature Review This chapter provides an in-depth review of previous research studies related to hybrid timber moment connections. It encompasses both experimental and theoretical investigations, examining the advancements, challenges, and findings in the field. The literature review serves as a foundation for understanding the existing knowledge and identifying research gaps.

Chapter 3: Selection and Design of Archetypes in Chapter 3, the process of selecting and designing archetypes is described. The methodology outlined in FEMA p695 (2009) is followed to define representative structural systems for evaluation. The key parameters, material selection, and detailing considerations are discussed, along with laboratory and field testing conducted to validate the archetype models.

Chapter 4: Numerical Modeling This chapter focuses on the numerical modeling techniques employed in the research study. The development of analytical models for hybrid timber moment connections is explained, considering the behavior and interaction of timber and steel elements. The modeling approach, assumptions, and validation methods are discussed in detail.

Chapter 5: Results and Discussion Chapter 5 presents the results obtained from the analysis of the hybrid timber moment connections. The findings regarding seismic performance, response modification factors, overstrength, and ductility-related factors are presented and analyzed. The results are discussed in relation to the research objectives and compared with relevant design codes and standards.

Chapter 6: Conclusion The final chapter provides a comprehensive conclusion based on the research findings. It summarizes the main outcomes, highlights the key contributions, and discusses the implications of the study. Recommendations for future research directions and potential applications are also provided.

2. Literature Review

This section provides a review of the research conducted in the field of timber moment resisting connections, as well as hybrid connections. The review encompasses both traditional as well as modern hybrid connections that have been proposed by various researchers.

The review of traditional timber connections will cover historical timber connection systems that have been in use for centuries. It will explore the design principles and limitations of these systems.

The review of modern hybrid connections will delve into the various types of hybrid connections proposed by researchers in recent years. This will include the combination of timber with other materials such as steel, concrete, or FRP composites. The review will examine the design principles, performance, and limitations of these connections in various applications.

2.1 Recent research on Timber Moment Resisting Connections

Gohlich et al. (2015) conducted a study that focused on the development and testing of a novel hybrid timber-steel moment-resisting connection, which can be utilized in mid-rise heavy timber structures. This connection primarily comprises timber members and incorporates a steel yielding link at the beam-column joint. The steel element, which is ductile, replaces the brittle wood connections and enhances the seismic performance of the system. The self-tapping screws are utilized for the steel-to-timber connection to transfer high bending moments while avoiding inelastic behavior. The authors tested four 2/3 scale moment-resisting connection specimens and reported that the connections had a high ductility capacity. The significant plastic rotation was limited to the ductile steel link, while the steel-to-timber connections remained undamaged. Furthermore, the authors conducted two-dimensional nonlinear dynamic time-history analyses of buildings incorporating the new connection. The seismic performance was compared with an equivalent steel-only moment frame, and the results indicated that drifts and accelerations were similar between the two systems, while the hybrid timber system had lower foundation forces. Figure 2-1 depicts the scheme of a test setup for this connection.

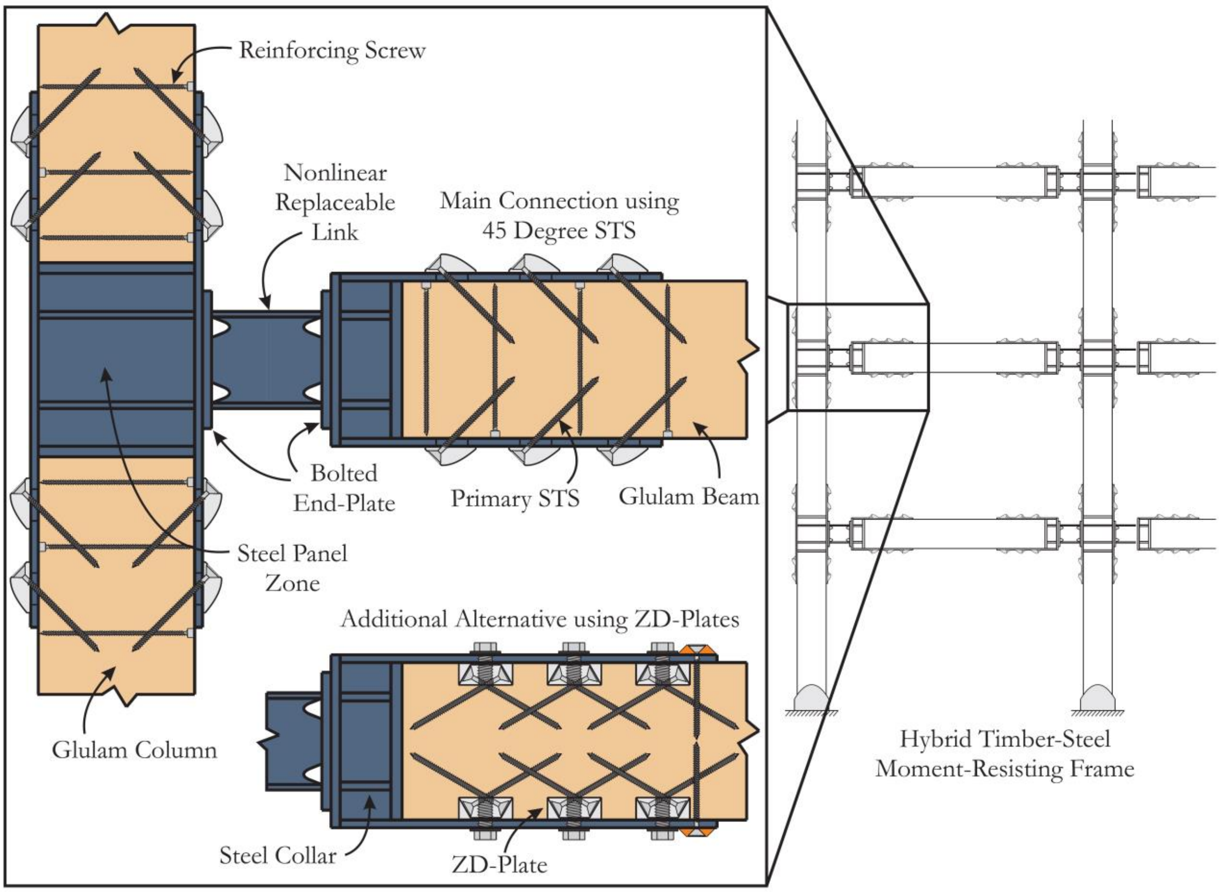

Figure 2- 1 Connection component summary (Gohlich, 2015)

Jeong et al. (2018) studied the moment carrying capacity of sagae connections that are commonly used in timber frame buildings in South Korea to connect a column and two cross beams (Figure 2-2). To achieve this goal, reverse-cyclic loading was applied to measure the load and moment carrying capacities of sagae connections.

The study evaluated yield load, yield moment, and ductility ratio of the sagae connections. In addition, the study employed finite element methods to analyze the stress distribution, failure behavior, and load carrying capacity of sagae connections. The findings of the study revealed that the most critical failure behavior of sagae connections was a split at the reentrant corner of the upper beam. The stress distribution analysis indicated that the highest stress values of combination of longitudinal stress (σ_L) and shear stress (τ_{RT}) was found at the reentrant corner. The study also found that the initial stiffness, yield load, maximum load, yield moment, and maximum moment of sagae connections increased as the size increased. However, yield displacement, maximum displacement, and ductility ratio of sagae connections did not demonstrate a consistent trend with an increment of size. These findings are important for understanding the behavior and design of sagae connections in timber frame buildings.

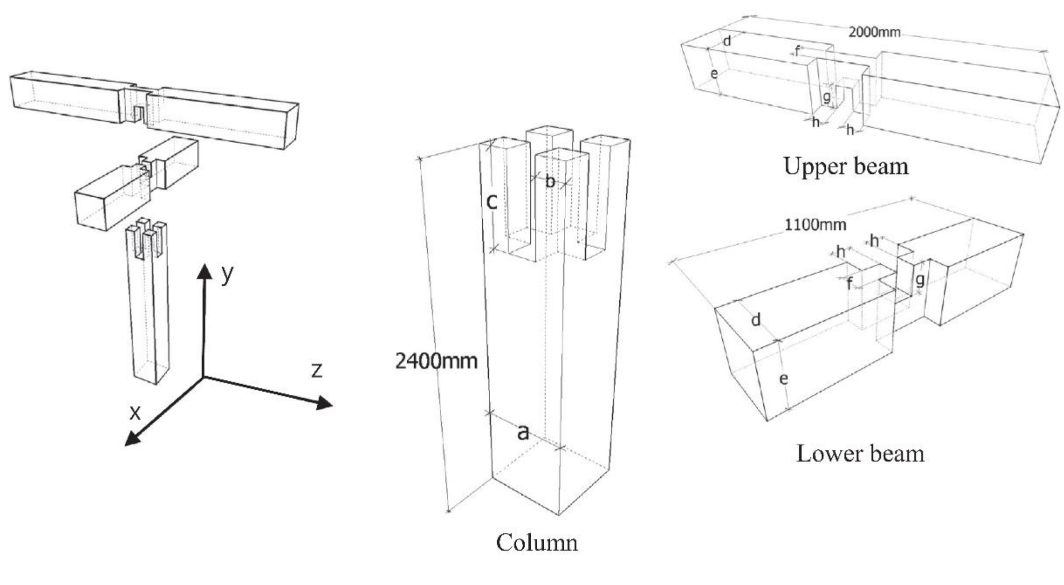

Figure 2- 2 Geometric variables from the mortise and tenon from column and beam used for the sagae joints (Jeong, 2018)

Nouri et al. (2018), investigated the structural behavior of steel–timber composite (STC) beam-to-column connections with shear tabs. Four full-scale STC beam-to-column subassemblies with shear tab connections were fabricated and tested under a monotonically increasing load that produced negative bending moments at the connections. In addition, a steel–concrete composite (SCC) beam-to-column connection with identical geometry to that of the STC joints was fabricated and tested to evaluate the structural performance (stiffness, strength, and ductility) of the STC compared with SCC connections. The main variables in the experimental program were the type of connection (i.e., continuous, spline, and bolted steel plate) between the cross-laminated timber slabs across the column and the depth of the shear tab. The study concluded that the STC beam-to-column connection with steel–CLT spline exhibited a peak load capacity 5% higher than that of identical SCC connections. Also, the ductility and deformability index of all STC beam-to-column connections were higher than those of the identical SCC beam-to-column connections. Figure 2-3 is a schematic outline and cross section of tested cruciform subassemblies.

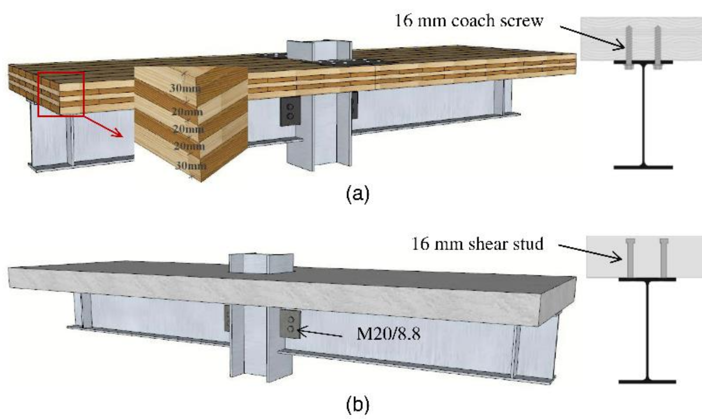

Figure 2- 3 Schematic outline and cross section of tested cruciform subassemblies: (a) STC; (b) SCC (Nouri, 2018).

Keipour et al. (2018) conducted another study on steel-timber composite beam-to-column connections, focusing specifically on the effect of connections between timber slabs (Figure 2-4). In this study, the connection between two prefabricated timber slab panels across the column was identified as a crucial factor influencing the structural performance of a STC beam-to-column connection. To investigate this, the researchers fabricated and tested eight full-scale STC beam-to-steel column cruciform specimens with different connections, including half lap, single and double surface spline with timber and/or steel plate for the timber slabs. The specimens were subjected to a monotonically increasing displacement, and their bending moment capacity, rotation capacity, failure mode, stiffness, and ductility were evaluated and discussed. The results showed that the composite steel-timber system exhibited appreciable ductility and rotation capacity, meeting the existing design requirements for semi-rigid composite connections in Eurocodes EC3 and EC4. Moreover, the negative bending moment capacity of STC connections was found to be significantly higher than that of bare steel connections without a timber slab.

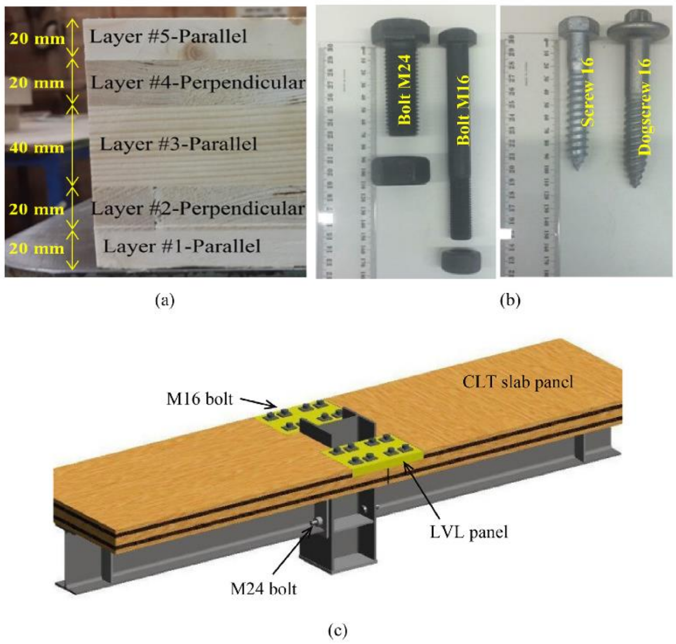

Figure 2- 4 (a) Layout and thickness of lamellae in CLT panels, (b) high strength bolts, coach screws and dogscrews and (c) schematic outline of specimens **(**Keipour, 2018).

The improvement in the rotation capacity of the beam-to-column connection was particularly significant when the CLT-to-CLT connection was constructed by a double surface spline with steel plates. Figure 6 is the layout of the CLT panels and their connections.

Ataei et al. (2019) conducted an experimental study to investigate the structural behavior of steel-timber composite (STC) beam-to-column connections with double-angle web cleats subjected to a negative bending moment (Figure 2-5). The aim of this study was to develop an efficient composite flooring system that is simple, cost-effective, and exhibits superior structural performance. The experimental program involved testing six cruciform subassemblies representing internal beam-to-column connections subjected to negative bending moments. Four of these specimens were STC beams connected to the flanges of a universal column by double web angles. Additionally, a specimen with bare steel beams and another specimen with steel-concrete composite beams were fabricated and tested to provide benchmarks for comparing the structural behavior of STC with bare steel and SCC beam-to-column connections.

The results showed that the STC connections exhibited semi-rigid behavior with negative bending moment and rotation capacities larger than that of SCC connections. The STC connections with double web angles showed enhanced load-carrying capacity and stiffness compared to the bare steel connections. The analytical model proposed in the study, which is an extension of the existing component-based method, was effective in estimating the negative bending moment capacity of the STC connections with double web angles. The use of STC connections with double web angles was found to provide improved structural performance and can be a viable alternative to conventional steel or SCC connections. Figure 2-5 illustrates the specifications of the slab connections between cross-laminated timber (CLT) panels across the columns in the specimens.

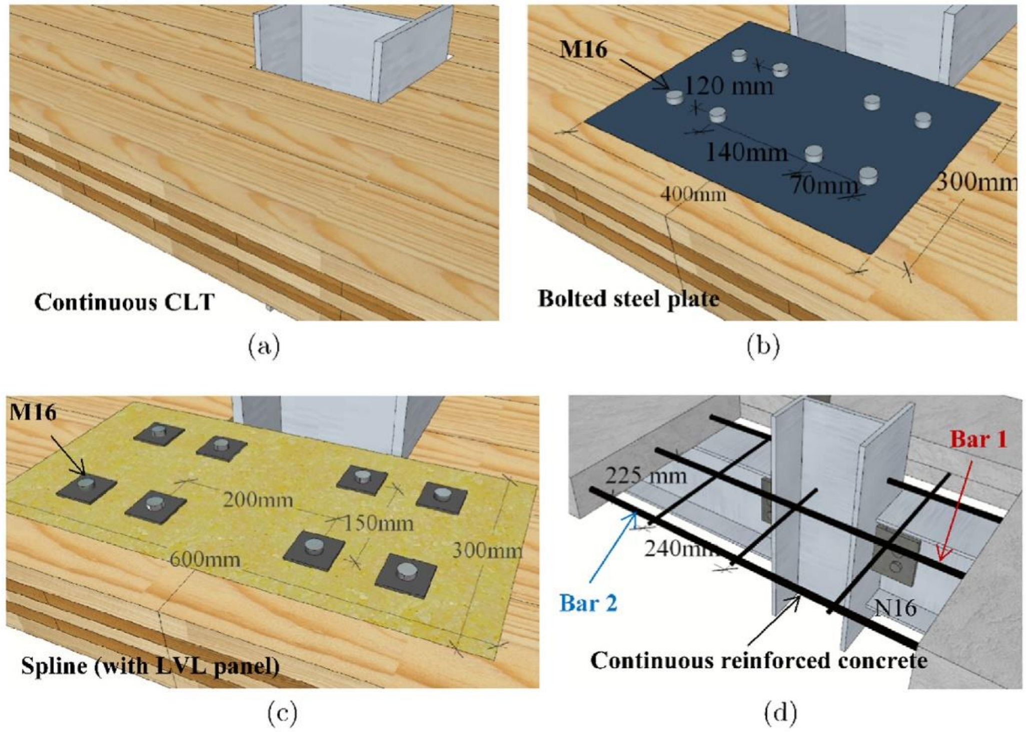

Figure 2- 5 Details of the CLT-to-CLT slab connections across the columns in specimens (a) CJ2, (b) CJ3 and CJ4, (c) CJ5 and (d) CJ6 (Ataei, 2019).

García et al. (2019) highlighted that bamboo has emerged as a promising sustainable material for structural applications, but normalized efficient connections need to be designed to increase its use in construction. Currently, bamboo connections are unable to transmit moment, which limits the material's application in several areas. To address this issue, the authors propose three new beam-column bamboo connections that are capable of transmitting moment. These joints are composed of five thin steel clamps that are tightened around the culms and connected with small steel angles and platens in different configurations (Figure 2-6). The steel clamps offer great versatility and avoid premature failures. On average, the stiffness and strength of these connections were at least 29% and 250% higher than those reported for conventional mortar-injected bolted connections. Cyclic tests of the connections showed hysteresis loops with pinched regions without strength degradation, which is characteristic of timber connections. This study suggests that clamp moment connections are a feasible alternative to improve the structural performance and versatility of bamboo structures.

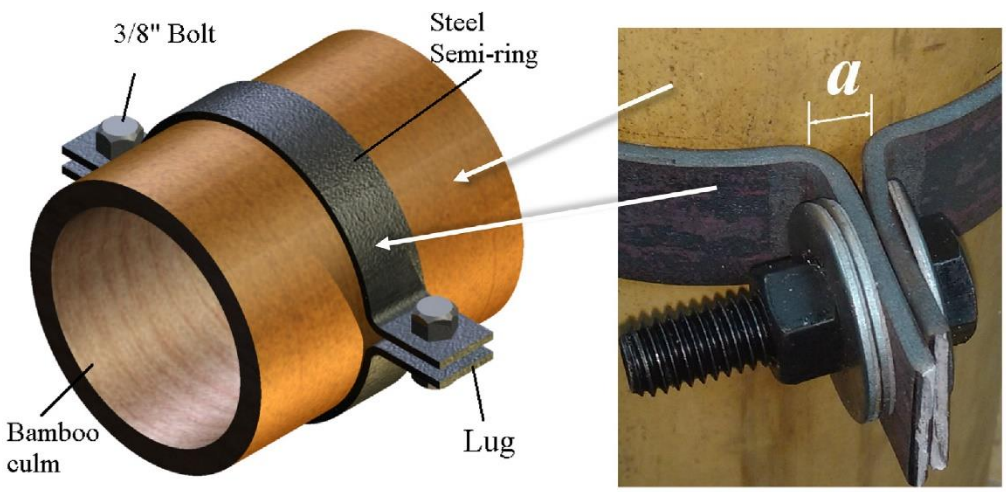

Figure 2- 6 Steel semi-rings fitted to a bamboo culm (García, 2019).

Ataei et al. (2019) investigated the performance of STC beam-to-column joints with extended end plates under negative bending moments (Figure 2-7b). The study involved the testing of one steel and four STC cruciform subassemblies in the laboratory to evaluate the stiffness, flexural resistance, ductility, and failure characteristics of the extended end plate connections. The composite system proposed in the study consisted of CLT panels connected to the top flange of steel girders using coach screws, and steel beams attached to steel columns by bolted extended end plates. Additionally, two CLT slabs subjected to tension were connected using mechanically anchored threaded rods and surface spline joints with steel plates. The results of the experimental tests indicated that the extended end plate STC connection exhibited sufficient rotation capacity to enable plastic analysis of the STC beams. Furthermore, the composite action in conjunction with the continuity of the timber slab increased the bending moment capacity of the connection by more than 50% of that for a pure steel connection. Therefore, the use of extended end plate connections in STC systems can significantly enhance their structural performance, offering a viable solution for timber construction in modern building applications.

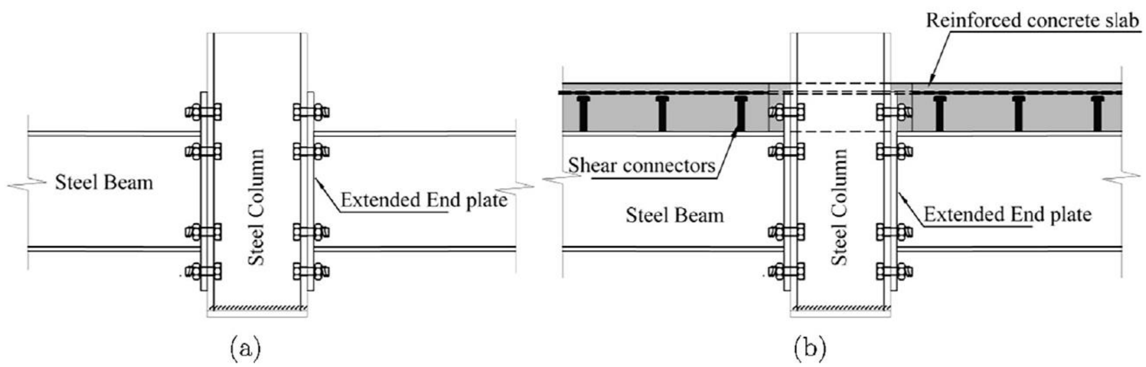

Figure 2- 7 Typical extended end plate joints (a) bare steel, and (b) composite joint (Ataei, 2019).

Jianyang Xue et al. (2020) conducted a study on the moment-rotation behavior of mortise-through-tenon connections in historic timber structures (Figures 2-8 and 2-9). The research focused on investigating the embedded compressive mechanism of these connections under cyclic loading and deriving a simplified equation for predicting the moment-rotation relationship. To validate the analytical model, three 1/3.2-scaled mortise-through-tenon connections were tested under quasi-static loading, including one intact connection and two connections with a gap between the mortise and through-tenon. The validated model was then used to perform a parametric analysis by changing various parameters, such as the gap between the mortise and through-tenon, friction coefficient, and wood material properties. The results showed that the friction coefficient and elastic modulus perpendicular to grain along the radial direction of wood had a significant impact on the moment-rotation relationship of the connections, particularly on the initial stiffness and ultimate moment. However, as the gap between the mortise and through-tenon increased, the initial stiffness and ultimate moment gradually decreased. The findings of this study emphasized the importance of considering the gap between the mortise and through-tenon, friction coefficient, and wood material properties perpendicular to grain in the design of such connections.

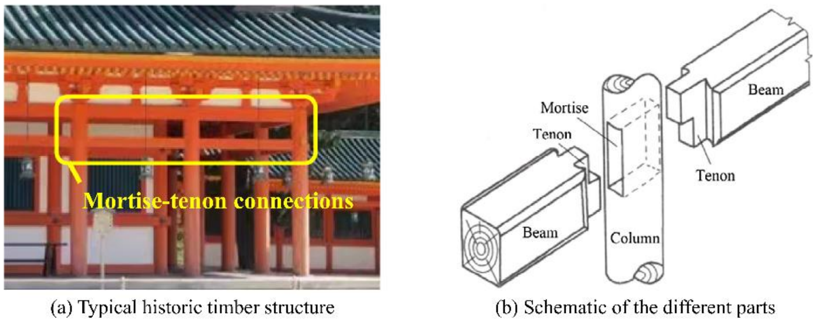

Figure 2- 8 Mortise-tenon connection (Jianyang, 2020)

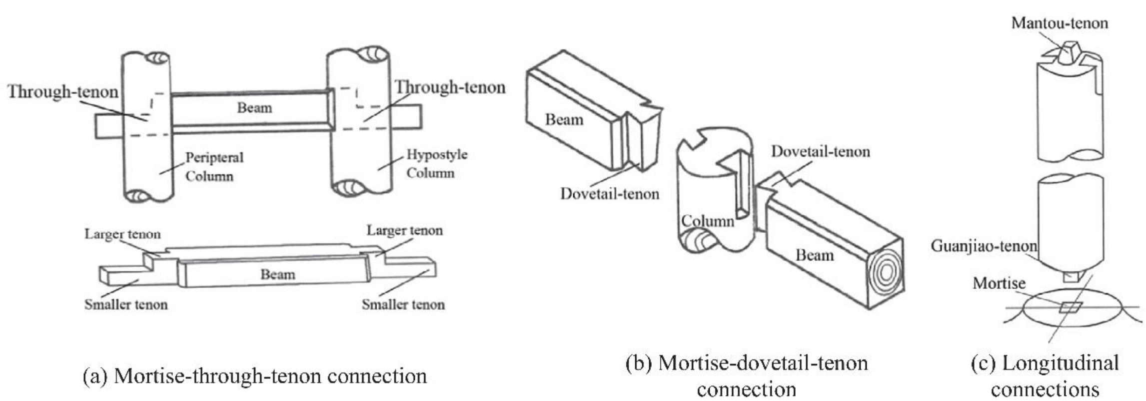

Figure 2- 9 Typical Mortise-tenon connection (Jianyang, 2020)

Polocoser et al. (2018) conducted an investigation into the seismic performance of three-dimensional moment-resisting timber frames equipped with frictional damping in their beam-to-column connections (Figure 2-10). The study introduced an innovative beam-to-column connection design that exhibits rigid behavior at moderate seismic intensities, but initiates frictional dissipating behavior at higher intensities. The researchers conducted cyclic loading tests to assess the energy dissipation of the proposed beam-to-column connection, and then mounted the connection in a three-level prototype frame and tested it on a shake table. The results of the cyclic loading tests indicated that the proposed beam-to-column connection achieved higher energy dissipation than an equivalent rigid connection. The researchers designed the three-dimensional beam-to-column connection to control the intensity of energy dissipation, ensuring that the connection would behave rigidly up to a certain level of seismic intensity, but then transition to frictional dissipating behavior when subjected to higher levels of intensity. The shake table testing of the three-level prototype frame demonstrated the practical applicability of the proposed beam-to-column connection, as well as its effectiveness in mitigating seismic damage.

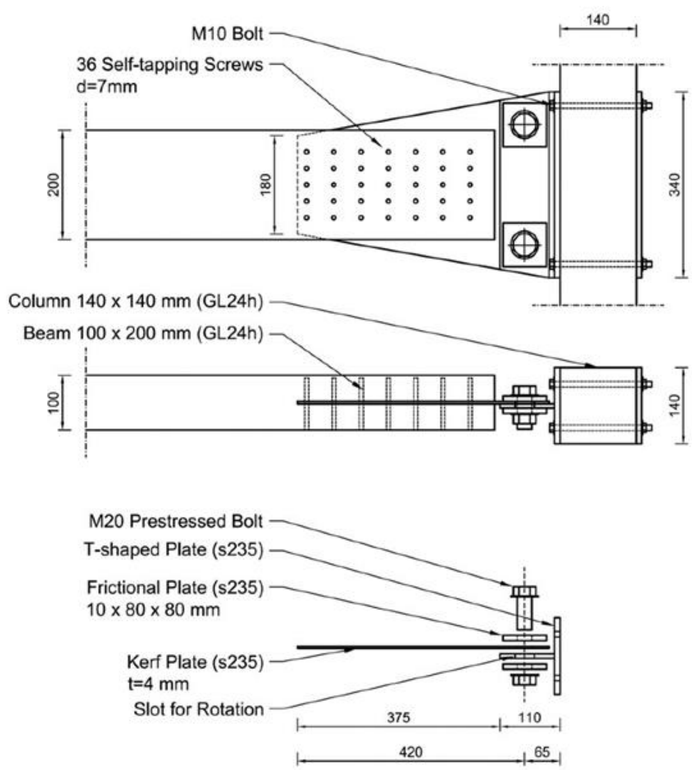

Figure 2- 10 Design of the frictional damping device, the top and side view of the connection to a wooden beam and column (Polocoser, 2018).

Shu et al. (2019) undertook a theoretical evaluation of the moment resistance for bolted timber connections. The researchers systematically tested and analyzed bolted joints with slotted-in steel plates. A typical slotted-in bolted glulam connection was simulated using the software Abaqus, and important design variables were measured and used to numerically estimate the embedment strength, the shear force per plane, and the ultimate moment capacity of the connections. Multiple configurations of joint design were compared, and a parametric design was undertaken. The results of the study showed that the bolted timber connections with slotted-in steel plates had good moment resistance and were able to withstand the imposed forces. The researchers found that the slotted-in steel plates could improve the strength and stiffness of the joints and that increasing the thickness of the steel plate could enhance the moment resistance of the connections. The study provides valuable results into the moment resistance of bolted timber connections and highlights the importance of connection type and material in timber buildings. The results of this research could be used to inform the design and construction of more resilient and sustainable timber structures.

Leonardo G. Rodrigues et al. (2018) investigated the seismic performance of a heavy-timber frame structure with ring-dowelled moment-resisting connections (Figure 2-11). The connections and members of the structure were designed to meet the seismic detailing requirements set by Eurocode 5 and Eurocode 8 for high ductility class structures. The performance of the structure was evaluated using a probabilistic approach that accounted for uncertainties in the mechanical properties of the members and connections. The researchers conducted nonlinear static analyses and multi-record incremental dynamic analyses to characterize the q-factor and develop fragility curves for different damage levels. The study's results indicated that the seismic detailing requirements of Eurocode 5 and Eurocode 8 are sufficient to achieve the required performance. However, the findings also suggest that these requirements could be optimized to achieve more cost-effective connections and members.

The study also highlighted the importance of considering uncertainties in the design and analysis of heavy-timber frame structures to ensure their safety and resilience against seismic events.

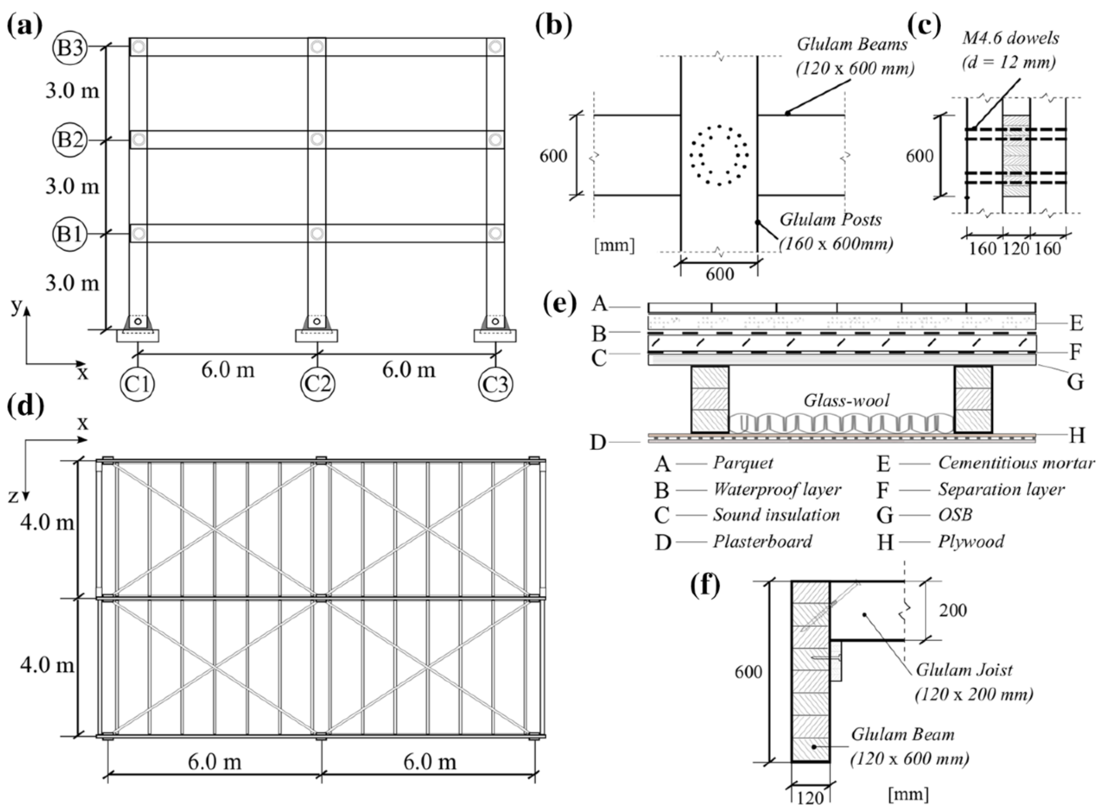

Figure 2- 11 Moment-resisting frame structure: (a) elevation; (b) ring doweled joint-elevation; (c) ring doweled joint-cross section; (d) floor components; (f) joist connection (Rodrigues, 2018).

Valipour et al. (2020) conducted a study on the behavior of steel-timber composite connections with slab continuity steel rods (Figure 2-12). The researchers utilized detailed 3D nonlinear finite element analyses of STC cruciform subassemblies, which included material, geometrical, and contact nonlinearities. The FE models were validated against experimental data, demonstrating that the material models and proposed modeling strategies accurately predicted the failure mode, as well as the local (load-strain) and global (bending moment-rotation) response of the STC connections. Furthermore, the variables influencing the bending moment-rotation response of the STC connections were identified and considered in a parametric study. The data obtained from the parametric study were then utilized to develop a mathematical model for the moment-rotation of the STC joints through non-linear regression. Finally, nonlinear FE analysis was conducted on a multi-story STC frame with nominally pinned connections, and it was demonstrated that the continuity steel rods anchored in the CLT slabs reduced the mid-span deflection of the STC beams by approximately one-third.

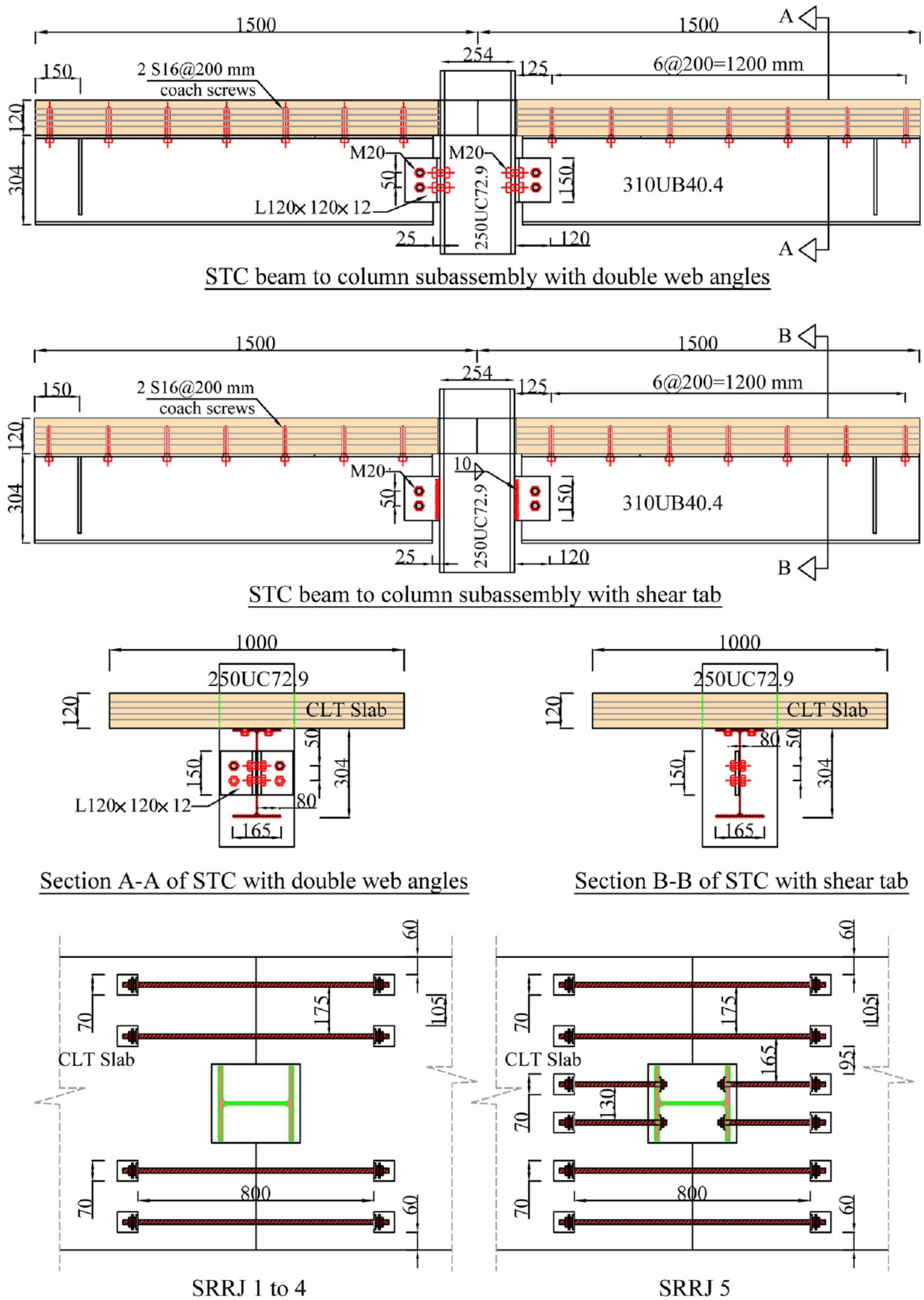

Figure 2- 12 Configuration of tested specimens (Valipour, 2020)

2.2 Response modification factor

Response modification factors (R-factors) are coefficients used in building codes to account for the inherent damping and energy dissipation characteristics of a building's structural system. The purpose of the R-factors is to adjust the seismic forces that a building experiences during an earthquake. They are based on the building's structural system, configuration, and seismic hazard level, and are applied to the calculated seismic loads in order to obtain a design force that is proportional to the building's seismic resistance.

R-factors can vary depending on the type of building and the seismic hazard level of the region in which it is located. In general, buildings with greater resistance to lateral forces and energy dissipation capacity have higher R-factors, indicating that they are better able to withstand seismic events. Conversely, buildings with lower energy dissipation capacity have lower R-factors, indicating that they are more vulnerable to seismic events. As such, R-factors are an important consideration in the design of earthquake-resistant buildings, as they can significantly impact the seismic performance of a structure. Understanding and applying appropriate R-factors are crucial for ensuring that buildings are safe and resilient against seismic events.

Typically, nonlinear behavior is idealized by a bilinear elastic‑perfect plastic relation, as demonstrated in Figure 2-13. The yield force of the structure is shown by V_y, and the yield displacement is Δ_y. In this figure, V_e or V_{max} corresponds to the elastic response strength of the structure. The maximum base shear in an elastic behavior is V_y (Uang, 1991). The ratio of maximum base shear considering elastic behavior V_e to maximum base shear in perfectly elastic behavior V_y is referred to as the force reduction factor (Uang, 1991), as shown in Equation (2-1).

$R_\mu = \frac{V_e}{V_y}$ ……………………………………………………………………………………(2-1)

The overstrength factor is defined as the ratio of maximum base shear in actual behavior V_y to the first significant yield strength in structure V_s,

$R_s = \frac{V_y}{V_s}$ …………………………………………………………………………………..(2-2)

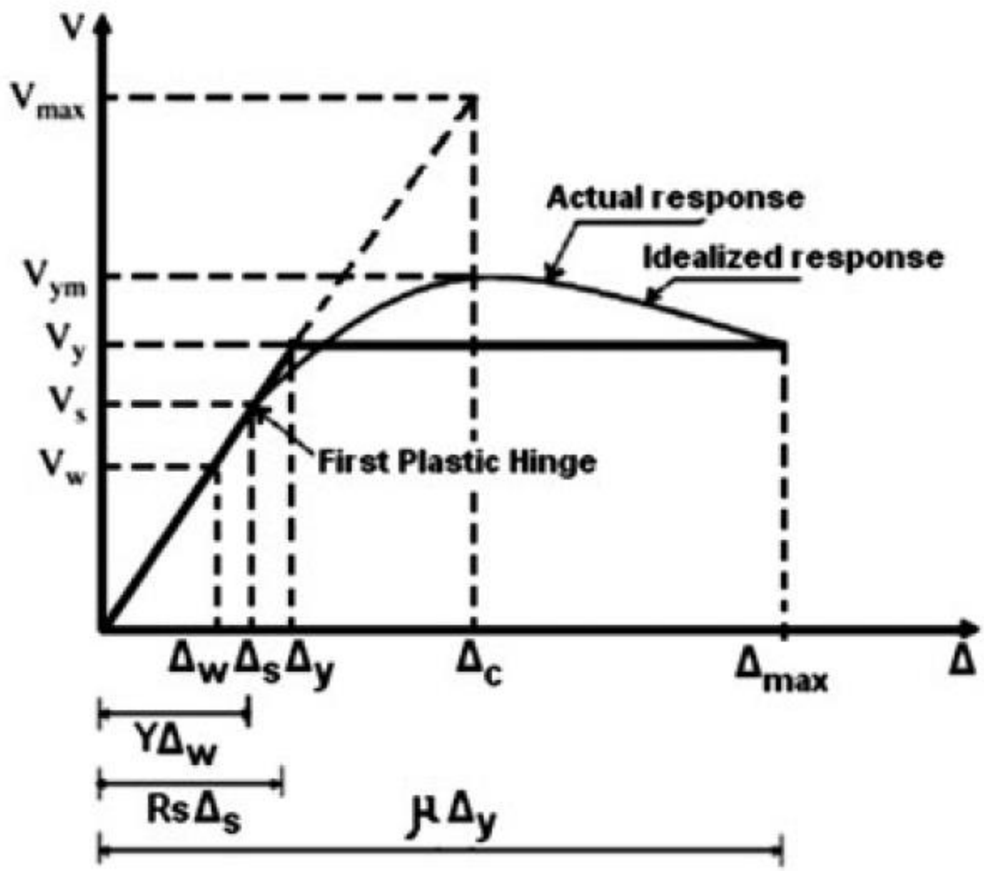

Figure 2- 13 General structure response (Uang, 1991).

To design according to the allowable stress method, the design loads are decreased from V_s to V_w. This decrease is done by allowable stress factor, which is defined as (Asgarian, 2009):

$$Y = \frac{V_s}{V_w} \quad \text{(2-3)}$$

The response modification factor, therefore, accounts for the ductility and over-strength of the structure and the difference in the level of stresses considered in its design. It is generally expressed in the following form, considering the aforementioned conceptions (Asgarian, 2009),

$$R = \frac{V_e}{V_w} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times \frac{V_s}{V_w} = R_\mu \times R_s \times Y \quad \text{(2-4)}$$

The force modification factor related to ductility, R_d, is essentially equivalent to the R factor used in previous editions of the National Building Code of Canada (NBCC, 2015). In NBCC (2015), R_d ranges from 1.0 for brittle systems such as unreinforced masonry to 5.0 for highly ductile systems such as ductile moment-resisting steel frames. This range is deemed realistic for multi-degree-of-freedom structures, as supported by previous studies (Park 1975; Paulay, 1992). The 2001 draft of Eurocode 8 (ECS 1998) provisions for seismic design specify a ductility-related force modification factor, q, varying from 1.0 to 5.0, which further supports the range for the factor R_d. Certain design codes specify higher values of force modification factors than those proposed in the NBCC (2015). For example, the National Earthquake Hazard Reduction Program (NEHRP 1997) provisions prescribe a combined force modification factor, R, as high as 8.0 for the most ductile systems. However, designers are cautioned against using these higher R factors out of context, as they represent more than just the ductility of the system and must be used only in conjunction with the corresponding ground motion design level.

The seismic design force values in previous codes were based on historical levels deemed appropriate. However, the NBCC (2015) represents a significant departure from this approach. It provides site-specific Uniform Hazard Spectra (UHSs) for all locations in the country, which give realistic estimates of the elastic force demand as a function of the period. The ground motions selected represent a relatively rare event with a probability of exceedance of 2% in 50 years (return period of 2500 years). Although structures with a normal importance category would sustain damage during such a severe event, they would not collapse. As a result, the actual capacity of the structure could be fully mobilized, with more ductile structures undergoing significant inelastic action.

Traditionally, structures have been designed such that member resistance factors are equal to or greater than the loads from factor effects. However, research has shown that structures, particularly more ductile ones, can have a considerable strength reserve that is not explicitly considered in the NBCC (2015).

Figure 2-14 depicts the stages of response for a simple-frame structure subjected to increasing lateral loads, from the design factored load, V_1, to the load V_3 that induces a collapse mechanism. The lateral load V_1 corresponds to factored moments M_{bf} and M_{cf} in the beams and columns, respectively. To achieve the actual yield strength of the beams $M_{b,yield}$, a higher load V_2 is required. This increased resistance is because beam sizes are typically larger than required, and actual yield stress is generally higher than the minimum specified yield strength [33].

Adoption of capacity design procedures enables further enhancement of the structure's resistance. For the simple frame illustrated, with the columns fixed at their bases, capacity design requires designing columns such that plastic hinging occurs first in the beams, with the full capacity of the system only being achieved when the columns yield at their bases. To achieve this, the ductile beams must be adequately detailed to maintain their capacity ($M_{b,capacity}$) under significant inelastic deformations without strength degradation, until the column capacities ($M_{b,capacity}$) are reached and a collapse mechanism forms under load V_3 (Mitchell, 2003).

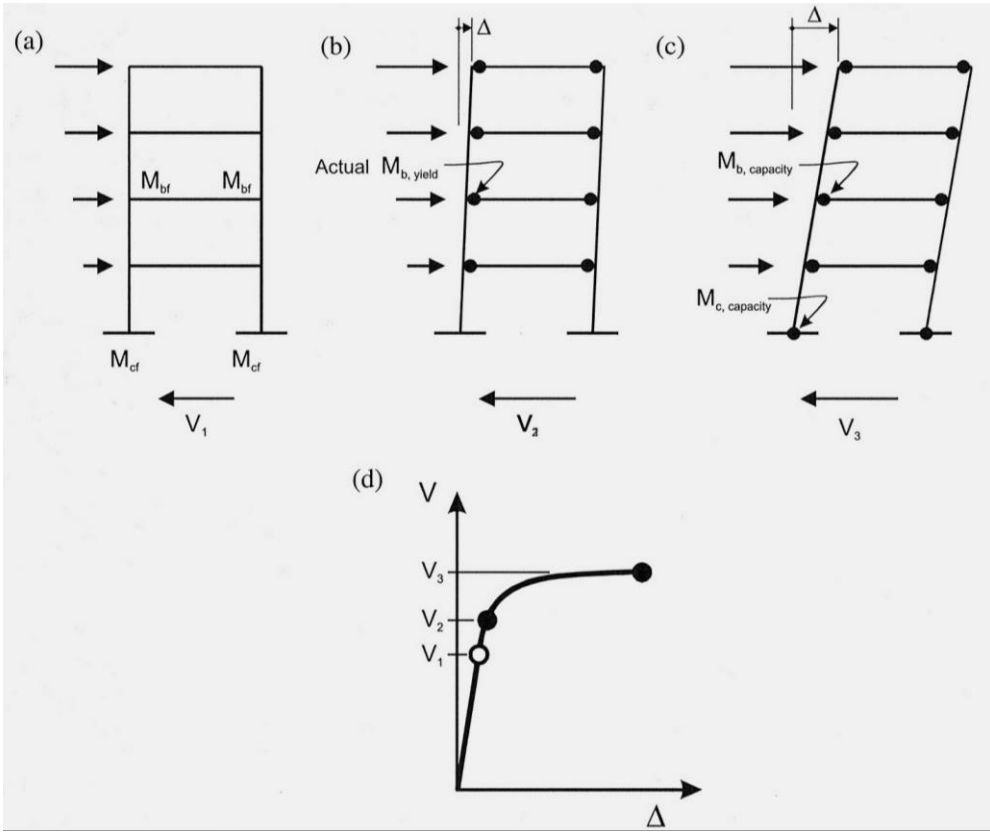

Figure 2- 14 Stages in the response of a frame structure (Mitchell, 2003).

The revisions proposed for the 2005 National Building Code of Canada (NBCC, 2005) include the addition of a force modification factor, R_o, explicitly related to overstrength, which accounts for the reserve of strength in a structure. Instead of increasing the factored resistance to accommodate overstrength, the design force level is reduced by incorporating the R_o factor. This approach aligns with standard design procedures, where factored resistance is compared with load effects obtained through linear analysis. Figure 2-15 illustrates the resulting reduction in design force, V.

For design purposes, only the dependable or minimum overstrength can be utilized for a particular structural system, which arises from the application of design and detailing provisions specified in the appropriate CSA standard. The NBCC (2015) specifies the overstrength factor, R_0, that has been determined uniformly for all systems, conforming to the CSA provisions.

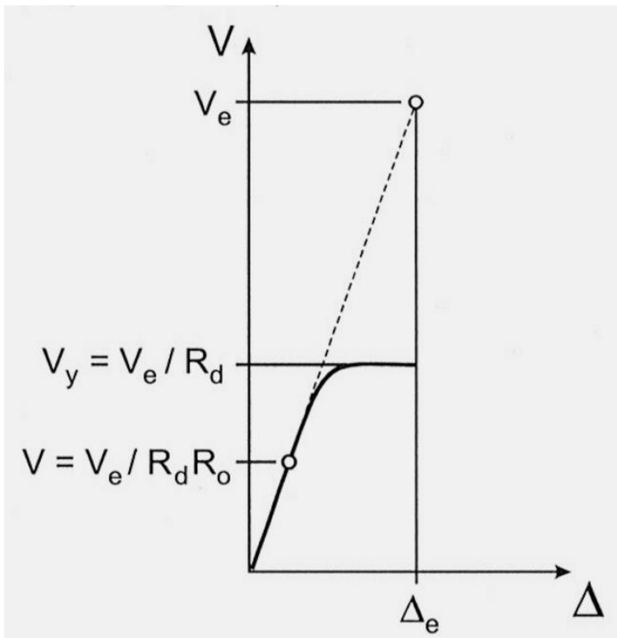

Figure 2- 15 Determination of the lateral design force, V, including ductility- and overstrength-related force modification factors. $\boldsymbol{V_y}$, lateral force at yielding; Δ, roof displacement; $\boldsymbol{\Delta_e}$, roof displacement corresponding to $\boldsymbol{V_e}$ (Mitchell, 2003).

To account for the contribution of various components to the overstrength-related force modification factor, R_o, the following formulation has been adopted.

$$R_o = R_{size} R_{\Phi} R_{yield} R_{sh} R_{mech} \quad \text{(2-5)}$$

The overstrength-related force modification factor, R_0, comprises various components, including R_{size}, which accounts for the overstrength resulting from restricted choices for member and element sizes, rounding of sizes, and negative bending. However, the R_{mech} factor generally experiences a similar decrease as building height increases (Mitchell, 2003)..

Figure 2-16 depicts the mechanisms that can develop in other structural systems. In tension-compression concentrically braced steel frames, overstrength arises once the compression brace buckles, requiring additional force to induce yielding in the tension brace (Tremblay 2001 [34]). In ductile steel plate wall systems, yielding occurs first in the plate, with the full mechanism forming only after plastic hinging arises in the more pliable surrounding steel frame. In ductile reinforced concrete coupled walls, yielding initially develops in the coupling beams, followed by flexural yielding at the base of the walls.

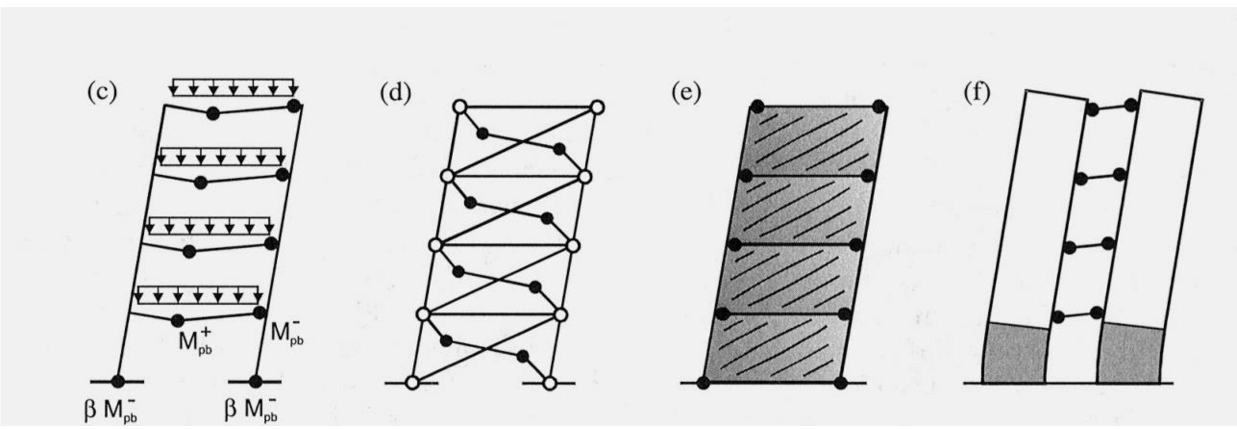

Figure 2- 16 Collapse mechanism for some structural systems (Mitchell, 2003).

2.3 Summary

Recent studies in timber frame connections have primarily focused on improving existing connections through the use of composite connections and minor layout adjustments. While these efforts have shown promise in enhancing the ductility and strength of timber connections, they have not yet resulted in the development of an economically feasible and reliable connection for mid-rise moment-resisting timber frames.

The work done at Carleton University offers a promising solution to this issue, which opens up further opportunities for research into the development and optimization of such systems. By incorporating this type of connection in timber frames, higher modification factors can be utilized for seismic design, leading to a more reliable moment frame.

The proposed hybrid connection could have the potential to provide solutions for both newly constructed structures and the retrofitting of existing ones. Moreover, this type of connection offers relatively good aesthetic properties. However, for reaching a level of ductility and overstrength factor, structural systems must be designed and more importantly detailed to reach those levels of resilience. Thus, for a design procedure, further investigation into this area of connection is required to clarify any ambiguities and enhance the reliability of timber frame connections.

3. Selection and Design of Archetypes

This section outlines the methodology employed in designing the archetype buildings containing the frames examined in this study, which are assumed to be located in Montreal, QC, and Vancouver, BC. The seismic analysis of these frames was performed using the equivalent static method, outlined in the NBCC (2015). The frames were modeled using ETABS 2016.2.1 software and analyzed based on NBCC (2015) load combinations (Figure 3-1). The resulting analysis data was imported into an Excel sheet to facilitate frame design using CSA O86-19. Subsequently, 2D-frames were extracted from the 3D-models and analyzed using OpenSees to investigate the frames' behavior under lateral loads (Figure 3-2). The design process was iterative, starting with an initial calculation of frame sections based on gravity loads, followed by a capacity design process to identify suitable sections.

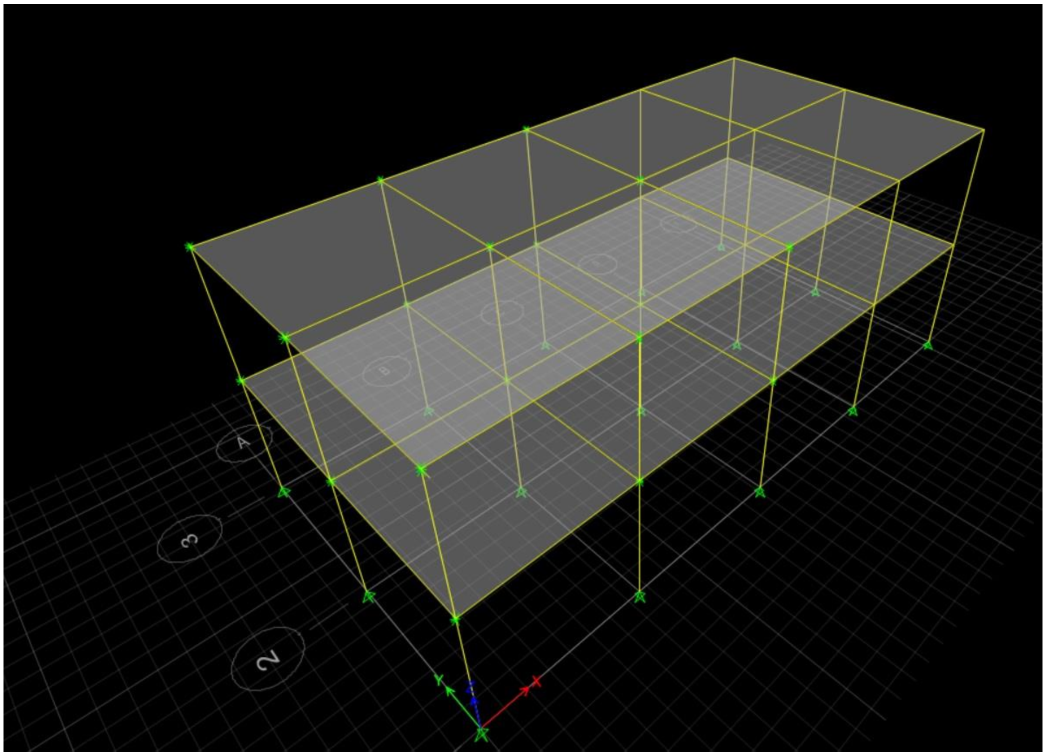

Figure 3- 1 a view of one of the models

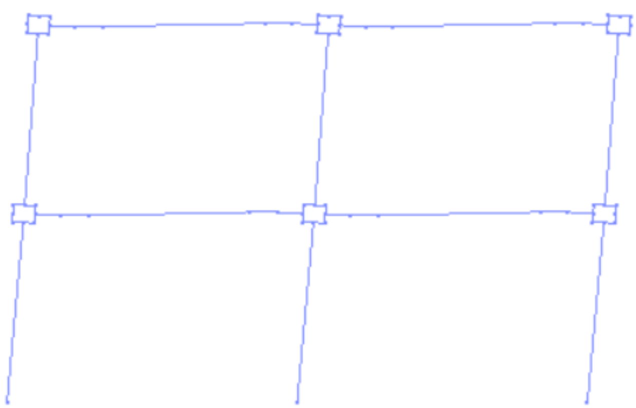

Figure 3- 2 the view of 2D analysis in OpenSees

3.1 Architectural Assumptions

Chapter 4 of FEMA p695 (2009) outlines the process for defining archetype models of new structural systems that are to be evaluated. This involves a path that outlines the steps necessary to develop and validate the archetype models, including defining the key parameters of the structural system, selecting appropriate materials and detailing, and conducting laboratory and field testing to verify the performance of the system. This process is critical in ensuring that the archetype models are representative of the actual structural systems and can be used to accurately predict their behavior under seismic loading.

As per the National Building Code of Canada (2015), the maximum height limit for moderately ductile timber moment-resisting frames is 20 meters (NBCC, 2015). Thus, the present study selected frames with 2, 4, and 6 stories and a 3.3-meter story height to analyze the behavior of various structures within the prescribed height limit. The objective of the study was to gain insight into the performance of different timber frame structures under seismic loading and to ensure compliance with the NBCC (2015) guidelines.

The seismic behavior of timber structures is affected by various factors, including span length of frames and the number of spans. In the present study, different span lengths (4, 6, and 8 meters) and number of spans (2 and 4) were considered in the archetype models to investigate their impact on seismic behavior. Recent research by Guo et al. (2021) suggests that increasing the span length of structures can increase their flexibility and result in greater lateral movement during an earthquake. These findings support and highlight the need to consider the effects of span length in the seismic design of longer span timber structures.

The location of a structure plays a crucial role in its seismic behavior. Therefore, in this study, the frames were located in Montreal, QC, and Vancouver, BC, which are known for having different seismic conditions. These studies have identified that Vancouver, BC is located close to the Cascadia Subduction Zone, which is considered one of the most seismically hazardous regions in North America (Petersen et al., 2014). Similarly, Montreal, QC is located near the Charlevoix Seismic Zone, which is known for having experienced several significant earthquakes in the past (Demets et al., 2010). Therefore, it is crucial to understand the seismic behavior of structures in these regions to develop effective seismic design strategies.

It is anticipated that the findings from this study could be used to develop new seismic provisions for moment connection timber frame systems.

3.2 Gravity Loads

In this study, the building archetypes were designed based on the gravity load values presented in Table 3-1. The details of the roof and floor systems are depicted in the table and the live load values are obtained from the NBC (2015).

Table 3-1-Gravity Loads

Roof	Load (kPa)	Floor	Load (kPa)
Roofing	0.24	Floor Finish	0.1
Insulation	0.1	Partitions	1.0
Fire Proofing	0.1	Fire Proofing	0.1
Ceiling	0.24	Ceiling	0.24
Mesh/Elec	0.48	Mesh/Elec	0.48
CLT Slab	0.623	CLT Slab	0.623
Wood Framing	0.3	Wood Framing	0.3
Total	2.083	Total	2.843
Live Load	1.0	Live Load	2.4

3.3 Equivalent Static Loads

The seismic analysis of the lateral load resisting system was conducted using the equivalent static method, as outlined in the NBCC (2015). The equivalent static method is a simplified approach that assumes the seismic forces acting on a structure can be approximated as static loads.

It is important to note that the accuracy of the equivalent static method relies heavily on accurate estimation of seismic loads, which can be challenging in practice due to the uncertainties in ground motion prediction, soil-structure interaction, and structural response (ASCE/SEI7, 2016). Therefore, it is essential to consider the limitations of the equivalent static method and perform sensitivity analyses to assess the effect of various design parameters on the seismic response of the structure (ACI 318, 2019).

To overcome the limitations of the equivalent static method, other advanced techniques such as response spectrum method and nonlinear dynamic analysis can be employed to capture the dynamic characteristics of the structure and the seismic excitation more accurately (ASCE/SEI 7, 2016. However, these methods are computationally intensive and require more sophisticated modeling and analysis tools, which may not be practical for every design scenario.

While the equivalent static method has its limitations, it is a practical and efficient tool for seismic design of structures in moderate seismicity regions, provided that the assumptions and input parameters are carefully selected and validated based on the site-specific conditions and structural behavior.

3.4 Iterative Analysis Procedure

In the present study, the initial step involved conducting a thorough analysis of the timber frame under gravity loads only, with the aim of selecting appropriate initial frame sections. Using these initial sections, the structures were modeled, and lateral seismic loads were then applied and analyzed. Based on the analysis results, steel elements were designed for both ends of the beams to enhance their seismic performance. To ensure that the remaining timber elements remain elastic, they were designed to accommodate the maximum credible capacity of steel beams. The next step involved designing the columns using capacity design procedures to ensure that their capacity is greater than that of the beams they are connected to. By following this process, the nonlinear behavior is expected to be limited to the steel elements, thereby improving the overall seismic performance of the timber frame.

3.5 Material Properties

The materials employed for constructing the test specimens comprised standard Canadian material grades or their corresponding equivalents. The nonlinear replaceable links integrated into the specimens were from steel rolled w-sections, adhering to CSA G40.21 - 350W standards, exhibiting a nominal yield strength of F_y=350 MPa, and an ultimate strength of F_u = 450 MPa. Additional steel constituents, including the panel zone, were modeled with CSA G40.21 - 300W grade steel, or its equivalent, possessing a yield strength of F_y = 350 MPa, and an ultimate strength of F_u = 450 MPa. The glulam sections employed for fabricating the beams and columns in the models were from Nordic Engineered Wood, and specifically employed 24F-ES/NPG grade lumber designed for Nordic wood products. This resulted in slightly divergent mechanical properties in comparison to the specifications outlined in CSA-086-09. A comprehensive overview of the mechanical properties associated with this grade is presented in Table 3.7 (Nordic Engineered Wood, 2013).

Table 3-2- Mechanical properties of timber elements

Specified Strength	24F-ES/NPG (MPa)
fb+	30.7
fb–	30.7
fv	2.2
fcp	7.5
fc	33
ft	20.4
E	12400
Mean Relative Density	0.47
Density (kg/m3)	560

3.6 P-Delta Effect

The P-Delta effect, also known as geometric nonlinearity, relates to the equilibrium and compatibility relationships of a structural system loaded about its deflected configuration. It involves the application of gravity load on laterally displaced multi-storey building structures, resulting in magnified story drift and certain mechanical behaviors while reducing deformation capacity. Abrupt changes in ground shear, overturning moment, and the axial force distribution at the base of a sufficiently tall structure or structural component are referred to as the P-Delta effect when subjected to a critical lateral displacement.

Several researchers have explored the effect of P-Delta analysis on various structures in their studies. Gupta et al. (1999) investigated the inelastic response of steel moment-resisting frames induced by negative post-yield story stiffness. Their results highlighted the need to consider the P-Delta effect as a potential collapse hazard during the design process. Black et al. (2001) introduced and evaluated two stability coefficients to quantify the P-Delta effect during elastic and inelastic lateral displacement of regular seismic moment resisting frames. The combined use of these coefficients permitted accurate prediction of the load-deformation curve of special moment resisting frames (SMRF) affected by P-Delta phenomena. Scholz et al. (2006) developed a novel method to allow for the P-Delta effect of steel sway frames analyzed by elastic methods, which suggests the possibility of obtaining more economical designs.

The P-delta effect in a structure is accounted for by amplifying the first order moment in the bending members, attained through an elastic analysis. This factor is determined using estimated story drifts based on previously studied frames for a moment resisting frame (MRF) system. Neglecting P-delta effects can lead to significant errors in structural analysis, which can compromise the safety and stability of the structure (Lin, 2020).

ETABS software incorporates this effect through an iterative analysis process. Initially, a linear static analysis is performed to determine the initial member forces and deflections. Then, an incremental load is applied to the structure in proportion to the initial member deflection, and the structure is re-analyzed iteratively until the solution converges. During each iteration, the stiffness matrix of the structure is updated based on the previous step's deflection until convergence is achieved. Lastly, the final member forces and deflections are obtained, which consider the P-Delta effect. This process ensures that the analysis accurately captures the behavior of the structure under lateral loads, making it a reliable tool for designing structures that are safe and stable under such loading conditions.

3.7 Structural Irregularities

Structural irregularities refer to the presence of deviations from the standard design and construction practices that can lead to disproportionate seismic responses and structural damage. One common type of structural irregularity is accidental torsion, which results from asymmetries in the building's mass and stiffness distribution. Accidental torsion can significantly amplify the seismic response of a building, which can result in serious damage or even collapse during an earthquake.

To account for accidental torsion effects, building codes, such as the National Building Code of Canada, permit the inclusion of a 10% offset between the center of mass and rigidity which is considered in the design of this study’s archetypes. It is important to note that such provisions should not be relied upon as a substitute for proper design and construction practices that aim to minimize the occurrence of structural irregularities.

The importance of addressing structural irregularities in seismic design and construction has been highlighted in numerous studies. For instance, a study by Chopra and Goel (2002) investigated the effects of torsion on the seismic response of irregular buildings and concluded that accidental torsion can significantly increase the lateral drift and bending moments in structural members. Similarly, a study by Kunnath et al. (2015) emphasized the need for proper modeling and analysis of accidental torsion effects in high-rise buildings to ensure their seismic safety.

3.8 Panel Zone

During the preliminary design phase of the building, it was determined that the high panel zone shear forces would likely lead to excessive longitudinal shear and tension perpendicular-to-grain stresses if the panel zone was made of timber. The brittle failure of the panel zone during a severe seismic event would hinder the effectiveness of the plastic hinge. Therefore, the timber panel zone was replaced with a custom steel section. The steel panel zone was designed and detailed according to Limit States Design of Steel Structures (CSA S16-19, 2019).

The steel panel zone was designed based on the bending capacity and the plates were proportioned to ensure that the panel zone remains elastic under peak plastic moment capacity at the plastic hinges. The design of the steel panel zone in the building aligns with the principles of modern seismic design, which focus on enhancing the ductility of structural elements and reducing their susceptibility to brittle failure during a seismic event. Previous studies have shown that the use of steel panel zones can lead to more ductile behavior of the structural system and increase the overall strength and stiffness of the building (Nakaki, 2009).

Furthermore, the use of continuity plates has been shown to improve the seismic performance of steel moment-resisting frames by enhancing the resistance of the beam-to-column connection to lateral loads and reducing the risk of brittle failure (El-Tawil, 2006).

By including the panel zones in the design of the archetypes in this study, a more comprehensive investigation of the structural behavior can be carried out, as the panel zones play a crucial role in determining the overall response of moment resisting structures subjected to dynamic loads.

3.9 Nonlinear Steel Link

The steel sections of beams use a W-section and is designed to resist the moment demand at the beam-column interface of the selected joint location. The sizing of the steel beams is based on the Limit States Design of Steel Structures (CSA S16-19, 2019), which specifies the full plastic factored moment capacity. To prevent any other components from reaching plastic deformation, all components must be designed to resist forces greater than the anticipated peak plastic moment capacity at the plastic hinge location, which is at the center of the link and effects of the expected yield stress of the section is also considered. The estimated value of the peak plastic moment capacity is determined using equation (3-1):

$$M_{pr} = C_{pr} R_y F_y Z_x \qquad (3\text{-}1)$$

where R_y is a factor applied to F_y to estimate the probable yield stress and is considered 1.1, and C_{pr} is 1.15 and Z_x is the plastic modulus of the gross cross-section.

3.10 Design of Timber Members

In designing the timber members, it is important to consider their brittle failure mechanism. To ensure adequate strength, timber beams are designed to withstand the full probable moment capacity of the steel sections, while each timber column should be designed to withstand half of the moment applied to the column center, due to plastic hinging in the link. The moment at the panel zone is then split between upper and lower columns based on relative stiffness. The moment capacity of the timber beams and columns is determined based on CSA O86-19, (2019).

4. Numerical Modelling

4.1 Introduction and objectives

Chapter 3 provided realistic designs for the building archetypes to be investigated, where each shearline was designed and detailed for the force received during the analysis. A simplification for the nonlinear push-over analysis is typically to consider only a single shearline and perform the analysis based on two-dimensional models. The objective of this numerical investigation is to ascertain whether the considered seismic load resisting system is capable of achieving the desired performance level and if it could attain the behaviour similar to that of a steel-only bracing system. An example of the 2D structural layout employed for one of the frame models is presented in Figure 4.1.

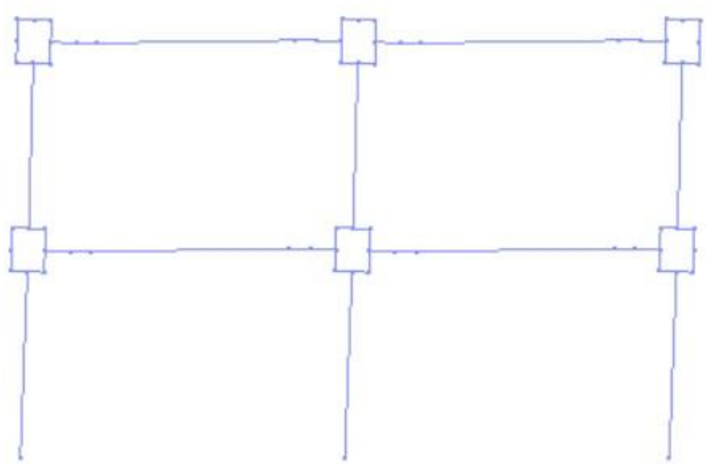

Figure 4- 1 An elevation view of the structural layout of a two-story frame

This chapter outlines the methodology used to numerically model the timber-steel moment frames evaluated in this study. The outcome from the analysis will help assess the force modification factors of such frames and determine if their performance meets the seismic performance assumed for moment connection frames structural systems in the NBC. To achieve this goal, full-scale 3D MRFs were modeled and design, taking into account different architectural and structural parameters, such as the number of stories, number of spans, and span length. Moreover, the frames were located in two regions with different seismic activity levels, namely Vancouver and Montreal.

The nonlinear structural analysis software OpenSees was utilized to carry out the numerical investigations. Nonlinear static analysis was executed to assess the force modification factors of the various archetypes. To calibrate the stiffness and nonlinear behavior of the connection models, the study conducted by Gohlich et al. (2015) was used as a basis for modeling the experimental results. This would ensure that the behavior of the complete building model adequately represents the connection behavior, which is essential to the behaviour of the overall building. The calibration procedure was mainly focused on establishing accurate connection models as these are crucial factors affecting the behavior of the frames subjected to seismic loading.

4.2 Modelling Verification

4.2.1 Loading Protocol

According to Gohlich et al. (2015), the loading protocol applied for the cyclic testing of the connection was adapted from AISC-341 Seismic Provisions for Structural Steel Buildings (2010), which contains testing provisions for prequalified steel connections. The specified loading protocol utilized was a cyclic displacement-based loading with target displacement amplitudes derived from the overall story drift of the frame, as illustrated in Figure 4-2.

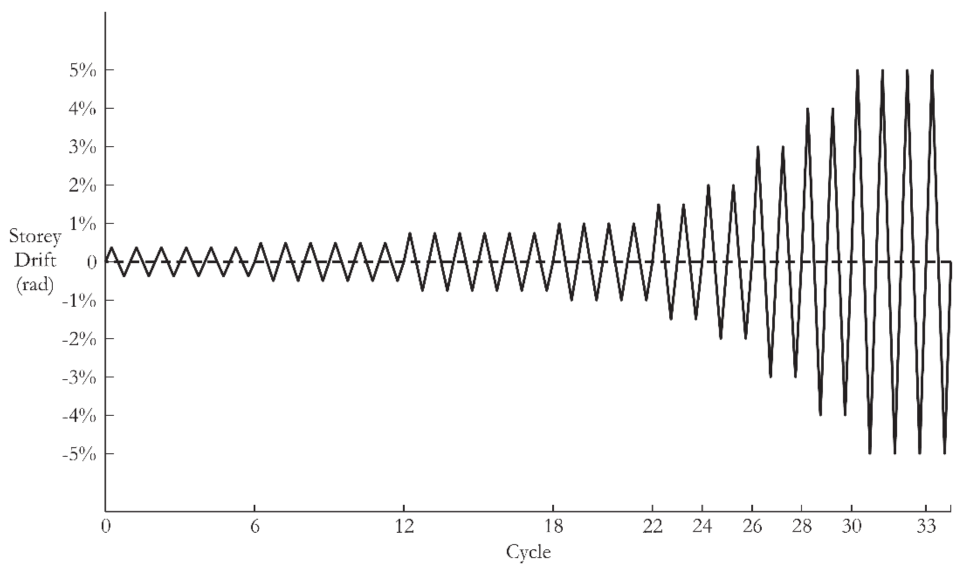

Figure 4- 2 cyclic loading protocol (AISC 341-10, 2010)

The loading protocol comprises nine cycles, with the initial cycle starting at 0.00375 rad and the final cycle concluding at 0.05 rad. It is noteworthy to mention that this loading procedure is theoretical and that there have been some discrepancies in imposed loadings during experimental tests. Thus, these differences may lead to variations between the numerical and experimental results (Table 4-1 and 4-2).

Table 4-1- Rotation vs. Displacement of the tip of the beam

No. of Cycles	Beam Rotation (rad)	Beam Tip Disp (mm)
6	0.00375	8.65
6	0.005	11.53
6	0.0075	17.3
4	0.01	23.06
2	0.015	34.59
2	0.02	46.13
2	0.03	69.19
2	0.04	92.25
n	0.05	115.32

Table 4-2- Deviation of drifts

Drift	Difference positive (%)	Difference negative (%)
0.00375	3.2	4.2
0.005	2.2	1.2
0.0075	1.7	1.3
0.01	1.5	1.5
0.015	1.9	1.9
0.02	1.9	2
0.03	1.8	0.7
0.04	2.3	0.02
0.05	2.7	0.3

In the subsequent step, a model was constructed and subjected to analysis, utilizing the specific experimental details outlined in the study conducted by Gohlich (2015). The ensuing investigation pertained to the predefined loading protocol discussed earlier. Figures 4-3 and 4-4 showcase the experimental test results alongside the outcomes obtained from the modeling conducted in the present study. it is worth noting that the analytical results obtained from the model in this present study closely align with the experimental findings, indicating a high degree of correlation between the two sets of data.

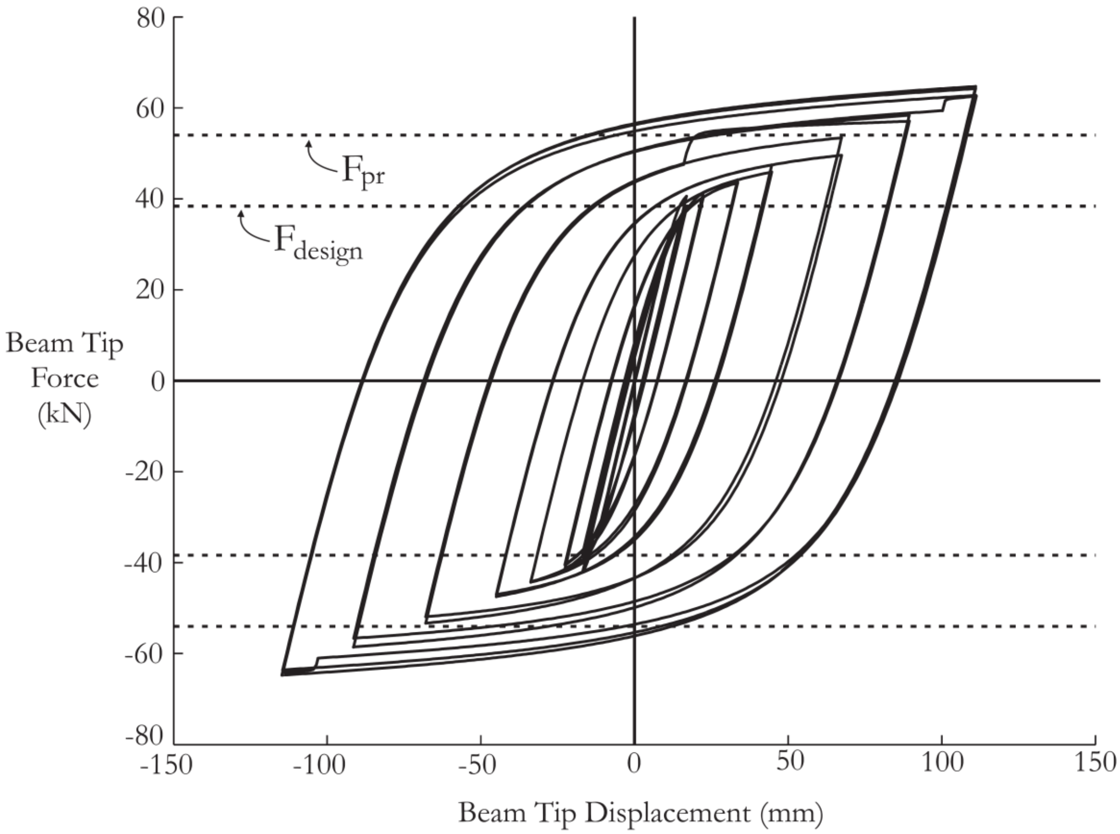

Figure 4- 3 Force-Displacement response of experimental tests (Gohlich, 2015)

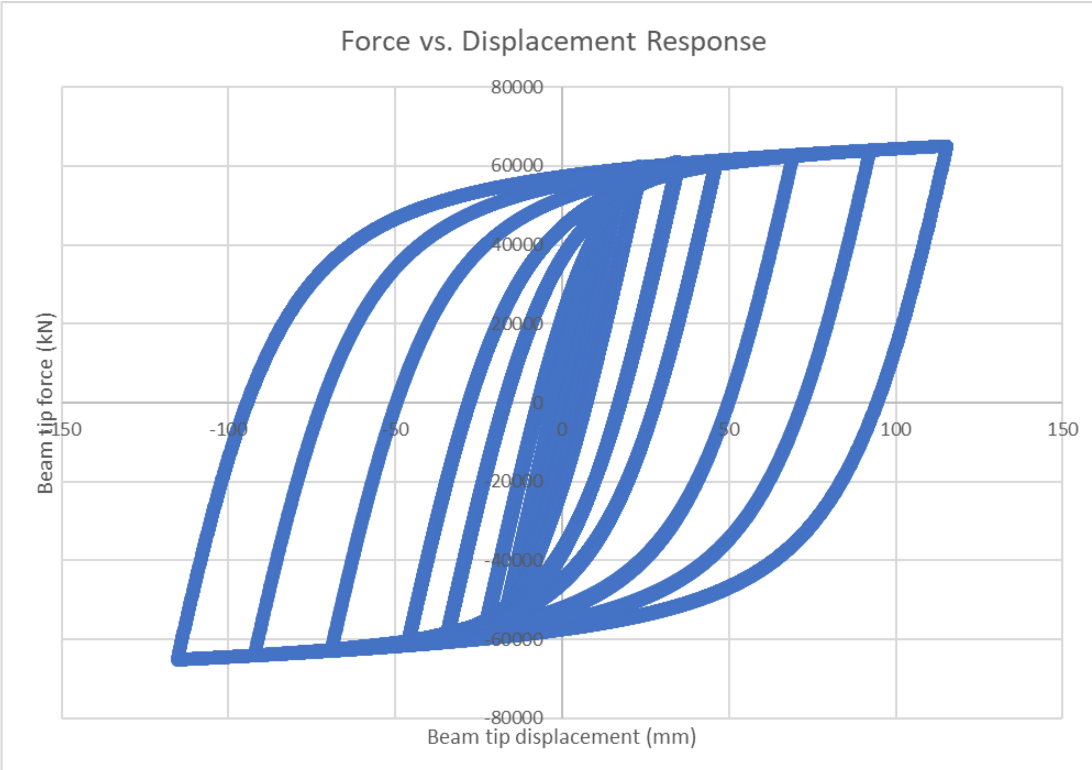

Figure 4- 4Force-Displacement response of current study analytical modeling

4.2.2 Modeling Elements

To construct the numerical model, all timber elements were modeled using the "ElasticBeamColumn" element found in OpenSees. The segment where the timber beam is connected to the steel section was assumed to be rigid, and as such a high modulus of elasticity was employed in the model. The most critical aspect of this model was the link element's modeling. The experimental test results indicated that the majority of nonlinear deformations were concentrated in the middle of the steel section, and in RBS links, specifically at the midpoint of the dog-bone region.

In the prior study, the authors segmented the RBS links into three distinct parts. Both ends of the links were modeled using the "ElasticBeamColumn" element. The middle part of the link was modeled using the distributed nonlinearity concept, employing a "NonlinearBeamColumn" element with fiber sections. One of the assumptions made in this segment was that the entire length of the dogbone area had the smallest width of the flange, which is not entirely accurate in reality. Hence, without conducting any tests, it is expected that this model has a lower strength than that observed in experimental tests. To address this issue, the authors attempted to increase the steel strength. They considered a steel material strength of 400 MPa, although it was actually 350 MPa.

Initially, it was observed that current study's numerical models had lower strength than expected. In an attempt to increase the strength, the steel-yielding stress was increased in current study. However, in order to reach the strength observed in experimental tests, a substantial increase in the Fy value was required.

To address the issue of low strength observed in the numerical models, there are several parameters that can be adjusted. One option is to decrease the length of the link. Another option is to increase the yielding strength of the material. A third option is to increase the width of the flange. In this study, the flange width has increased, and this adjustment led to acceptable results.

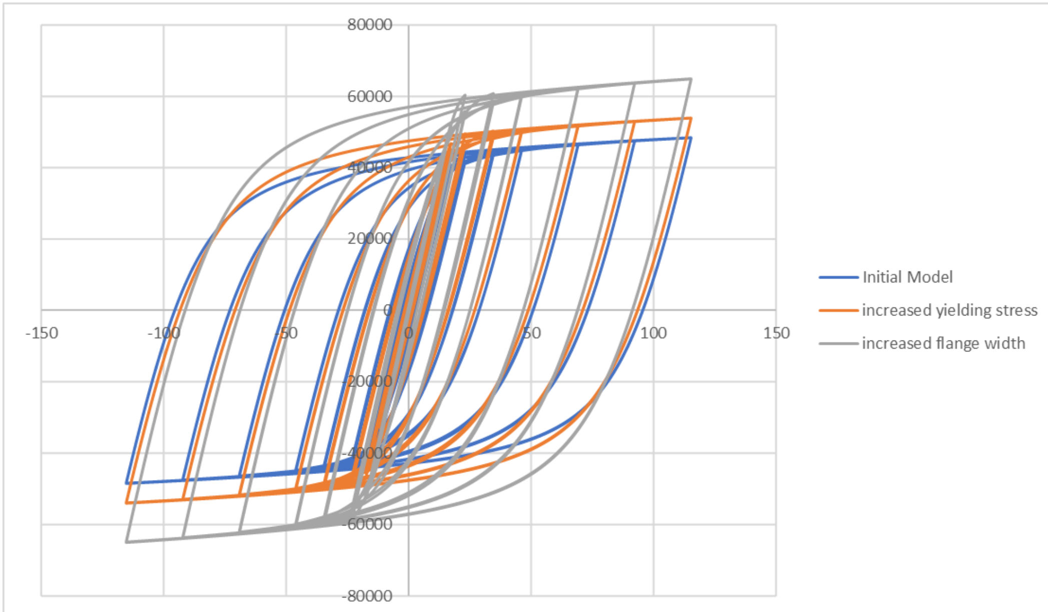

Figure 4-6 Effect of different changes on the results

4.3 Pushover Analysis

The pushover analysis is typically performed by incrementally increasing the lateral loads in the structure until a specified performance level is reached. The performance level can, for example, be defined in terms of a maximum inter-story drift or a maximum global drift, which corresponds to a particular damage or collapse limit state. The pushover curve is a plot of the base shear versus the top displacement of the structure, and it provides information on the capacity and response of the structure at different levels of lateral loading.

The pushover analysis is a simplified method for assessing the seismic performance of a structure, and it has some limitations. It assumes that the structure behaves in a linear elastic manner up to the yield point, and that the inelastic deformations are concentrated in a few critical elements or regions of the structure. It also assumes that the lateral loads are applied uniformly across the height of the structure, which may not be the case in real earthquakes.

Pushover analysis provides information on many response characteristics that cannot be obtained from a linear static or dynamic analysis, such as estimates of the deformation demands for elements deforming inelastically to dissipate the energy imparted onto the structure by the ground motions. It can also identify strength discontinuities in plan or elevation that will lead to changes in the dynamic characteristics in the inelastic range and estimates of inter-story drifts that account for strength or stiffness discontinuities and that may be used to control damage and to evaluate P-delta effects. Therefore, it can be a valuable tool for evaluating seismic performance and identifying critical regions that require attention.

The present study focuses on the seismic analysis and design of structures and aims to comply with the requirements outlined in FEMA 356 (2000), a standard for seismic rehabilitation of buildings. The study involves selecting a control node, which is a key element in the analysis process, and determining appropriate lateral load patterns. The fundamental period of the structure is also determined, as it is crucial in assessing the building's response to seismic loads. To accurately model the behavior of the structure, the components gravity loads are included in the mathematical model, which is combined with lateral loads. The control node for all models was located at the center of mass at the roof level. The displacement of the control node in the analytical model is recorded for the specified lateral loads.

4.3.1 Force-displacement relationship

To determine the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building, an idealized bilinear relationship should be used to replace the nonlinear force-displacement relationship between base shear and displacement of the control node. Figure 4-8 illustrates this relationship, which consists of two-line segments with an initial slope K_e and a post-yield slope α K_e. The location of the line segments on the idealized force-displacement curve is determined using an iterative procedure that balances the area above and below the curve. The effective lateral stiffness, K_e, is calculated as the secant stiffness at a base shear force equivalent to 60% of the effective yield strength of the structure. The post-yield slope, αK_e, is determined by a line segment that intersects the actual curve at the target displacement Figure 4-9 depicts the nonlinear curve and idealized bilinear curve for one of the models.

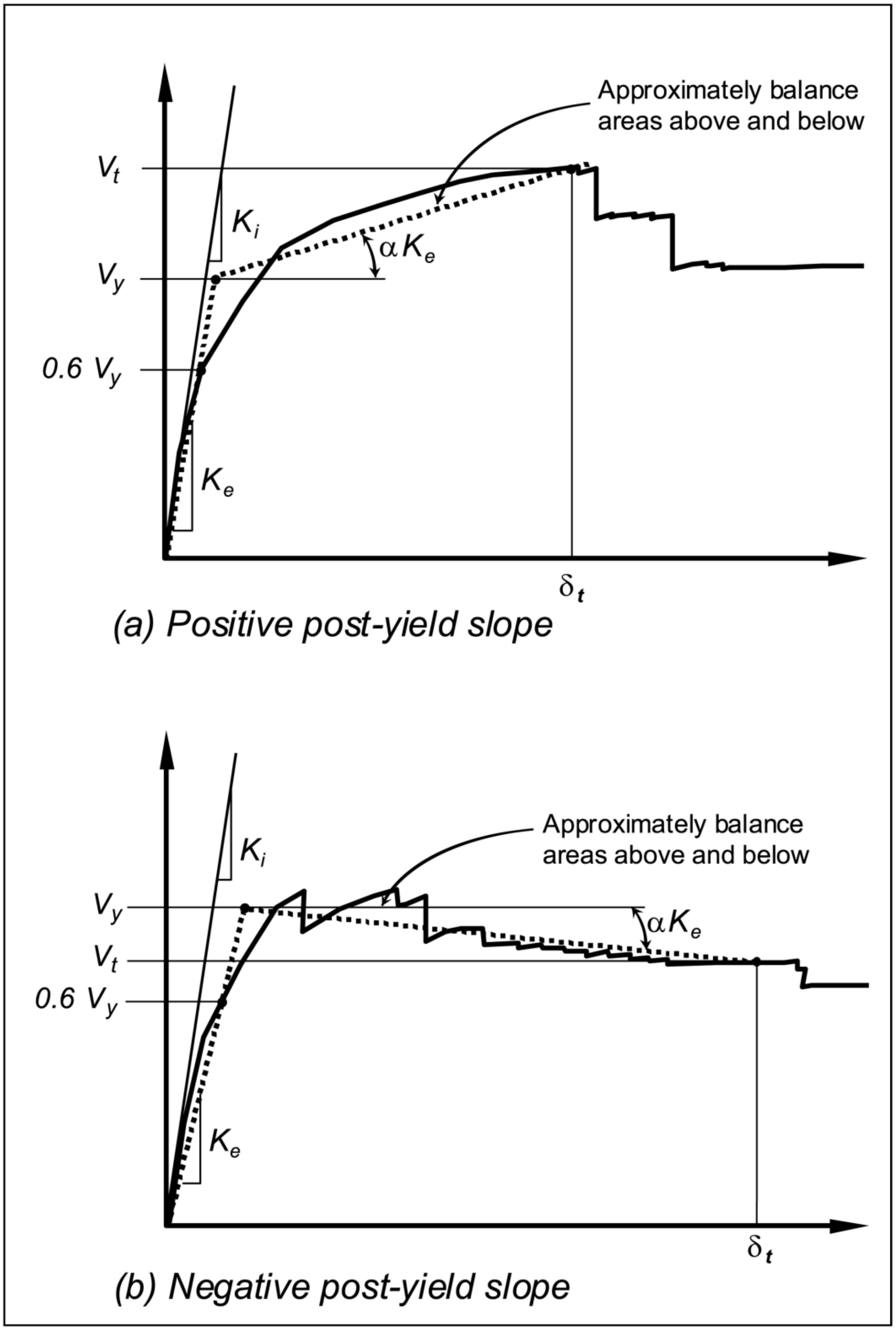

Figure 4-8 Idealized force-displacement curves (FEMA 356, 2000)

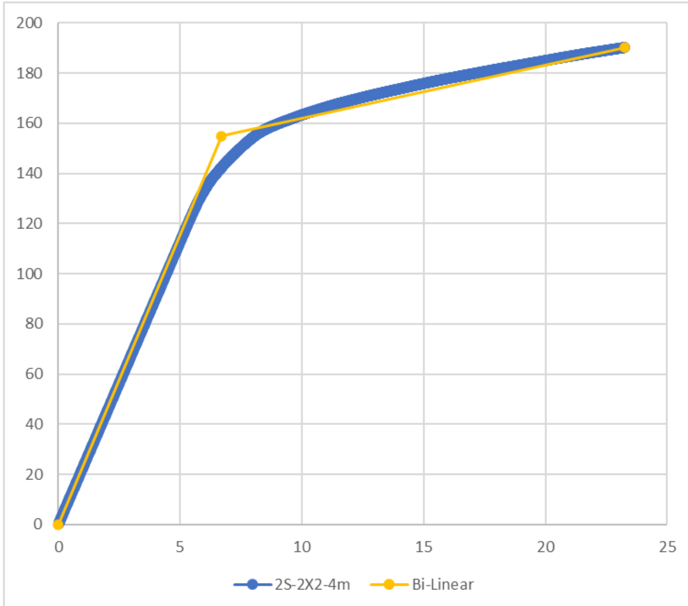

Figure 4-9 idealized curve of a two-story hybrid frame

4.3.2 Target displacement

The target displacement, δ_t, at each floor level, is calculated in accordance with Equation 4-1 and as specified in FEMA 356 (2000):

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{4-1}$$

In which C_0 is modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system, C_1 is modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, C_2 is modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response, C_3 is modification factor to represent increased displacements due to dynamic P-Δ effects, S_a is response spectrum acceleration, g is acceleration of gravity, and T_e is effective fundamental period.

The aforementioned methodology was employed to ascertain the target displacement for the models under investigation. However, it was observed that the displacement achieved through this method was insufficient to induce full plastic deformation in the models. Tracking the behavior of fibers within the steel sections revealed that in order to attain a plastic section, the pushover analysis needed to be extended to higher displacement levels. Thus, to ensure the attainment of fully plastic steel sections, a recording mechanism was incorporated into the codes to monitor the stress in the steel fiber elements. Subsequently, each model was subjected to progressive loading until the steel sections reached a plastic state, allowing the maximum capacity of each frame to be effectively utilized.

5. Results and Discussion

This study aims at examining a novel timber-steel moment-resisting frame system and assessing its seismic performance. The findings of this study is expected to pave the way for additional investigations into this system's comprehensive properties and characteristics. Thus, this study represents a preliminary stage in exploring the nonlinear behavior of timber-steel moment-resisting frames.

To establish a satisfactory range of archetype models and account for the most influential factors in the seismic behavior, the archetypes were varied in terms of span length, number of stories, and seismicity levels. The span lengths of the archetypes were varied by 4, 6, and 8 meters and since timber moment resisting frames are limited to a height of 20 meters, the number of stories was selected based on three different options: 2, 4, and 6, resulting in a total of nine models. The study also considered seismicity by examining two regions with different seismic activity levels, namely Montreal and Vancouver. In total, 18 models were investigated in this study.

5.1 Overstrength-related response modification factor

One of the applications of pushover analysis is to assess response modification factors. Figure 5-1 shows the determination of the lateral design force, V, including ductility- and overstrength-related force modification factors.

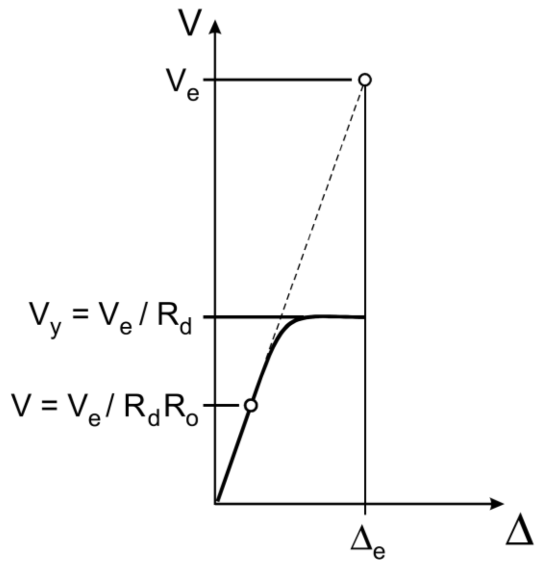

Figure 5.1 Determination of the lateral design force, V, including ductility- and overstrength-related force modification factors (Mitchell, 2003).

For a particular structural system, overstrength arises from the application of the design and detailing provisions prescribed in the appropriate design standard. The NBCC (2015) has overstrength factors, R_o, that have been determined for all systems in conformance with the CSA provisions. To account for the various components contributing to the overstrength-related force modification factor, R_o, the following formulation has been proposed by Mitchell (2003):

$$R_o = R_{size} R_{\phi} R_{yield} R_{sh} R_{mech} \quad \text{5-1}$$

where R_{size} accounts for the overstrength resulting from restricted options for member and element sizes, as well as rounding of sizes and dimensions. R_{ϕ} expresses the difference between nominal and factored resistances, with a value equal to 1/ϕ, where ϕ represents the material resistance factor defined in the CSA standards. R_{yield} represents the ratio of the actual yield strength to the minimum specified yield strength, while R_{sh} indicates the overstrength resulting from strain hardening development. Finally, R_{mech} represents the overstrength arising from the complete mobilization of the structure's capacity, culminating in the formation of a collapse mechanism.

Table 5-1 presents a summary of the R_o values obtained for the 18 archetypes examined in this study. These values were derived as the ratio between peak base shear and design base shear depicted in Figure 5.1.

Table 5- 1 R_o for all studied archetypes

Archetype			V (MPa)	V_d (MPa)	R_o
Region	Span Length (m)	Story			
Montreal	4	2	27.5	16.3	1.7
		4	39.0	23.0	1.7
		6	58.0	33.9	1.7
	6	2	59.0	31.2	1.9
		4	75.0	46.0	1.6
		6	90.0	55.0	1.6
	8	2	90.0	46.0	2.0
		4	108.0	68.0	1.6
		6	120.0	71.0	1.7
Vancouver	4	2	50.0	27.9	1.8
		4	70.0	39.5	1.8
		6	76.0	45.0	1.7
	6	2	105.0	62.0	1.7
		4	155.0	101.0	1.5
		6	213.0	128.0	1.7
	8	2	178.0	110.0	1.6
		4	260.0	150.0	1.7
		6	345.0	189.0	1.8

The National Building Code of Canada (NBCC, 2015) provides an overstrength-related force modification factor of 1.5 for both moderately ductile and limited ductility timber moment resisting frames, as well as ductile moment resisting steel frames. The results in Table 5.1 indicate that the evaluated R_o factor for various models ranges between 1.5 and 2.0, and that with a minimum value of 1.5 the NBC (2015) recommendation seems reasonable.

From Table 5-1, it is evident that in general an increase in the number of stories, or building height, resulted in a decrease in the R_o factor. An exception to this is found in models featuring 6 stories and an 8-meter span length or one of the 4-story models. The rationale behind this inconsistency may be attributed to the utilization of larger beam sections in an effort to mitigate story drift.

5.2 Ductility-related response modification factor

The ductility reduction factor is a function of the characteristics of the structure including ductility, damping, fundamental period of vibration, and the characteristics of earthquake ground motion. Newmark and Hall proposed a set of equations expressing ductility-related response modification factors and concluded that for T>0.5 s, the ductility-related factor is effectively equal to the ductility of the structure. Table 5.2 presents the R_d factor values for the archetype models.

Table 5- 2 Rd factors all studied archetypes.

Archetype			V_e	V	R_d
Region	Span Length	Story			
Montreal	4	2	168.1	27.5	6.1
		4	217.2	39.0	5.6
		6	327.4	58.0	5.6
	6	2	330.9	59.0	5.6
		4	376.4	75.0	5.0
		6	455.4	90.0	5.1
	8	2	482.1	90.0	5.4
		4	568.8	108.0	5.3
		6	396.41	120.	3.3
Vancouver	4	2	326.9	50.0	6.5
		4	400.0	70.0	5.7
		6	437.5	76.0	5.8
	6	2	604.4	105.0	5.8
		4	826.5	155.0	5.3
		6	1075.9	213.0	5.1
	8	2	1012.9	178.0	5.7
		4	1363.6	260.0	5.2
		6	1523.3	345.0	4.4

In accordance with the National Building Code of Canada (NBCC, 2015), timber MRFs ductility factors of 2.0 and 1.5 are used for moderately ductile and limited ductility MRFs. However, since the system use involves deformation and contribution to the ductility primarily from the steel section, a comparison with steel MRF is undertaken. For steel MRFs values of 5.0, 3.5, and 2.0 are proposed for ductile, moderately ductile, and limited ductility MRFs, respectively.

The results of the present study demonstrate that, with the exception of the 6-story 8-meter model located in Vancouver, all models exhibit ductility-related factor values greater than the recommended level of 5 for ductile steel moment resisting frames. This suggests that timber-steel moment-resisting frames may represent a viable alternative to steel moment-resisting frames.

It is worth noting, however, that a more comprehensive investigation is needed to fully explore the characteristics of this system, including its connection details.

It can be observed that an increase in the number of stories (building height) results in a decrease in the ductility-related response modification factor (Figures 5.2 and 5.3). This trend could be attributed to the concentration of lateral demands within specific lateral load resisting elements, and as such not optimizing the energy dissipative capacity of the building. In contrast, shorter frames or those with fewer stories exhibit a better distribution of lateral forces, resulting in more evenly distributed demands among all lateral load resisting elements.

The height limitations for timber frames imposed certain constraints that prevented the selection of taller buildings. Consequently, the selected stories were in close proximity, and any alterations in the design process or changes in steel section choice could have a significant impact on the observed trends.

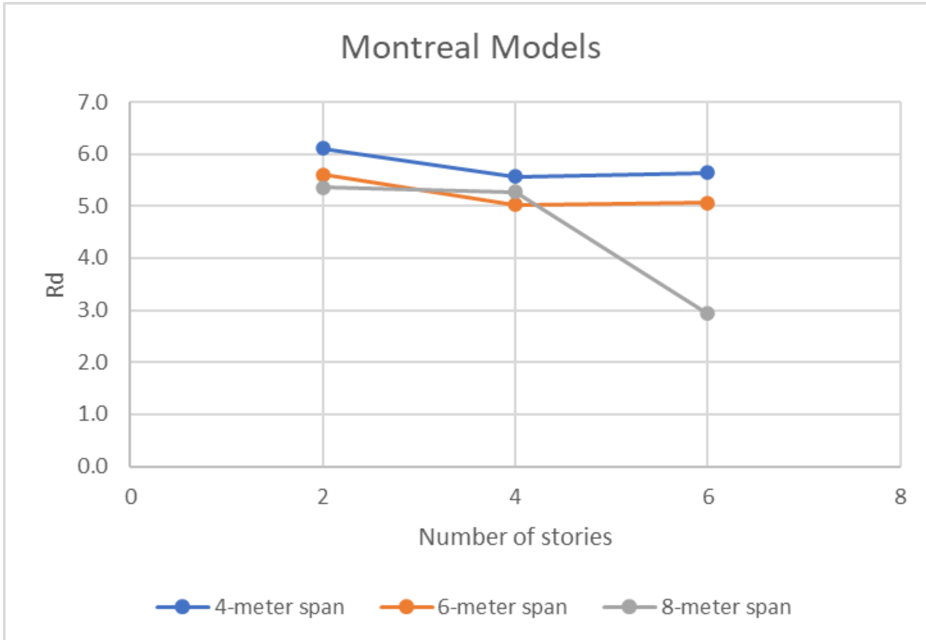

Figure 5.2 Clustered Rd factors for Montreal models

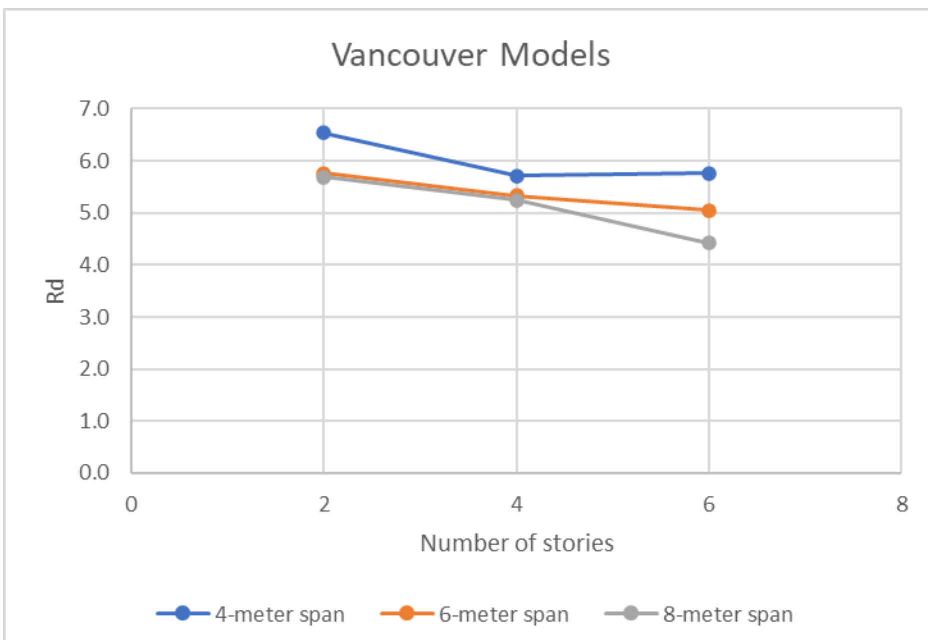

Figure 5.3 Clustered Rd factors for Vancouver models

Table 5.2 and Figures 5.4 and 5.5 provide additional insights into the R_d factors, where it can be observed that an increase in the span length has a decreasing effect on the ductility-related factor. This trend could be attributed to the lower contribution of shear in the steel elements. Laboratory studies on the behavior of steel links (Hjelmstad and Popov, 1983, 1984, and Malley and Popov, 1984) demonstrated that shorter steel beams exhibit larger and more stable hysteretic loops, leading to better energy dissipation and ductility.

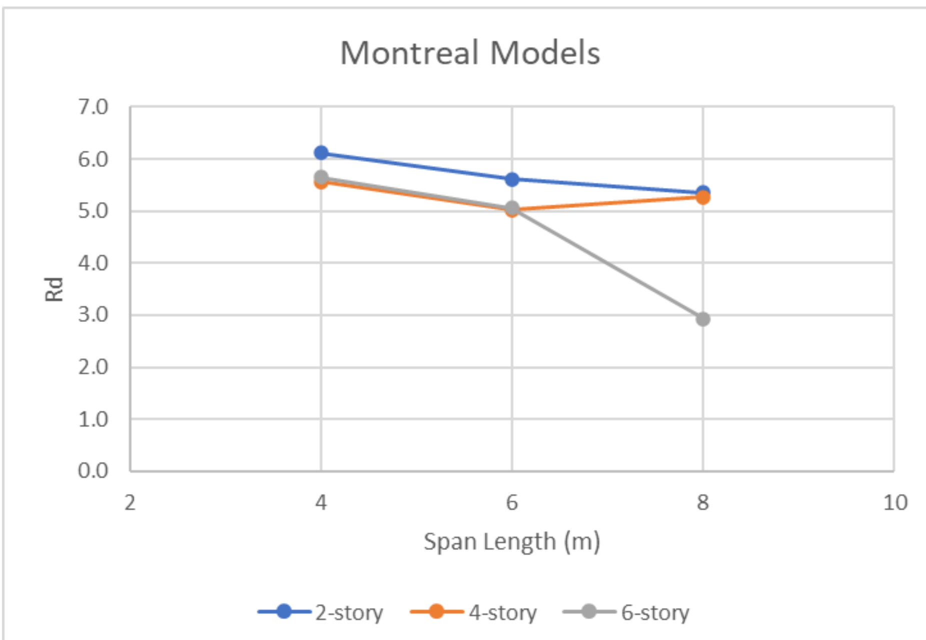

Figure 5.4 Clustered Rd factors for Montreal models

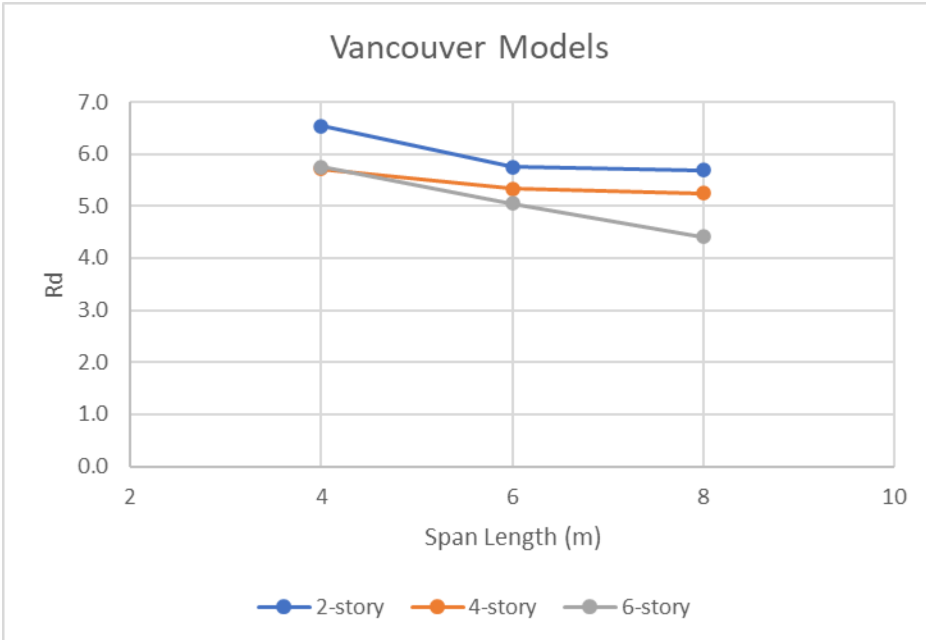

Figure 5.5 Clustered Rd factors for Vancouver models

5.3 American and Canadian code approaches for evaluating response modification factors

In evaluating the ductility factors from the pushover curves, the elastic strength should be divided by yielding strength. Additionally, the yield strength should be divided by an overstrength factor to reach the design level. The American approach considers the strength at the intersection of the bilinear behavior on the pushover curve as a basis for calculating the overstrength- and ductility-related portions of response modification factor (Uang, 1991). Conversely, Canadian approach utilizes the peak strength on the pushover curve as the basis for calculating response modification factors.

This approach to calculating response modification factors in Canadian codes leads to proposing higher overstrength factors compared to American codes. Consequently, the ductility factor calculated through this approach is lower in Canadian codes than in American codes.

5.4 Review of pushover curves

In this section models with the same span length are grouped together due to their similar architectural characteristics and, thus, their relatively comparable structural behavior. Additionally, models located in the same seismic region are clustered together to provide a more comprehensive understanding of the potential trends that may be observed in these results.

Figures 5.6 to 5.11 present the pushover curves for all models grouped relative to their location and span length.

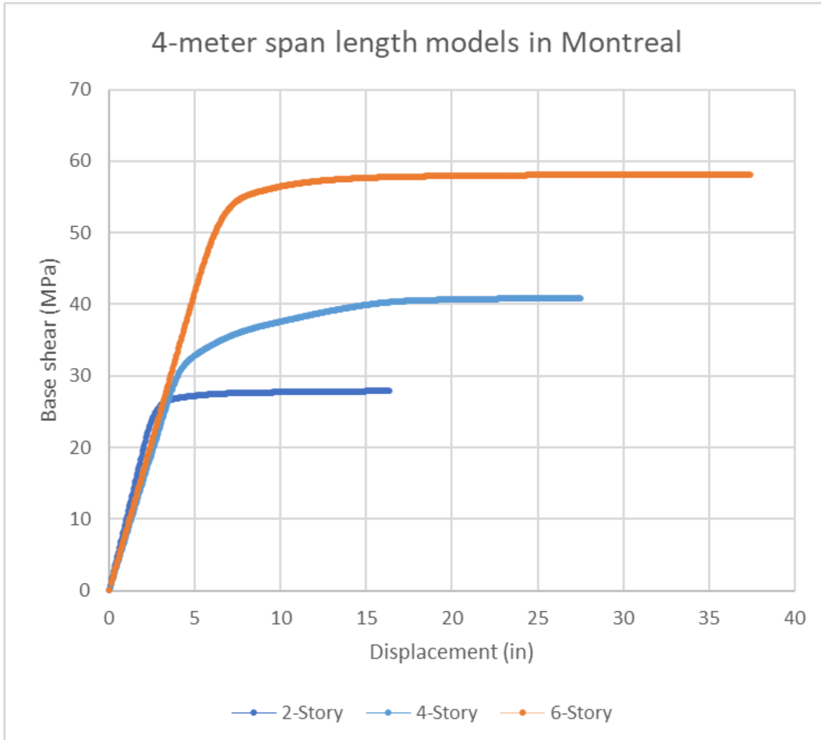

Figure 5.6 Pushover curves for Montreal Models

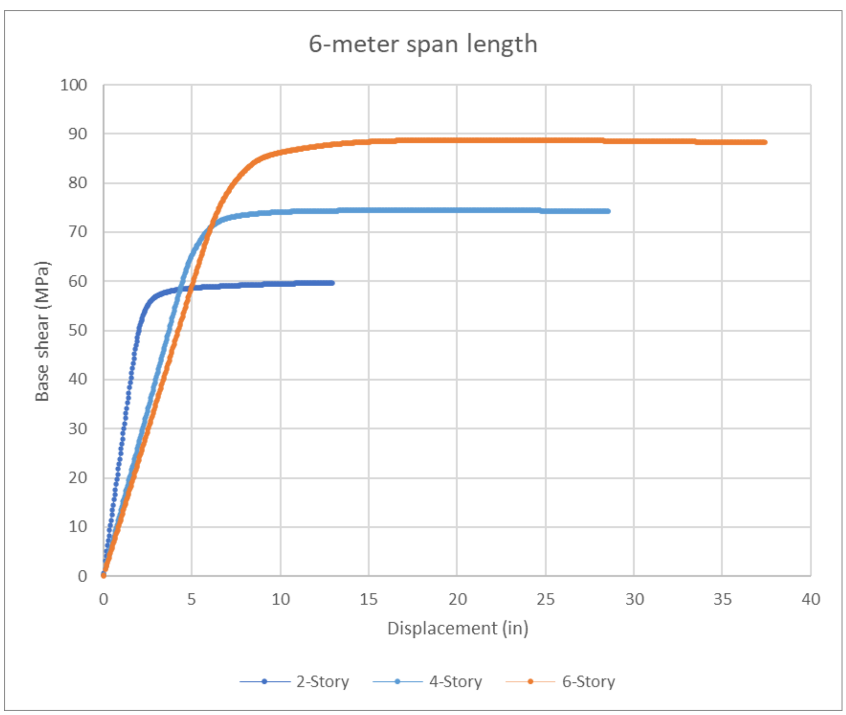

Figure 5.7 Pushover curves for Montreal Models

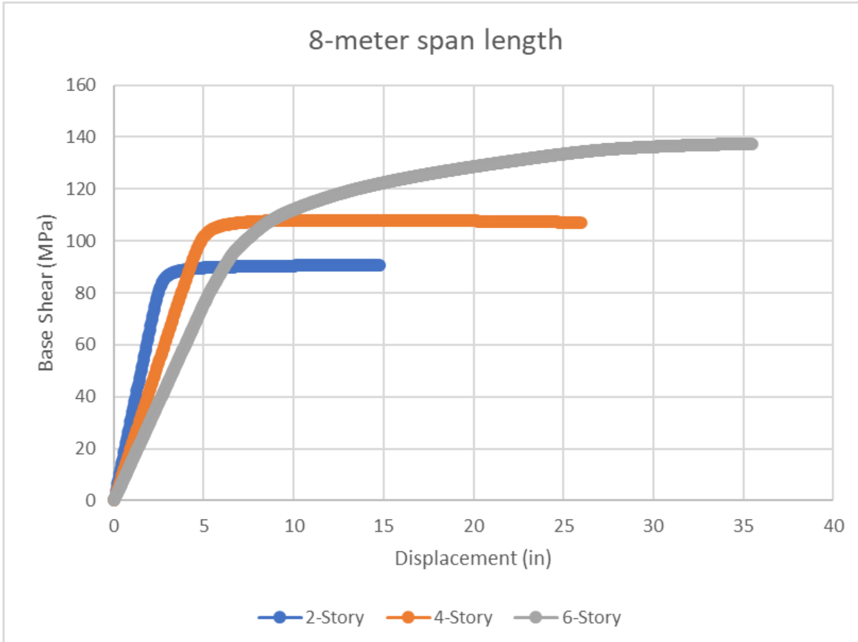

Figure 5.8 Pushover curves for Montreal Models

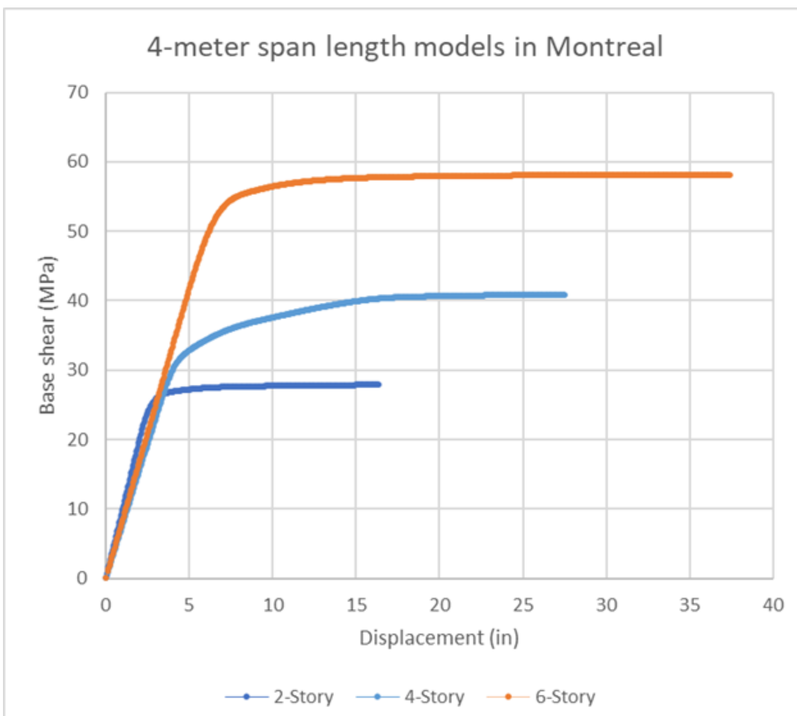

Figure 5.9 Pushover curves for Vancouver Models

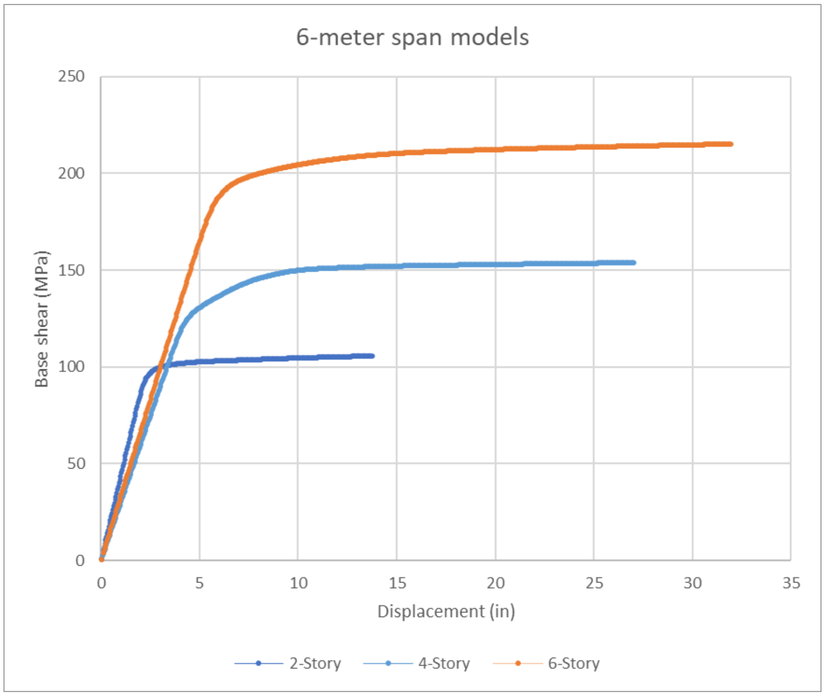

Figure 5.10 Pushover curves for Vancouver Models

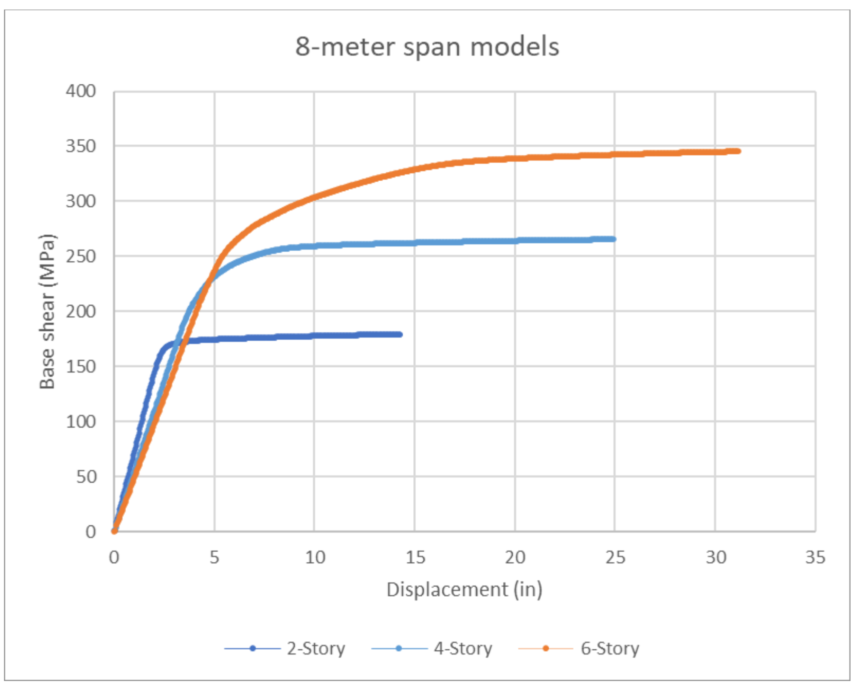

Figure 5.11 Pushover curves for Vancouver Models

The pushover curves reveal that an increase in the span length results in a decrease in the initial stiffness of the frames. This trend is evident in most of the clusters, however, instances where the stiffness of a model does not conform to this trend could be attributed to the selection of steel sections. During the design phase of these frames, multiple steel sections with different section properties, specifically section height but with the same moment capacity, were available. The choice of a deeper or wider section could affect the stiffness of the element and, consequently, the frames.

6. Conclusion

In conclusion, this research has explored the potential benefits of hybrid timber-steel moment connections in enhancing the seismic performance of timber structures. Through preliminary analysis and consideration of various parameters, including span length, number of stories, and seismicity, valuable insights have been gained regarding the behavior and advantages of steel-timber moment connection frames as follows:

- The main objective of this study was to assess whether the hybrid timber-steel system offers superior ductility and performance compared to conventional timber frames. Given that the primary load-carrying element in this system is a steel beam section, it was anticipated that it would exhibit performance characteristics more akin to steel frames than to timber frames. After conducting analyses on different timber-steel moment resisting frames, it became evident that such a system indeed possesses significantly higher response modification factors compared to conventional timber frames prescribed in building codes. Notably, the response modification factors of the hybrid system were found to be closer to those of steel moment resisting frames.

- The analysis results revealed that the overstrength-related factors (R_o) evaluated for different frames exceeded the requirements specified by building codes, albeit not excessively. This indicates that the hybrid timber-steel frames exhibit robustness and the capacity to resist seismic forces effectively.

- In contrast to the overstrength-related factor, the ductility-related factors (R_d)) demonstrated by the hybrid frames were found to be considerably higher than those observed in timber moment resisting frames and approached the levels typically seen in steel moment resisting frames. This signifies that the hybrid system offers enhanced ductility, which is a critical aspect in ensuring structures can undergo significant deformation without catastrophic failure during seismic events.

- Further analyses revealed that taller frames tend to exhibit lower ductility-related factors, suggesting a correlation between frame height and the achievable level of ductility. It is important to consider this relationship in the design of tall timber structures to ensure adequate seismic performance.

- Additionally, frames with longer spans displayed reduced levels of ductility-related factors, indicating that the span length has an impact on the seismic behavior of timber structures. This highlights the importance of considering span length as a significant design parameter in optimizing the seismic performance of long-span timber structures.

- The findings from this research underscore the potential of hybrid timber-steel moment connections to significantly enhance the seismic performance of timber structures, offering improved ductility and overall performance characteristics. These results contribute to the advancement of timber structural design and provide valuable insights for engineers and designers seeking to enhance the seismic resilience of timber buildings.

- Future research endeavors should focus on validating these findings through full-scale experimental testing, exploring additional design considerations, and further optimizing the performance of timber structures under seismic loading conditions.

6.1 Recommendations for Future Work

Based on the findings and limitations of the current study, several research directions are suggested for future investigations:

1. Nonlinear Dynamic Analysis: While the present study utilized nonlinear static analysis to evaluate response modification factors, it is recommended that future research explores the use of nonlinear dynamic analysis. Nonlinear dynamic analysis considers the time-dependent behavior of structures under seismic loading and can provide more accurate insights into the performance and response modification factors of hybrid timber-steel moment connection frames.

2. Full-Scale Experimental Testing: Although the current study focused on analytical aspects of evaluating modification factors, it is essential to validate the analytical models by conducting full-scale experimental tests. Constructing and testing a set of full-scale frames would provide valuable data to verify the accuracy of the analytical predictions and validate the proposed hybrid timber-steel moment connections. These experiments would help in gaining a deeper understanding of the actual behavior and performance of the system.

3. Detailed System Investigation: Further investigation and refinement of the hybrid timber-steel moment connection system's details are necessary to introduce it as a viable structural system. The current study made assumptions regarding the fixity and load-carrying capacity of each element in the system. Future work should focus on obtaining detailed information about each component, such as connections, fasteners, and joint behavior, in order to achieve a higher level of ductility and reliability. This would involve detailed

design considerations and testing of the individual elements to ensure their performance under seismic conditions.

4. Performance under Different Loading Scenarios: The seismic behavior of the hybrid timber-steel moment connection frames can be further explored by investigating their performance under various loading scenarios. This may include studying the effects of different ground motions, dynamic characteristics of the structures, and assessing their response under different intensity levels of seismic events. Such investigations would provide a comprehensive understanding of the system's behavior and help refine the design guidelines for practical applications.

5. Cost-Benefit Analysis: Conducting a thorough cost-benefit analysis of the hybrid timber-steel moment connection frames would provide valuable insights for decision-makers and industry professionals. Comparing the construction and maintenance costs, as well as the expected benefits in terms of enhanced seismic performance and durability, can assist in determining the economic feasibility and viability of implementing these connections in real-world projects.

By addressing these recommendations, future research can contribute to further advancing the understanding and application of hybrid timber-steel moment connection frames in seismic design. The outcomes of these investigations will provide valuable information to engineers, architects, and policymakers, ultimately leading to improved design practices and the adoption of more resilient and sustainable timber structures.

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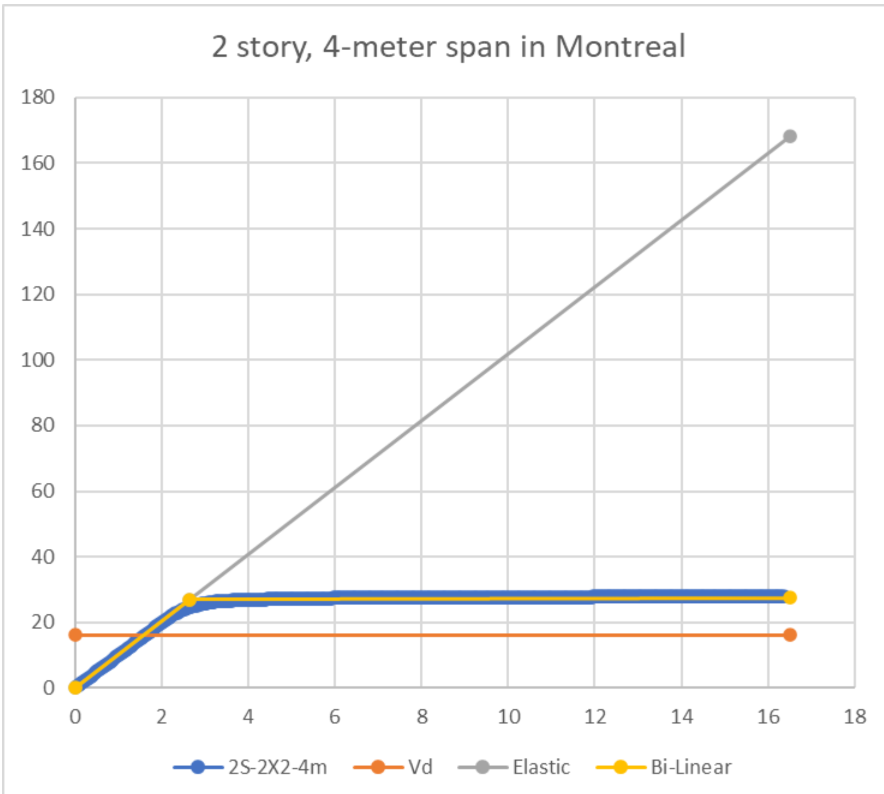
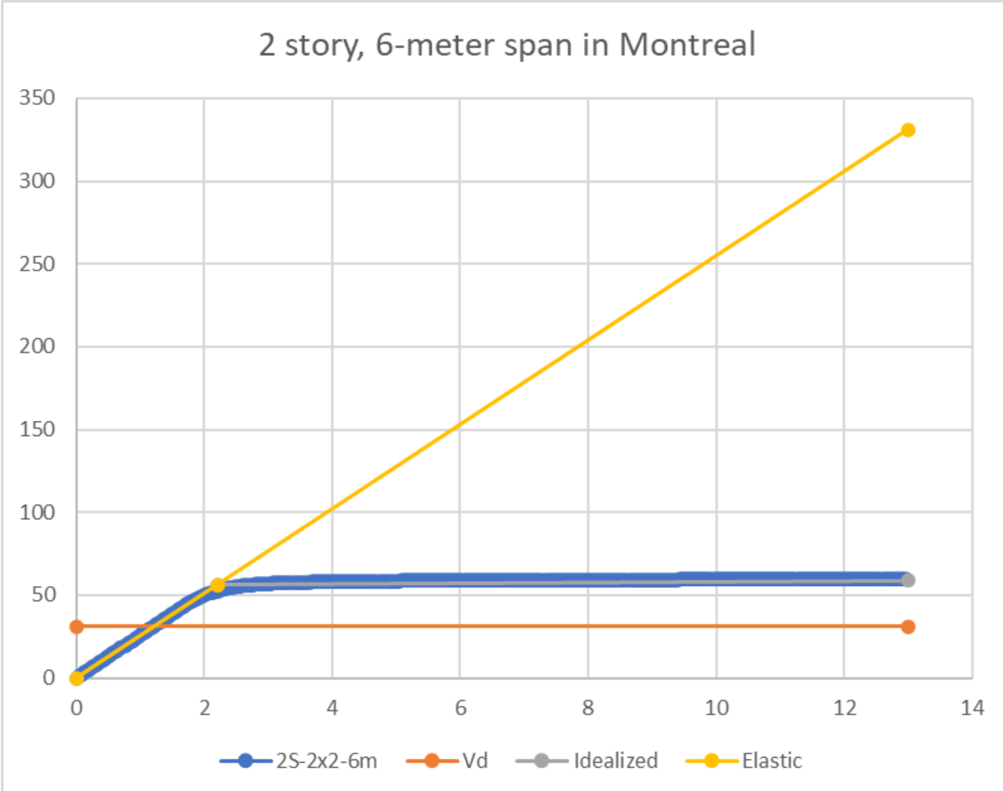
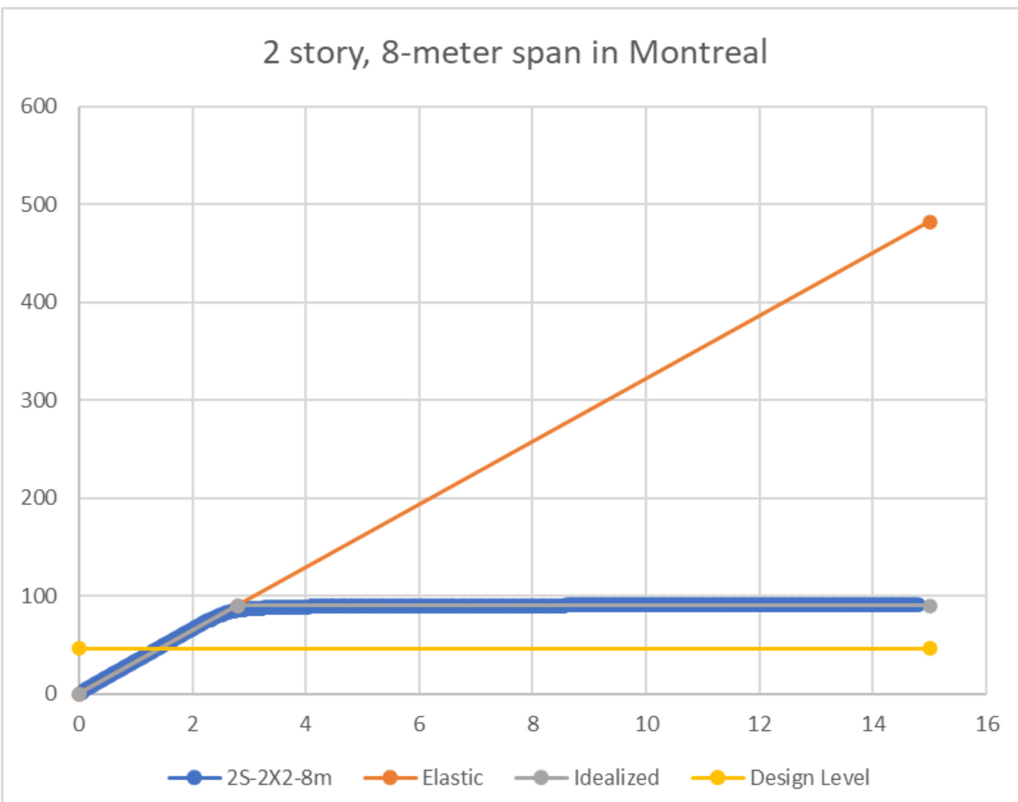
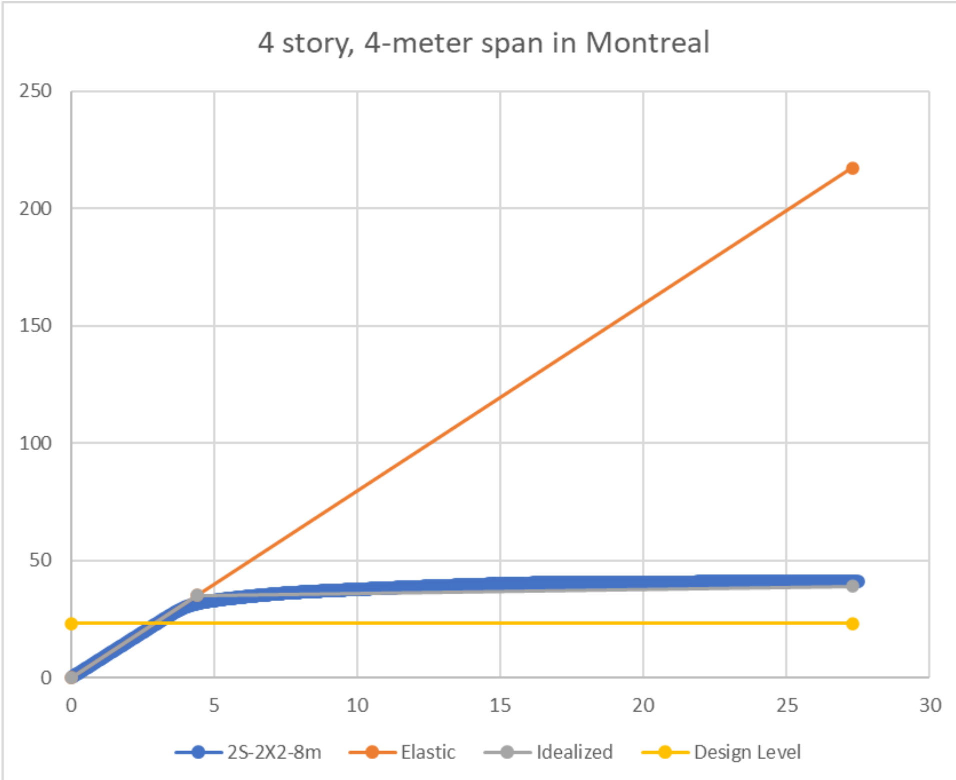
APPENDIX A- Detailed pushover curves for all models

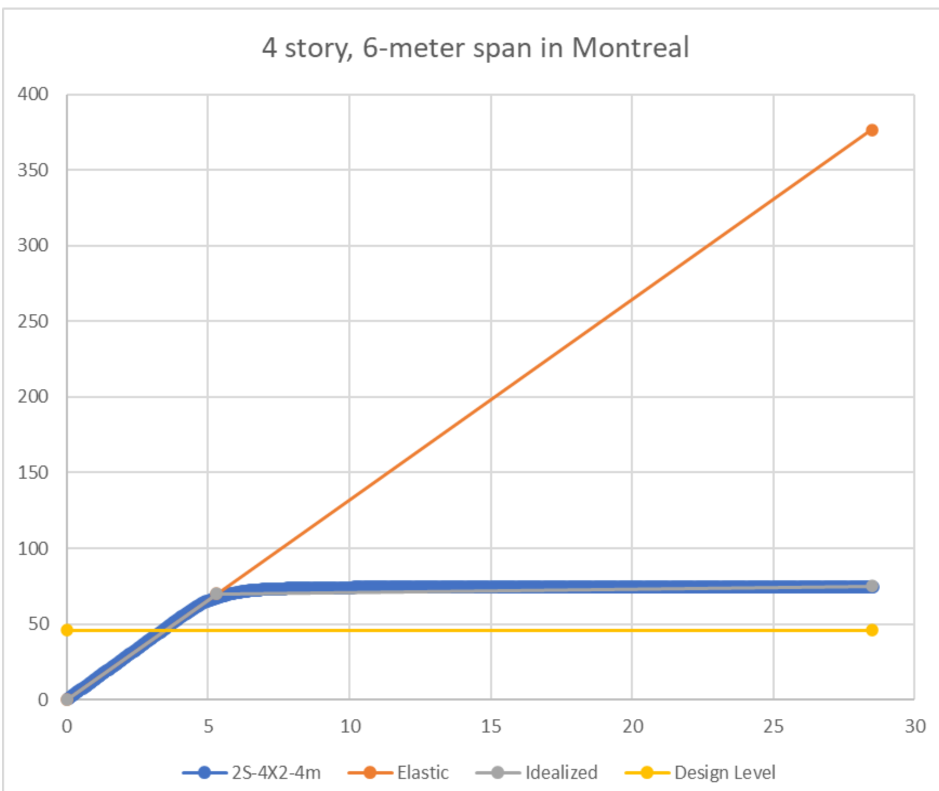
This section presents the comprehensive set of pushover curves derived from the conducted analyses, along with their corresponding bilinear regression fits.

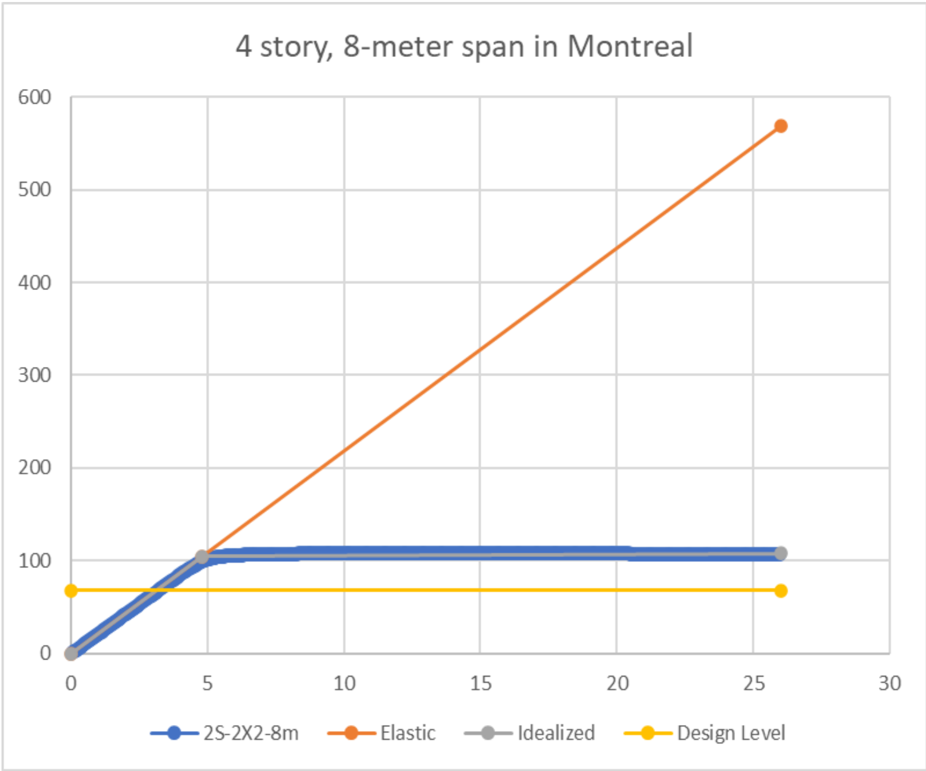


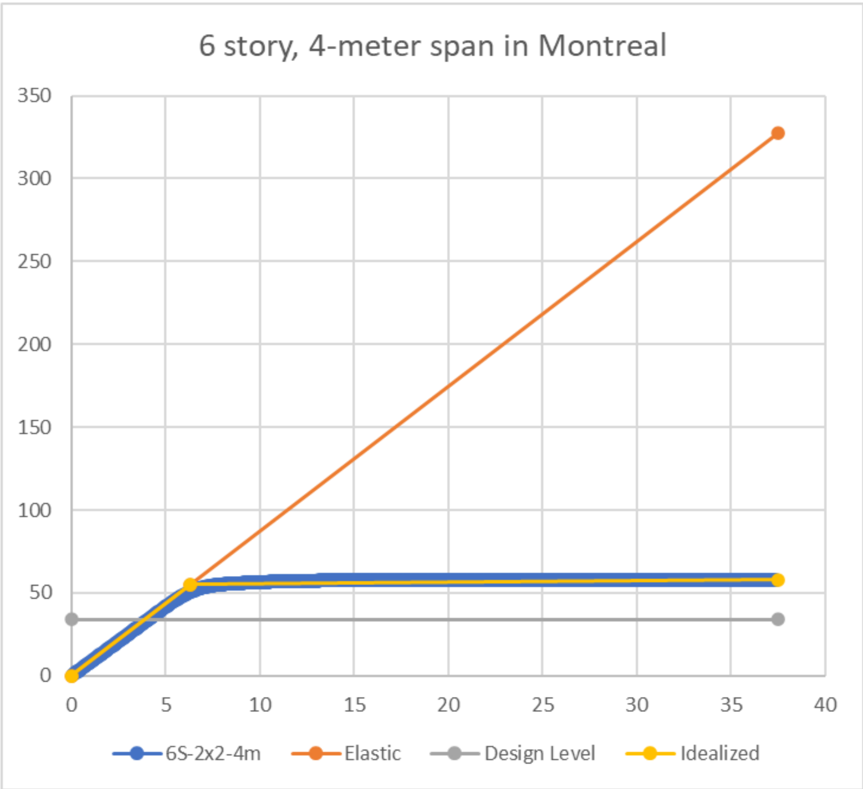

2 story, 6-meter span in Montreal
350
300
250
200
150
100
50
0
0
2
4
6
8
10
12
14
2S-2x2-6m
Vd
Idealized
Elastic

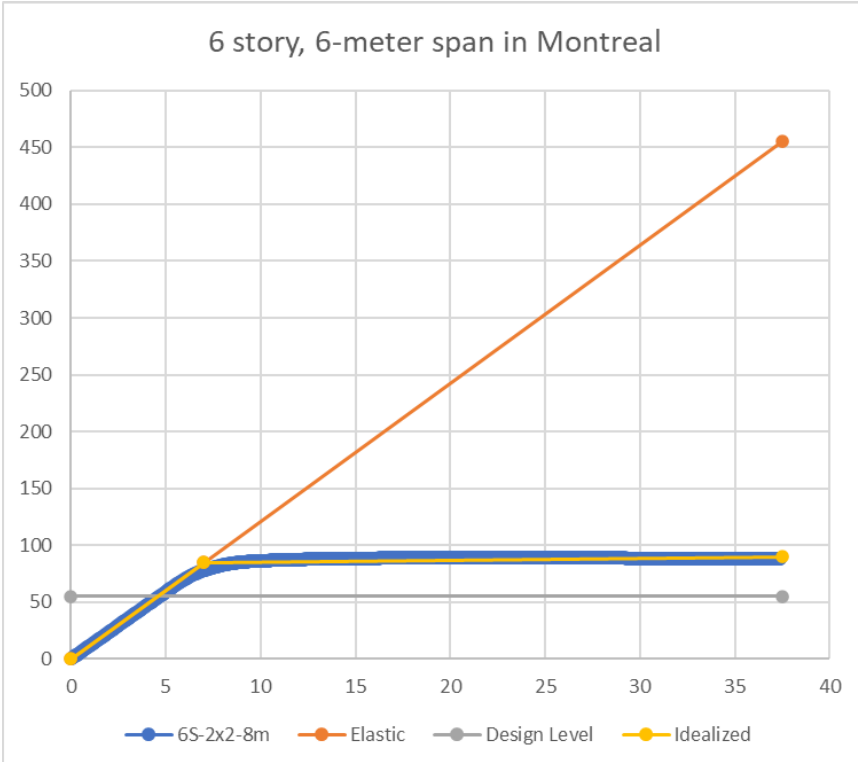

2 story, 8-meter span in Montreal
600
500
400
300
200
100
0
0
2
4
6
8
10
12
14
16
2S-2X2-8m
Elastic
Idealized
Design Level

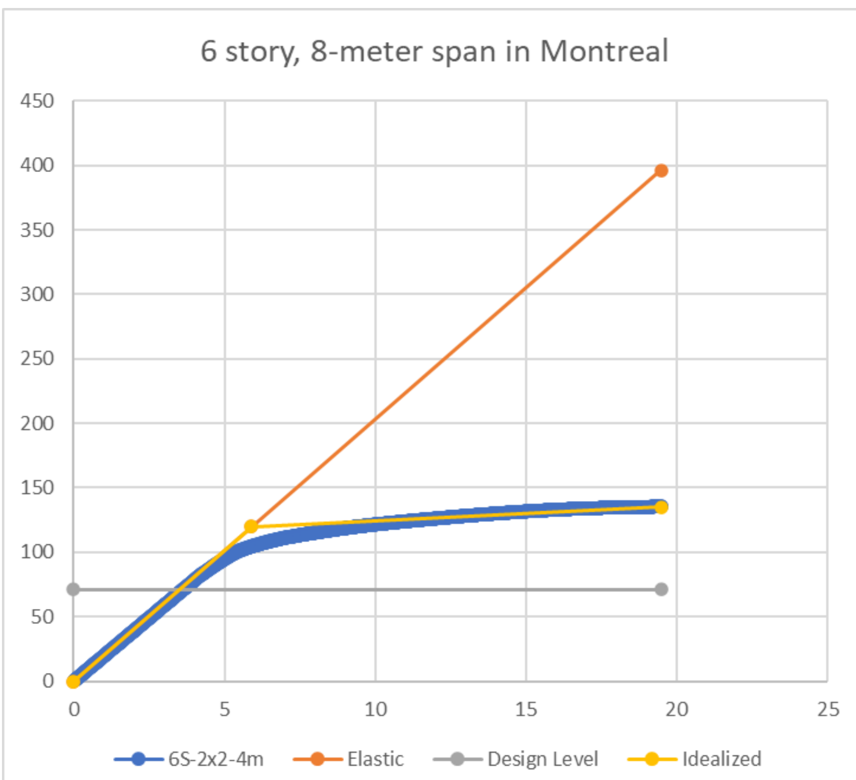

4 story, 4-meter span in Montreal
250
200
150
100
50
0
0
5
10
15
20
25
30
2S-2X2-8m
Elastic
Idealized
Design Level

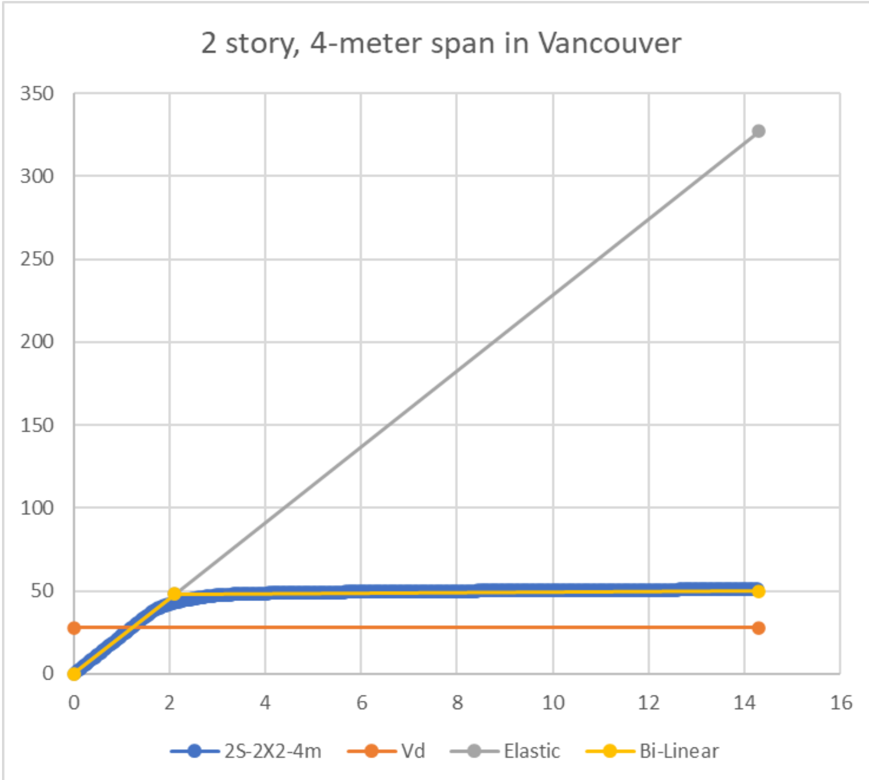

4 story, 6-meter span in Montreal
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
2S-4X2-4m
Elastic
Idealized
Design Level

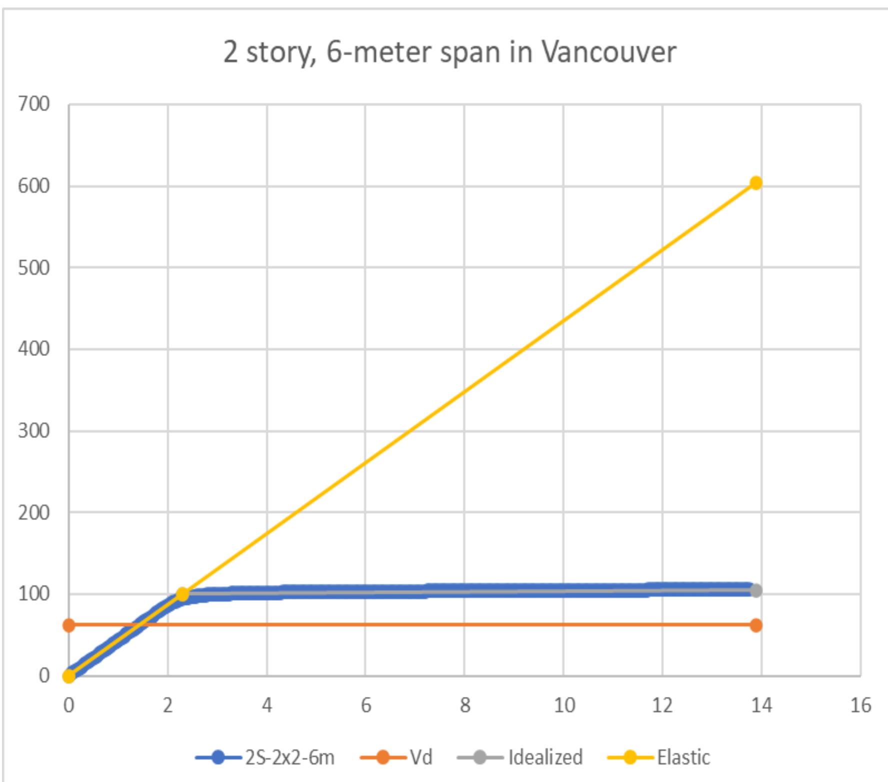

4 story, 8-meter span in Montreal
600
500
400
300
200
100
0
0
5
10
15
20
25
30
2S-2X2-8m
Elastic
Idealized
Design Level

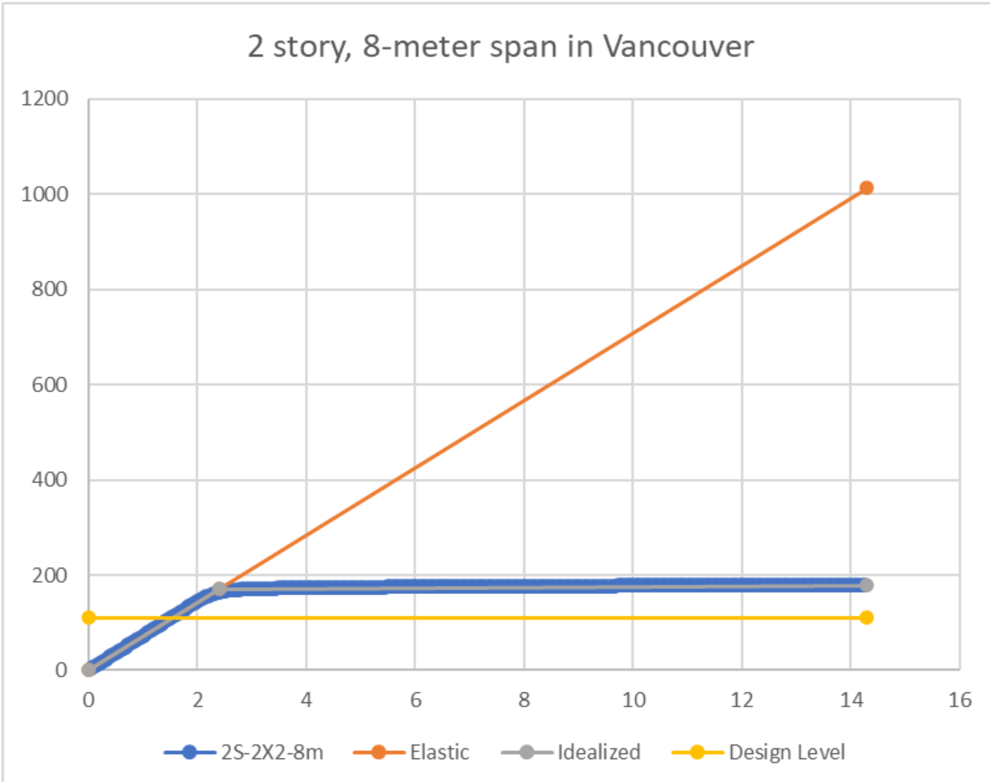

6 story, 4-meter span in Montreal
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
6S-2x2-4m
Elastic
Design Level
Idealized

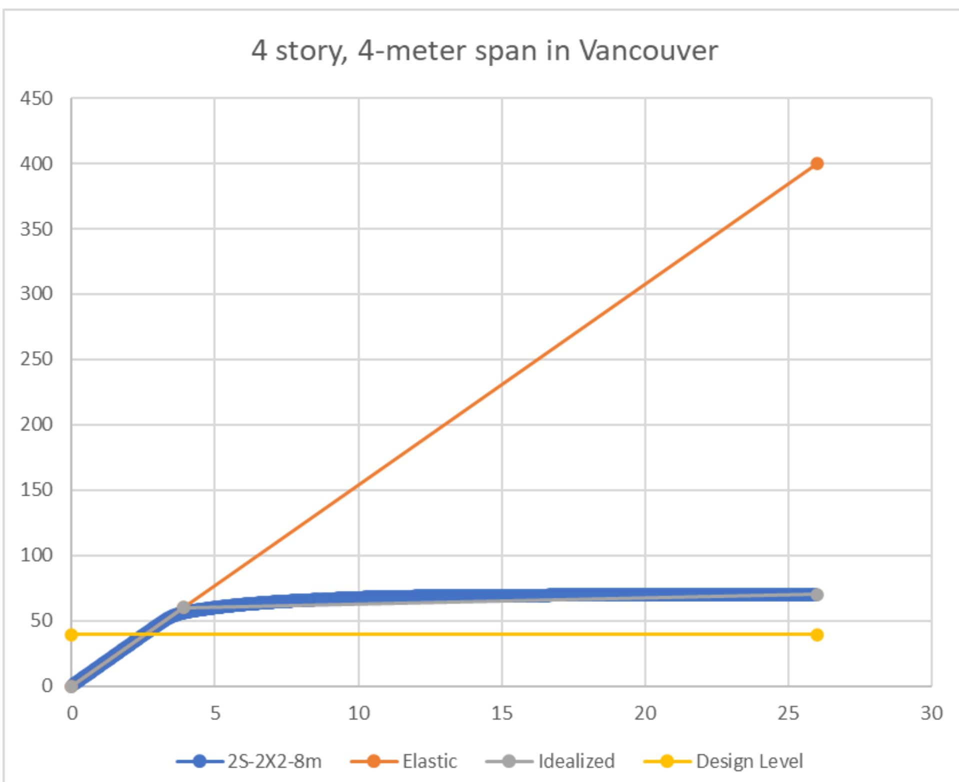

6 story, 6-meter span in Montreal
500
450
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
6S-2x2-8m
Elastic
Design Level
Idealized

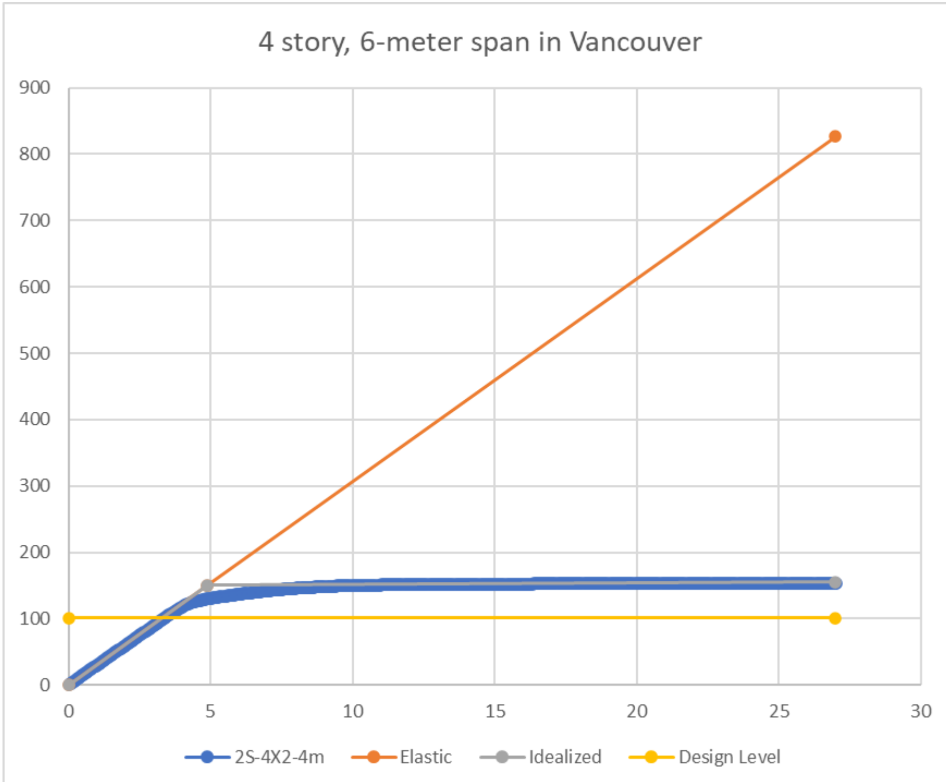

6 story, 8-meter span in Montreal
450
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
6S-2x2-4m
Elastic
Design Level
Idealized

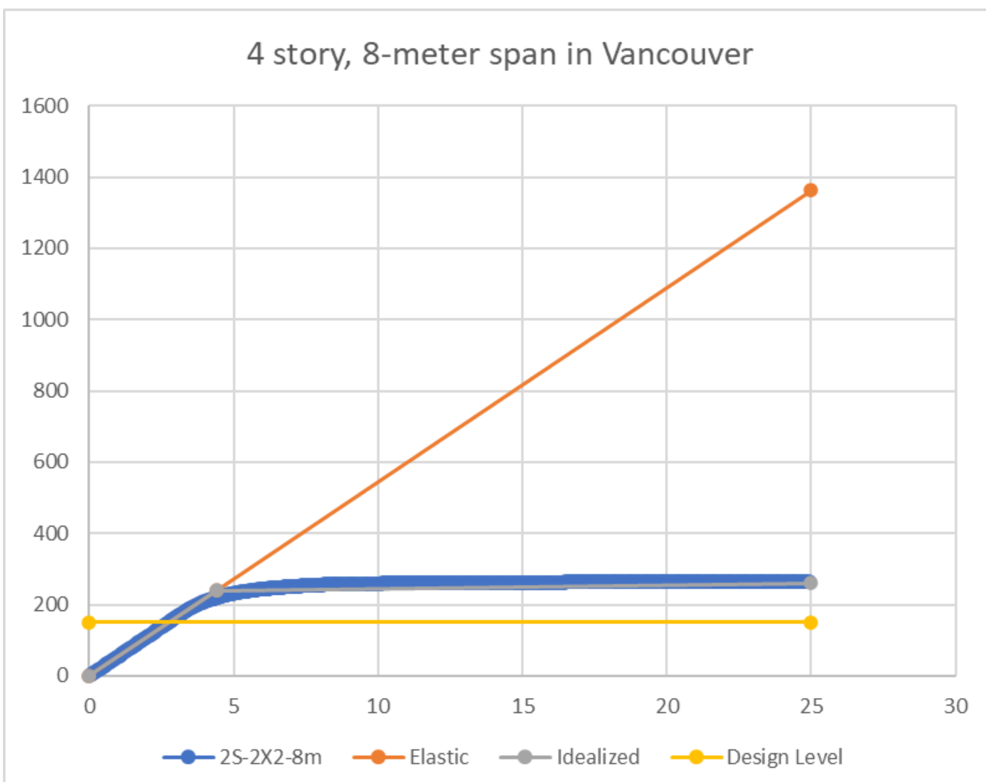

2 story, 4-meter span in Vancouver
350
300
250
200
150
100
50
0
0
2
4
6
8
10
12
14
16
2S-2X2-4m
Vd
Elastic
Bi-Linear

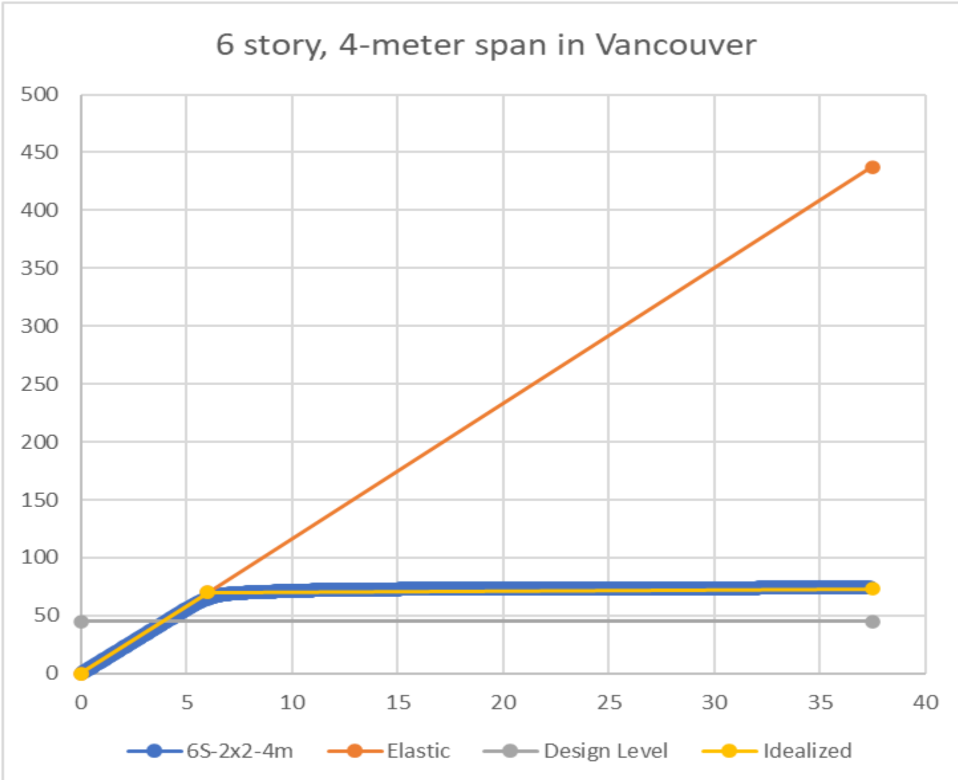

2 story, 6-meter span in Vancouver
700
600
500
400
300
200
100
0
0
2
4
6
8
10
12
14
16
2S-2x2-6m
Vd
Idealized
Elastic

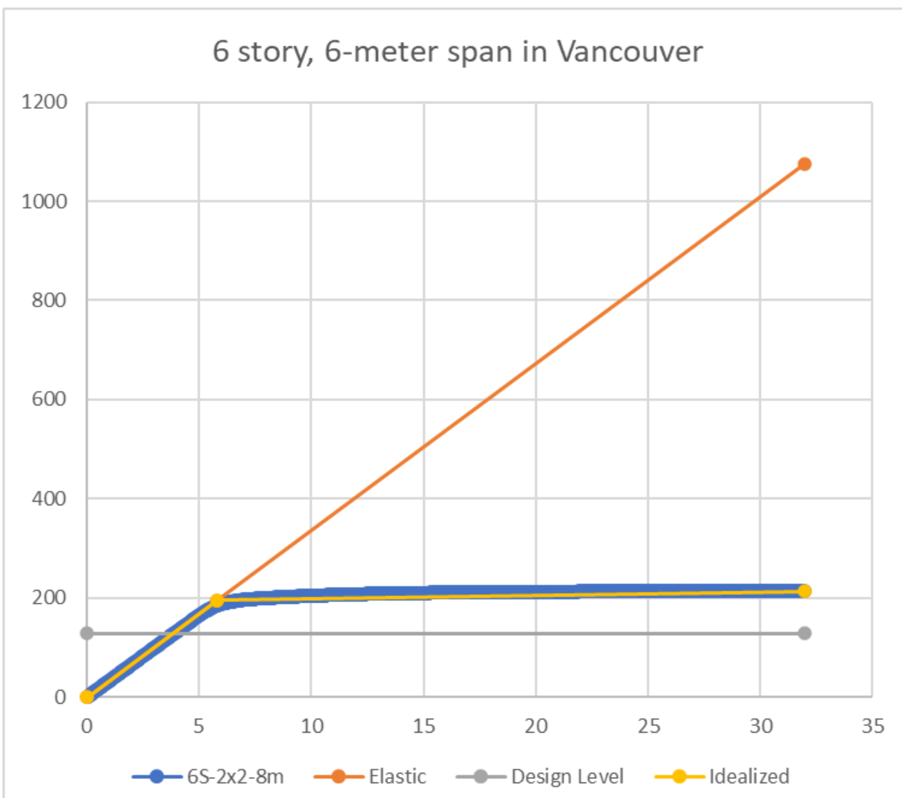

2 story, 8-meter span in Vancouver
1200
1000
800
600
400
200
0
0
2
4
6
8
10
12
14
16
2S-2X2-8m
Elastic
Idealized
Design Level

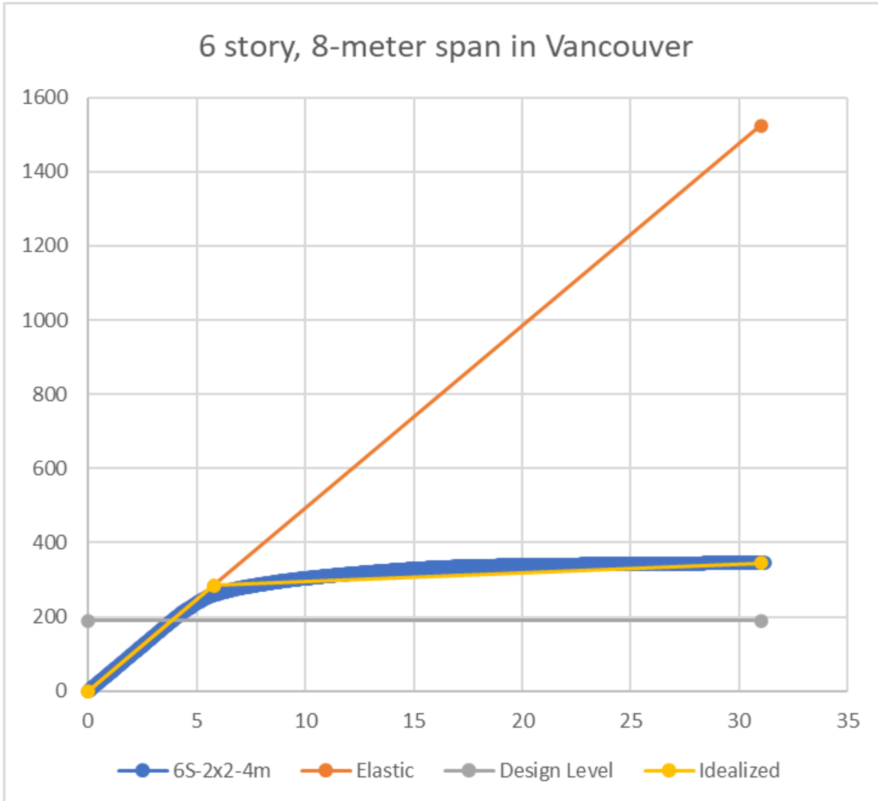

4 story, 4-meter span in Vancouver
450
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
2S-2X2-8m
Elastic
Idealized
Design Level


4 story, 6-meter span in Vancouver
900
800
700
600
500
400
300
200
100
0
0
5
10
15
20
25
30
2S-4X2-4m
Elastic
Idealized
Design Level


4 story, 8-meter span in Vancouver
1600
1400
1200
1000
800
600
400
200
0
0
5
10
15
20
25
30
2S-2X2-8m
Elastic
Idealized
Design Level


6 story, 4-meter span in Vancouver
500
450
400
350
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
6S-2x2-4m
Elastic
Design Level
Idealized


6 story, 6-meter span in Vancouver
1200
1000
800
600
400
200
0
0
5
10
15
20
25
30
35
6S-2x2-8m
Elastic
Design Level
Idealized


6 story, 8-meter span in Vancouver
1600
1400
1200
1000
800
600
400
200
0
0
5
10
15
20
25
30
35
6S-2x2-4m
Elastic
Design Level
Idealized