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SEISMIC PERFORMANCE EVALUATION OF TIMBER MOMENT FRAMES WITH REINFORCED DOWEL-TYPE CONNECTIONS

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ABSTRACT: This study evaluates the seismic performance of a two-storey timber moment-resisting frame (TMRF) used as the seismic force-resisting system (SFRS) in a building located in Vancouver, Canada. The analysis considers two connection configurations: unreinforced and self-tapping screw (STS)-reinforced bolted connections. A numerical model was developed in OpenSees and validated using published experimental data. Nonlinear time history analysis (NLTHA) was performed using ground motions (GMs) scaled from various tectonic regimes to assess the seismic response. Incremental dynamic analysis (IDA) was conducted to determine inter-storey drift ratios at different seismic intensity levels, while collapse fragility curves were determined for both connection types. The results reveal that reinforcing bolted connections increased the median collapse intensity and collapse margin ratio (*CMR*) by approximately 15%, demonstrating improved structural performance. Additionally, although the structure exceeded the allowable inter-storey drift limits outlined in NBCC 2020, reinforcing the timber moment connections with STSs reduced the roof-level drift ratio by 30%. These findings highlight the challenges of drift control in TMRFs while confirming that reinforcing dowel-type connections with STSs is an effective strategy for enhancing structural performance.

KEYWORDS: Timber moment resisting frame (TMRF), seismic collapse performance, dowel-type connections, self-tapping screws (STSs) reinforcement.

1 – INTRODUCTION

Approximately 40% of global energy-related emissions originate from the construction sector, a challenge exacerbated by the growing demand for medium- to high-rise buildings due to rapid urbanization [1]. Consequently, there is a pressing need for sustainable materials in these structures. Mass timber buildings are increasingly favoured due to their high strength-to-mass ratio, the carbon-sequestering properties of wood, and their aesthetic appeal compared conventional materials such as steel and concrete. However, the inherent heterogeneity of wood, along with natural defects, such as knots and grain slope, can impact structural performance. To address these challenges and meet the demand for larger structural sections, modern mass timber buildings commonly utilize engineered wood products such as glued-laminated timber (glulam), structural composite lumber like laminated veneer lumber (LVL), and mass timber panels like cross-laminated timber (CLT)

In this context, timber moment-resisting frames (TMRFs), which incorporate semi-rigid beam-to-column connections, are favoured due to their architectural flexibility compared to shear walls or braced systems. However, no design guidelines or technical documents currently exist for achieving certain level of system ductility in TMRF, using available timber moment connections tests. Among the available semi-rigid connections, glued-in rods and slotted-in steel plates with dowel-type fasteners are commonly used. However, glued-in rod connections are less desirable due to challenges in gluing quality control, bonding reliability, and long-term durability. On the other hand, dowel-type connections are preferred for their ease of fabrication, superior fire resistance compared to connections with exposed metal assemblies, and aesthetically pleasing appearance. However, the low rotational stiffness, and limited bending moment resistance make dowel-type connections less suitable option for designers as moment connections in mass timber buildings [2,3]. One of the most critical issues with slotted-in steel bolted connections, a type of dowel-type connection, is initial slippage caused by oversized holes, as well as premature failure modes due to tension perpendicular to the grain and longitudinal shear induced by fasteners—two of the weakest properties of wood products [4].

One of the most effective techniques to address these issues is the reinforcement of the dowel-type joints with self-tapping screws (STSs). Fig. 1(a) and (b) illustrate

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examples of an unreinforced and an STS-reinforced bolted beam-to-column connection, respectively. Lam et al. [5] found that reinforcing joints with STSs inserted perpendicular to the grain resulted in a 75% increase in moment capacity and a fourfold increase in rotational capacity.

This paper aims to establish a systematic approach for evaluating the seismic performance of TMRFs as a sustainable all-timber seismic force-resisting system (SFRS). Developing verified numerical models is a crucial step in this process, as conducting experimental tests on multiple prototypes is highly time-consuming and costly. Initially, the study will rely on existing data from available tests to develop and validate a simplified numerical model of a two-storey timber frame with semi-rigid bolted beam-to-column connections, a type of dowel-type connection, considering both unreinforced and STS-reinforced scenarios. The effect of reinforcement on seismic collapse performance and inter-storey drift ratios will also be examined. However, as the study progresses, additional testing may be required to further refine the model and enhance its accuracy and reliability.

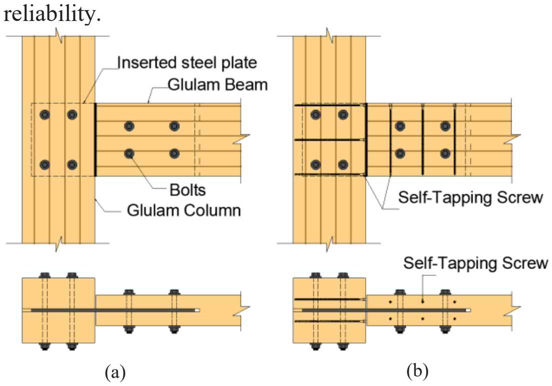


Figure 1. *Elevation and top views of bolted Connection: (a) Unreinforced, (b) Reinforced by STSs*

2 – BACKGROUND

2.1 INTEGRARTION OF TMRF IN THE CODE

Unlike steel or reinforced concrete, for which reliable empirical and analytical formulations have been developed to determine the moment-resisting capacity of joints, there is currently no reliable or direct method for assessing the capacity of timber moment connections. TMRFs have not been recognised in European code Eurocode 8 [6] or U.S. standard ASCE/SEI 7-22 [7]. The 2020 National Building Code of Canada (NBCC) [8] specifies ductility-related force modification factors (R_d) and overstrength-related force modification factors (R_o) for different SFRSs. Two different ductility categories of TMRFs are considered in the 2020 NBCC—moderate and limited ductility—similar to those for timber-braced frames (TBF). Moderately ductile TMRF are assigned values of 2.0 and 1.5 for R_d and R_o, respectively, while limited ductile frames have a value of 1.5 for both R_d and R_o. Although TMRFs are included in Part 4 of the NBCC, the Canadian standard for engineering design in wood, CSA O86-24 [9], does not provide design guidelines for achieving these ductility levels, nor does it specify appropriate connection details to ensure the required ductility.

2.2 REINFORCEMENT TECHNIQUES

Various approaches have been employed to enhance the performance of dowel-type connections carrying bending moment under both cyclic and monotonic loads. For example, glass fibre-reinforced polymer (GFRP) [10] have been used to increase the load-carrying capacity of dowel-type connections. To address low initial stiffness, techniques such as using steel tubes with resin injection [11] and pre-stressed tube bolted connections [12] have been explored. Additionally, improving the wood material itself is another approach to preventing premature brittle failures. Leijten et al. [13] achieved this by using densified veneer wood (DVW) or locally cross-laminating timber [14], both of which improved the ultimate capacity of the joints. Although all of these techniques are effective in increasing the ultimate capacity of dowel-type joints, they are complex, time-consuming, and costly to implement. Reinforcing dowel-type moment connections by STSs has gained popularity due to the ease of installation and handling. Table 1 summarises previous studies conducted on the effect of reinforcement by STSs in increasing moment capacity and ultimate rotation. All tests demonstrate the effectiveness of STSs in increasing the ultimate capacity, and preventing the tendency of wood to experience premature failures, such as splitting and plug shear.

Table 1. Summary of the past studies on dowel-type moment connections reinforced by STSs

Tests	Increase in peak moment (%)	Increase in ultimate rotation (%)
Wang et al. [14]	73	260
He et al. [15]	60	320
Zhang et al. [16]	30	50
Petrycki et al. [17]	50	70
Dong et al. [18]	14	240

2.3 NUMERICAL MODELS

Several efforts have been made to develop numerical models of dowel-type connections in previous studies. Shu et al. [19] developed a finite element model of self-centering moment-resisting joints using OpenSees software [20] and evaluated the seismic performance of a three-storey building with specified beam-to-column connections. A similar approach was adopted by Tao et al. [21] for timber-steel hybrid beam-to-column joints and by Li et al. [22] for multi-storey glulam post-and-beam structures reinforced with knee braces. Cao et al. [23] developed a simplified calibrated model for a portal timber frame with bolted connections using the 'Zero Length' element with the 'Pinching4' uniaxial material model. However, no comprehensive study has been conducted to evaluate the seismic performance of multi-storey dowel-type moment connections through the development of numerical modelling.

3 – DESIGN DETAILS AND MODEL DESCRIPTION

3.1 DETAILS OF THE BUILDING

To achieve the research objectives, a two-storey residential building with bolted connections, considering both unreinforced and STS-reinforced scenarios, was designed in accordance with the NBCC 2020 and CSA O86-24. The structure, as illustrated in Fig. 3(a) and (b), is located in Vancouver, British Columbia, Canada (49° 15' 39.6" N, 123° 6' 50.4" W). The site has an average shear wave velocity to a 30 m depth (Vs_{30}=450 m/s), corresponding to Site Class C. The beams and columns are made of Canadian spruce-pine-fir (SPF) 20f-E glued-laminated timber. The frame structure, with a span-to-height ratio of 1.5, had a span of 4110 mm and a height of 2740 mm. The geometric sections of the columns and beams were 280 mm × 230 mm and 280 mm × 180 mm, respectively. To connect the beams and columns, both the beam-column and column-base joints were bolted (14 mm bolts Grade 8.8) with slotted steel plates with thickness of 10 mm, specified according to experimental properties tested at Tongji University, China [24]. Fig. 2 provides an overview of the tested timber frame with bolted connections.

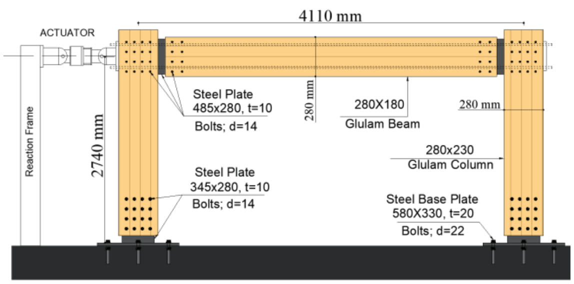
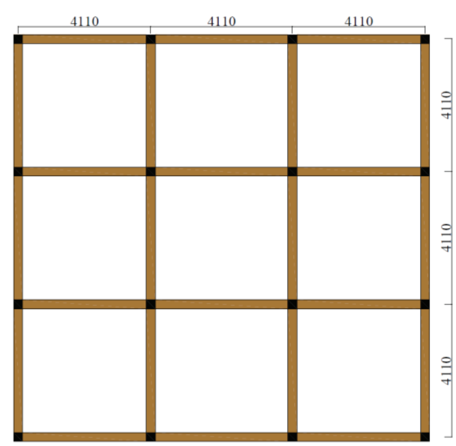


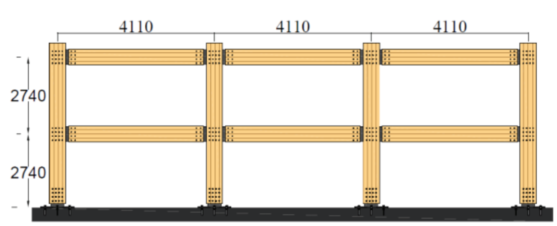
Figure 2. *Information of the tested timber frame (unit: mm)* [23,24]

In real-world practice, particularly in steel moment-resisting frame buildings, it is neither economical nor practical to implement SFRSs in all frames throughout the structure. Instead, interior frames are typically assumed to have pinned connections, allowing them to carry only gravity loads. However, in this study, all frames—four frames per direction, each with three bays in both orthogonal directions—are assumed to be moment-resisting. It is primarily due to, first, in timber structures, bolted connections are commonly considered as beam-to-column connections; Second, ensuring sufficient strength and stiffness to resist lateral seismic actions, as equipping only the perimeter frames with TMRFs is insufficient to withstand seismic lateral forces.

In addition, since the developed numerical model for seismic evaluation is validated against a system-level experimental test, the study is limited to two-storey structures. This limitation arises from the fact that bolted connections, which have been experimentally tested, lack sufficient moment capacity to support higher-storey buildings under the specified loads and site conditions. Furthermore, there is limited reliable data in the literature on moment connections with higher moment capacities, which is essential for accurate numerical model calibration. Given this data gap, developing archetypes with more stories requires further research efforts in future studies, both at the component level to establish scalable moment connections and at the system level through experimental tests.



(a)



(b)

Figure 3. *(a) Plan and (b) elevation view of the TMRF building under study (unit: mm)*

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3.2 DESIGN LOADS

Table 2 summarises the gravity loads considered in accordance with NBCC 2020. In addition, the seismic spectra used for the preliminary design of the building were obtained from the online seismic hazard tool outlined in NBCC2020. This tool provides uniform hazard spectral (UHS) acceleration values at ten discrete periods, necessitating interpolation if the building's fundamental period falls in between. Fig. 4 illustrates these spectral acceleration values for Vancouver at the 2% probability (design level) and 10% probability in a 50-year return period.

Table 2. *Summary of the gravity loads*

Load type	Load (kN/m^2)
Floor dead load	2.40
Floor live load	1.90
Partition load	0.50
Snow load	1.64
Roof dead & live load	1.0

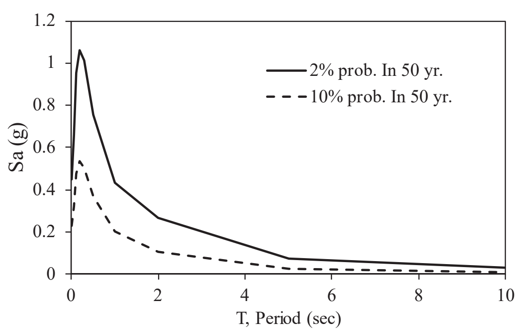

Figure 4. *UHS of Vancouver on soil with Vs_{30}=450 m/s* [8]

3.3 DESIGN OF THE TMRF BUILDING

A preliminary design of the two-storey building with unreinforced bolted connections with limited ductility as specified in Section 3.1, was carried out by obtaining the demands via load combinations, with the analysis conducted in SAP2000 software [25]. A 3D structural model of the building was developed and analysed, as shown in Fig. 5. The load combinations used for limit state verification of both members and connections were in accordance with Part 4 of NBCC 2020.

The floor diaphragms were assumed to be rigid, with the seismic weight distributed uniformly across the floor. Seismic forces were considered by defining the response spectra for the specified location, and the seismic analysis was performed using dynamic analysis. The adequacy of the shear, axial, and bending moment capacities of the structural elements was verified in accordance with Section 7 of CSA O86-24. Additionally, the assessment of bolted connections was carried out following the provisions outlined in Section 12.4 of the same standard.

Adequacy of the shear resistance of the semi-rigid beam-to-column connections was evaluated by considering the yielding lateral resistance for three members (wood-steel-wood). The resistance is taken as the minimum of four limit states, as defined in Equations (1) to (4). The parameters f_1 and f_2 represent the embedment strength of the side and main members, respectively, in accordance with Clause 12.4.4.3.3 of CSA O86-24. The parameters t_1 and t_2 correspond to the thickness of the side and main members, while f_y and d_f denote the yield strength and diameter of the fastener, respectively, which was assumed to be an ASTM A325 bolt. Since the bending moment capacity of the connections has not been incorporated into any existing code, the corresponding limit state was verified based on test results, as detailed in [23].

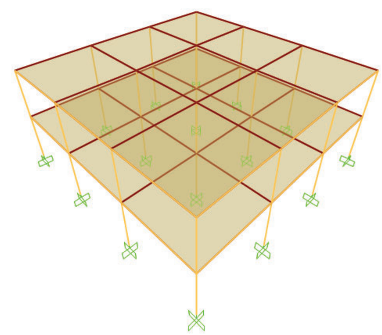

Figure 5. *3D model of TMRF building in SAP2000*

$$f_1 d_f t_1 \tag{1}$$

$$\frac{1}{2} f_2 d_f t_2 \tag{2}$$

$$f_1 d_f^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{f_1+f_2} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_f} \right) \tag{3}$$

$$f_1 d_f^2 \sqrt{\frac{2}{3} \frac{f_2}{f_1+f_2} \frac{f_y}{f_1}} \tag{4}$$

3.4 NUMERICAL MODEL

Since the building is symmetrical in both orthogonal directions, X and Y, a 2D nonlinear model of the TMRF building was developed in OpenSees software, as shown in Fig. 6. The mass source was considered as 1.0× dead load+ 0.5× live load or 0.25 × snow load, whichever was greater. The first modal period of the building with unreinforced connections in SAP2000 and OpenSees was 0.68 s and 0.95s, respectively, highlighting the significant contribution of connection stiffness to the overall system stiffness in TMRFs. In SAP2000, connections were assumed to be fully rigid, whereas in OpenSees, the actual stiffness of the semi-rigid connections was incorporated into the model.

Timber glulam beam and column elements were modelled using the Elastic Beam-Column element, while the nonlinear behaviour of the connections was represented by nonlinear springs modelled with 'zero-length' elements. The 'Pinching4' material model was considered as uniaxial material, with its parameters determined through an optimisation process to match the hysteresis response of the nonlinear model with experimental test results, as reported in [23]. Additionally, leaning columns with co-rotational truss elements were included to capture the P–Δ effects during dynamic loading. These leaning columns were modelled with high axial stiffness and negligible lateral stiffness.

However, the referenced test did not evaluate the same system with reinforced connections. Based on studies in the literature, summarised in Table 1, the peak moment and ultimate rotation were found to increase by factors of 1.5 and 2.0, respectively.

The verification of the simulated numerical model for the beam-to-column connection and the overall timber frame (one-bay, one-storey) is demonstrated through both hysteresis and envelope curves, as shown in Fig. 7 (a) and (b), respectively.

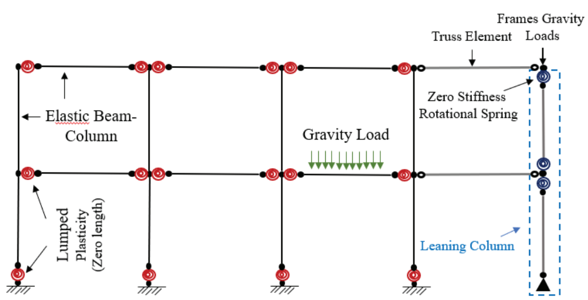

Figure 6. *Nonlinear analytical model of TMRF in OpenSees*

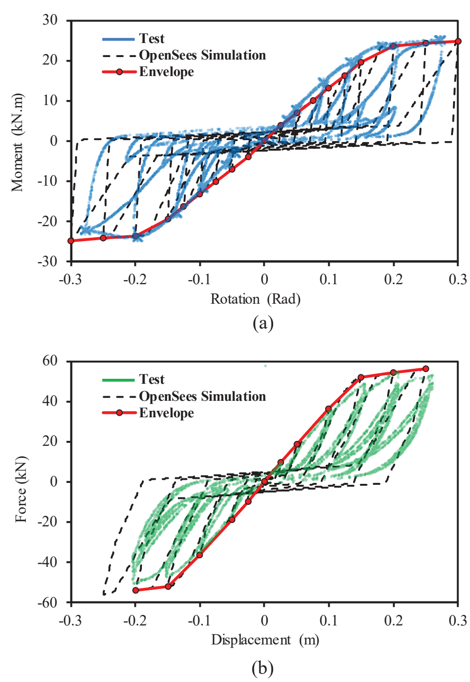

Figure 7. *OpenSees nonlinear model verification for (a) beam-to-column connection, and (b) timber frame*

3.5 GROUND MOTIONS (GMs) SCALING

To achieve a reliable probabilistic collapse analysis, a sufficient number of GMs must be considered for nonlinear time history analysis (NLTHA). The west coast of Canada has a complex tectonic regime, with three primary seismic sources—crustal, in-slab, and interface—significantly contributing to seismic hazards in Vancouver, BC [26].

A total of 33 GMs were selected, with 11 from each seismic source. The crustal earthquake records were obtained from the PEER NGA West-1 database [27], while the PEER preliminary NGA-Sub databases [28] provided the subduction in-slab and interface records. GMs were selected and scaled following the method A outlined in Commentary J of NBCC 2015 [29]. Based on this methodology, the records should be scaled to target spectra (S_T) within the period range of interest (T_R), which lies between the minimum period (T_{min}) and the maximum period (T_{max}). The lower bound, T_{min}, is defined as the minimum of 0.15 times the first-mode period or the period corresponding to 90% mass participation. The upper bound, T_{max}, is taken as the greater of either twice the first-mode period or 1.5 s. In this study, the period of interest ranges between 0.14 s and 1.9 s. Additionally, it is crucial to scale the records of each seismic source within its scenario-specific period range (T_{RS}). Fig. 8 illustrates the scaling of GMs for each specific seismic source, along with the average of all GMs, demonstrating their alignment with the NBCC 2020 design spectra.

3.6 COLLAPSE FRAGILITY ASSESSMENT

To obtain the collapse fragility of the building, incremental dynamic analysis (IDA) was performed using the algorithm outlined by Vamvatsikos et al. [30], where GMs were scaled up until structural collapse occurred. To capture collapse, non-simulated collapse mechanisms were investigated, accounting for the drift capacity of the frame system. Sarti et al. [31] conducted a parametric study and found that a roof drift of 5% is associated with the collapse criterion in timber frames. Additionally, the collapse margin ratio (*CMR*) is another key parameter, which can be determined using Equation (5), where S_{CT} represents the median collapse intensity (50% probability) and S_{MT} denotes the spectral acceleration at the maximum considered earthquake (MCE) level. Collapse fragility can be assessed by extracting all collapse points from the IDA and fitting the data to a lognormal distribution.

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{5}$$

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4 – RESULTS AND DISCUSSION

For the IDA curves, a total of 560 NLTHA were performed. The maximum inter-storey drift ratio (IDR_{max}) was considered as the damage index of the building, while the first-mode elastic pseudo-acceleration (Sa_{T1}) served as the intensity measure. Fig. 9(a) and (b) present the median, as well as the 16^{th} and 84^{th} percentiles, of the IDA curves for the TMRF building with both unreinforced and reinforced bolted connections, respectively. As expected, the TMRF with reinforced bolted connections reached the collapse point at higher intensity values compared to the unreinforced connections. This is because reinforcing the connections significantly increases their stiffness, ductility, and ultimate deformation capacity thereby controlling inter-storey drift in moment-resisting frame systems. The increased ductility also delays connection failure in the post-yield stage, enabling the structure to withstand higher spectral accelerations before reaching the target drift and collapse threshold.

Fig. 10 illustrates the collapse fragility curves of the building for both scenarios—unreinforced bolted connections and connections reinforced with STSs. Reinforcing the connections with STSs increased the median collapse intensity and *CMR* by approximately 15%. This enhancement can assist the system in meeting the performance evaluation criteria outlined in FEMA P-695 [32], which assesses whether the system provides the required ductility level. To comprehensively evaluate system performance, the design provisions for timber moment connections must first be developed. Subsequently, various archetypes with different ductility levels, seismic categories, and fundamental period ranges (from low-rise to mid- or high-rise structures) are needed. However, the available tests in the literature do not sufficiently cover these parameters, necessitating further experimental studies to expand the database of timber moment connections across different strength and ductility levels.

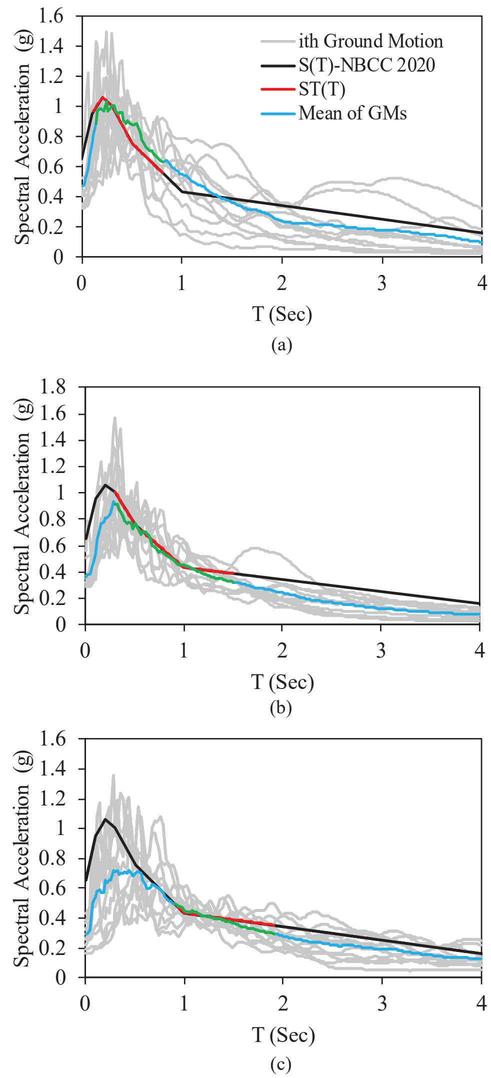


Figure 8. *Scaling of GM records (a) Crustal T_{RS}=0.14s-0.8s, (b) In-Slab T_{RS}=0.3s-1.5s, (c) Interface T_{RS}=0.9s-1.9s*

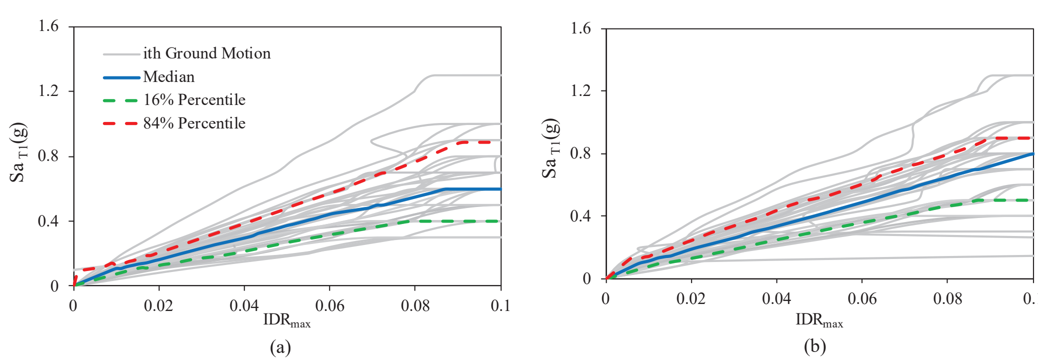


Figure 9. *IDA curves of the building with (a) unreinforced, (b) reinforced bolted connections*

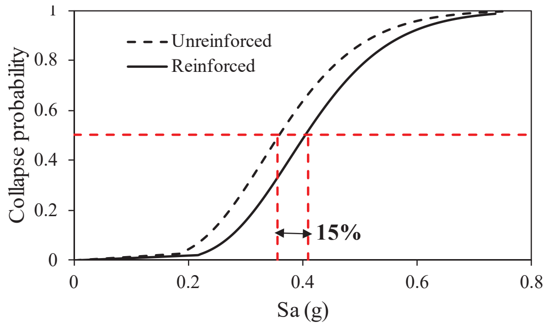
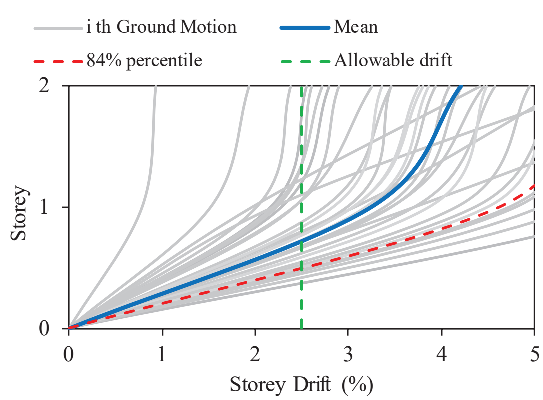


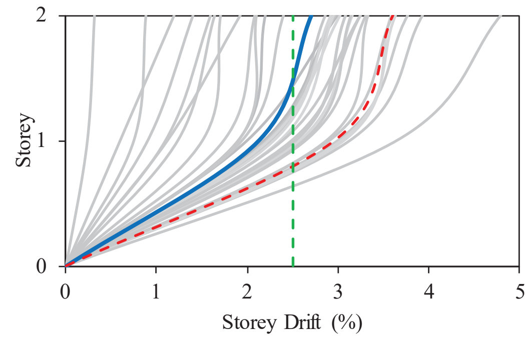
***Figure 10.**, Collapse fragility curves of the building with reinforced and unreinforced bolted connections*

Fig. 11(a) and (b) present the median and 84th percentile storey drift ratio (%) at the MCE level for the structure with unreinforced and reinforced connections, respectively. It is evident that in both connection scenarios, the building fails to meet the 2.5% maximum inter-storey drift limit specified in NBCC 2020 for a 2475-year return period event. This finding highlights the susceptibility of moment-resisting frames in controlling the allowable drift ratio, even when the capacity of the members and connections remains within the allowable range. However, the results also clearly demonstrate that reinforcing the connections significantly reduces the maximum inter-storey drift of the structure. The analysis shows that roof drift decreased by approximately 30% with connection reinforcement. This emphasises the impact of reinforcing dowel-type connections in TMRFs, demonstrating an increase in both connection and system stiffness while maintaining overall system ductility.

Overall, TMRFs with dowel-type connections are prone to premature failures and low rotational stiffness, making drift and collapse control challenging. Reinforcing with STSs is an effective, economical, and practical solution to address these issues. Improving seismic performance by reinforcing the moment connections can help mitigate the substantial economic, social, and environmental costs associated with structural collapse of TMRFs. This supports the development of more sustainable mass timber buildings with moment-resisting frames.



(a)



(b)

***Figure 11.** Storey drift responses of TMRF at MCE level with (a) unreinforced and (b) reinforced connections*

5 – CONCLUSION

This study examines the impact of reinforcing timber moment connections with dowel-type fasteners on seismic collapse performance. It aims to establish a systematic approach to evaluating the seismic response of a two-storey building with a TMRF as the primary structural system for resisting lateral seismic forces. Located in Vancouver, Canada, the analysis was conducted for two specific scenarios: unreinforced and STS-reinforced bolted connections, as a type of dowel-type bema-to-column connections. A numerical analytical model of the structure was developed in OpenSees and validated using data available in the literature. After scaling GMs from different tectonic regimes, NLTHA was conducted to determine the structural responses.

The peak inter-storey drift ratios at different levels of spectral acceleration, used as the intensity measure, as well as collapse fragility curves, were obtained via IDA. The results indicate that reinforcing the bolted connections increased the median collapse intensity and collapse margin ratio *CMR* by approximately 15%, highlighting the effectiveness of reinforcement in enhancing the system performance in TMRFs.

Furthermore, the drift ratios at different storey levels were assessed at the MCE level, revealing that the structure exceeded the allowable drift limits outlined in NBCC 2020. However, reinforcing the moment connections with STSs significantly reduced the drift ratio, with a 30% decrease observed at the roof level. This finding underscores the susceptibility of TMRFs to drift control challenges but also demonstrates that reinforcing dowel-type connections with STSs is an effective approach to addressing this issue.

This study demonstrates that reinforcing timber moment connections with dowel-type fasteners using STSs significantly reduces collapse probability and improves drift control, addressing a key challenge in moment-resisting frames. The improved performance is primarily due to the increase in stiffness, ductility, and ultimate deformation capacity, which effectively limits inter-storey drift in TMRFs. The enhanced ductility delays

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connection failure in the post-yield stage, enabling the structure to withstand higher spectral accelerations before reaching the target drift limit and collapse. Consequently, this technique helps mitigate potential economic, social, and environmental losses, contributing to the advancement of sustainable mass timber buildings while establishing moment-resisting systems as a viable SFRS.

Although this study provided an example of how reinforcing dowel-type connections in TMRFs can enhance the collapse performance of a structure, further research is required to assess system performance across various archetypes with different ductility levels, as outlined in NBCC 2020. This necessitates experimental testing and the development of design provisions at both the connection and system levels to expand the database of timber moment connections across different strength, stiffness, and ductility levels. Furthermore, by developing the calibrated numerical models and following the FEMA P-695 procedure, the system force modification factors (R_d and R_o) can be evaluated, providing further insight into the seismic design of TMRFs.

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