WORLD CONFERENCE ON TIMBER ENGINEERING 2025
BRISBANE, AUSTRALIA
Advancing Timber for the Future Built Environment

SEISMIC RESPONSE OF BRACED TIMBER FRAMES WITH BOLTED CONNECTIONS

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ABSTRACT: Braced timber frames (BTFs) are one of the most efficient structural systems to resist lateral loads induced by earthquakes or winds. In the National Building Code (NBC) of Canada, BTFs are included as a seismic force resisting system (SFRS) with two ductility categories and corresponding *R*-factors. No design guidelines for BTFs, however, currently exist in CSA O86, the Canadian Standard for Engineering Design in Wood, making the system out of reach of the average designer. To remedy the situation, FPInnovations is leading a multi-year research project to determine the seismic behaviour of BTFs as a SFRS and generate the technical information needed for development of design guidelines for BTFs in CSA O86. This paper presents results from a study on the seismic response of BTFs with bolted connections located in Montreal, Canada. Typical archetypes were designed following the proposed design provisions, including capacity design and column tree design methodology. Nonlinear finite element models were developed with Pinching4 for bolted connections at the ends of diagonal braces on OpenSees. Incremental dynamic analysis was conducted, with 11 far-field ground motions that were selected and scaled to a Montreal spectrum, to investigate the seismic response of designed BTF archetypes. The seismic performance of the investigated archetypes was evaluated following the CCMC methodology.

KEYWORDS: timber structures, braced frames, R-factors, seismic performance, nonlinear dynamic analyses

1 – INTRODUCTION

Braced timber frames (BTFs), see Figure 1, are efficient structural systems to resist lateral loads induced by earthquakes or winds. Due to their inherent high stiffness properties, they have been used as a Lateral Load Resisting System in many mass timber buildings. In the National Building Code (NBC) of Canada [1], BTFs are included as a seismic force resisting system (SFRS) with two ductility categories: (a) moderately ductile with a ductility-related force modification factor, R_d = 2.0, and an over-strength-related force modification factor, R_o = 1.5, and (b) limited ductility with R_d = 1.5 and R_o = 1.5. For design details on BTFs, NBC references CSA O86, the Canadian Standard for Engineering Design in Wood [2]. No design guidelines for BTFs, however, currently exist in CSA O86, making the system out of reach of the average designer [3].

To remedy the situation, FPInnovations is leading a multi-year research project to determine the seismic behaviour of BTFs as a SFRS and generate the technical information needed for development of design guidelines for BTFs in CSA O86. The research project include (a) connection testing, e.g., bolted connections, see Figure 1; (b) theoretical analysis, e.g., the derivation of relationship between local and global ductility; (c) seismic design investigation, e.g., the design provision development; and (d) seismic response analyses, such as pushover and nonlinear time history analysis.

Figure 1. A brace specimen with two end bolted connections

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https://doi.org/10.52202/080513-0171

This paper presents the results from a study on the seismic response of braced timber frames with bolted connections located in Montreal, Canada.

2 – DRAFT DEISGN PROVISIONS

Draft seismic design provisions were developed based on the best understanding of the seismic behaviour of BTFs obtained from the research thus far. This section covers the main aspects of the draft design provisions.

2.1 GENERAL DESIGN

BTFs should have all members triangularly connected with the diagonal braces oriented between 30° to 60° from the horizontal beam (strut). Diagonal braces inclined at ~45° are recommended, as in most cases this provides more efficient system compared to other arrangements. It is sometimes convenient to use several braced bays rather than a single bay to reduce the overturning demands [3]. It is the most efficient to place the frames at the perimeter of the building to provide large box effect and torsional resistance. Frames should be arranged symmetrically in the floor plan, to lower the effects of the torsional moments and decrease irregularity of the building.

Depending on the design, the BTFs as an SFRS can also be designed to be part of the Gravity Load Resisting System (GLRS). Whether or not a BTF belongs to the GLRS will affect the design of the frame elements. The effects of the deformed geometry of the structure (second order effects) need to be considered if the deformations during the response significantly increase the forces in the structure, or if the deformations significantly modify the structural behaviour [4].

2.2 SEISMIC DESIGN CONSIDERATIONS

BTFs should be designed according to the capacity-based design provisions. Inelastic deformations and energy dissipation should only occur in the connections between the diagonal braces and the rest of the frame (called brace connections). Brace connections should be able to yield by combination of wood crushing and fastener yielding. To ensure ductile behaviour, the lowest brittle failure mode resistance of the brace connections should be at least 60% higher than that of their governing yielding failure mode. In addition, moderately ductile dissipative connections should have their resistance of the most ductile modes (d) or (g) be at least 30% lower than that of the other less ductile failure modes. Also, the braces and the parts of the brace connections connecting them to the rest of the frame should not buckle in-plane or out-of-plane direction. Dissipative connections should possess sufficient deformation capacity to allow the frame to attain its target lateral deflection. A sufficient gap should be left between the end of the diagonal brace and the rest of the frame to ensure that the brace connection is able to develop the deformation needed. Based on the tests results [5], a minimum diagonal gap of 50 mm was suggested.

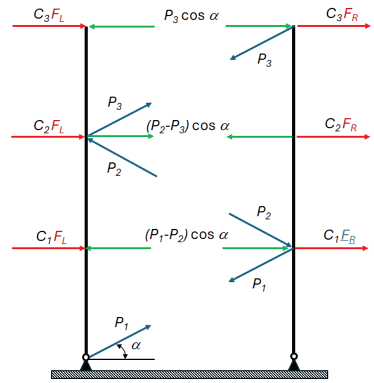

Figure 2. Forces on the left and the right column of a typical BTF according to the column tree design method

All other connections should be designed as non-dissipative ones. Non-dissipative connections should be designed to resist the force and displacement demands that are induced in them when the brace connections reach the 95th percentile of their ultimate resistance, or their target displacement. Based on the fastener type used, this can be achieved by designing the connections with an overstrength factor. An overstrength factor of 2.0 was chosen for the bolted connections based on statistical analysis of the test results and factored designed resistances from CSA O86.

Similarly, frame members (columns, diagonal braces, and beams) should be designed for seismic forces that are developed when ductile brace connections reach the 95th percentile of their ultimate resistance. This can also be considered achieved if they are designed using the same overstrength factor as non-dissipative connections, i.e., 2.0. Meanwhile, columns should be designed to be continuous along the entire height of the frame with adequate strength and stiffness to spread the yielding in all brace connections along the height of the frame. This can be achieved by using either the "column tree design method" proposed for steel structures or the worst soft-storey scenario with the removal of a diagonal brace [6]. Figure 2 illustrates an application of column tree design on a 3-storey BTF.

As shown in Figure 2, according to the column tree design, each column is designed as a pinned supported beam balanced by external and internal forces. The lateral forces on each storey of the columns are defined as C_iF_L for the left column, and C_iF_R for the right column, for each storey i. The C_i coefficients are related to the distribution of the seismic forces along the height of the building determined either using the equivalent static force procedure or response spectrum analysis. For example, if the equivalent static force procedure was used for the example shown in Figure 2, the inverse triangular distribution will define the values for the coefficients as: C_1=0.334, C_2=0.667, and C_3 = 1.0. The forces P_i on the columns are the forces that are obtained from the static analysis of the braced frame subjected to the design lateral loads. These forces should include the overstrength factor as well. Using moment

https://doi.org/10.52202/080513-0171

equilibrium about the bottom of each column, the force F_L acting on the left column and the force F_R on the right one can be determined. The columns are then designed to withstand the internal forces that act on them. Preliminary nonlinear dynamic analyses have shown that this method results in column sizes that are able to adequately spread the yielding along the height of the structure and prevent a soft storey mechanism from occurring [6].

Splice connections in the columns should be designed as non-dissipative connections with adequate strength and stiffness. Splices should be placed on the columns where bending moments are at their minimum. Reduction of column cross section along the height is allowed according to the design and stiffness requirements. Columns should not buckle in either the in-plane or out-of-plane direction. All members of the frame should be designed to be concentric to avoid development of bending moments in the connections between the braces and the rest of the frame and between the beams and the columns. Influence of the brace rotation on the performance of the brace connections should be minimized. Connections anchoring the frame to the foundation should be designed and detailed as pinned to allow for the column to rotate.

3 – ARCHETYPES DESIGN

3.1 DESIGN TOOLS

Due to the large number of designs required for seismic evaluation, a new module was developed in Altair S-TIMBER computer program in collaboration with Altair staff, to automatise the seismic design of different BTF building archetypes. The module was developed using integrated Python scripting interface in S-TIMBER, and it consists of three main parts, as shown in Figure 3.

The output of the module is detailed information of the designed archetypes that could be used for developing the nonlinear models of BTF archetypes that were used for seismic evaluation. Figure 4 shows a screenshot of the developed module in S-TIMBER, showing BTF code-check results.

3.2 BUILDING ARCHETYPES

A total of 144 archetype buildings were designed for concentrically braced frames with bolted connections for determining the R_d-factors. A summary of the archetype breakup is shown in Table 1.

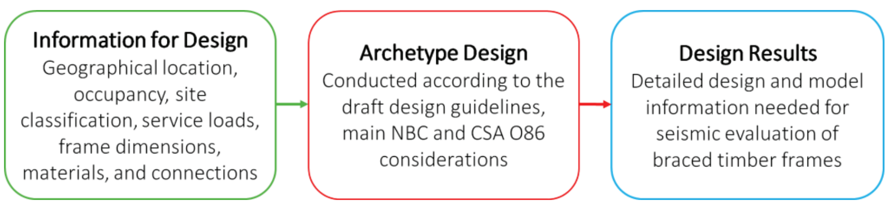

Figure 3. Main parts of the developed seismic design module for BTFs in Altair S-TIMBER

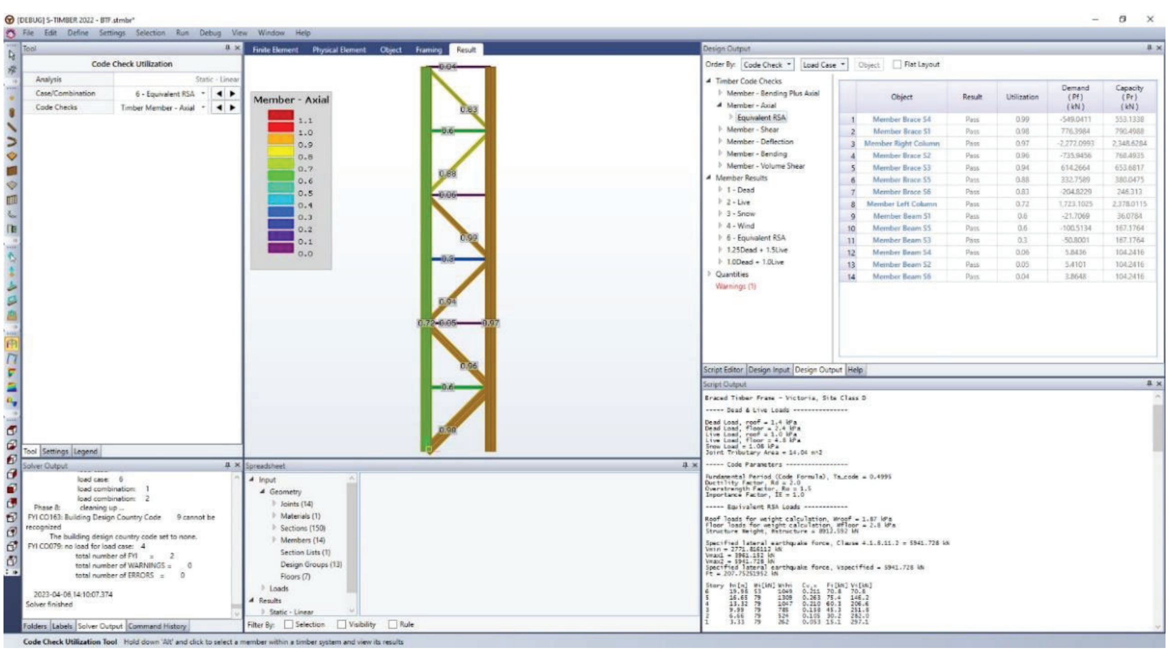

Figure 4. A screenshot of the developed module in Altair S-TIMBER showing BTF code-check results - all member's summaries

Below is the main process adopted in the engine:

Table 1: Building Archetypes

Parameter	# of cases	Case
Location	1	Montreal
Material	2	D-fir and SP
Occupancy	1	Commercial
Configuration	4	6m (2-S), 12m (4-S), 15m (5-S), and 21m (7-S)
Tier aspect ratio	3	2:3, 1:1, and 3:2
Connection	3	9.5mm, 12.7mm, and 15.9mm bolts
R_d-factor	2	1.5 and 2.0
Total	144	=1×2×1×4×3×3×2

Archetypes were designed for site class D, Montreal, Canada by using either D-Fir or SP glulam. All building archetypes were assumed to have an identical floor plan with a length of 31.2 m, a width of 18.0 m, and a storey height of 3.0 m. They were commercial applications with four different heights: 6 m, 12 m, 15 m, and 21 m, corresponding to 2-, 4-, 5-, and 7-storey buildings, respectively. Three different tier aspect ratios: 2:3, 1:1, and 2:3, were considered. R_d factors of 1.5 and 2.0 were considered as those are the R_d-factors provided for BTFs in the NBC. Finally, the archetypes used connections with three different bolt diameters: 9.5 mm, 12.7 mm, and 15.9 mm, to match the tested brace configurations. For each of these 144 archetypes numerical models were built using the OpenSees software. One half of these archetypes were also modelled using reinforced connections, making the total number of nonlinear numerical models developed to 216. Tens of additional archetypes with higher R_d factors were also designed to investigate how higher R_d -factors can be used for certain archetype configurations.

4 – SEISMIC EVALUATION

4.1 ANALYSIS ENGINE

An analysis engine was developed in MATLAB to automatize the modelling and nonlinear dynamic analyses of different models of the archetypes (Figure 5). The engine was able to generate nonlinear models for the selected archetypes based on the information from the seismic design module mentioned before, i.e., Section 3. The engine was also used to conduct Incremental Dynamic Analysis (IDA) to investigate the seismic response of BTFs in Montreal with different R_d-factors under different levels of ground motions. An assessment of the seismic performance of the frames was conducted based on the results from the analysis engine.

(a) Read the key design information from the CSV files generated by Altair S-TIMBER Design Module;

(b) Develop nonlinear models based on the key design information;

(c) Scale the ground motions to a specific level;

(d) Run nonlinear time-history dynamic analyses for each model with the scaled ground motions;

(e) Output the key results;

(f) Repeat steps (c) to (e) as needed, e.g., until the model collapses under more than 50% of ground motions at a specific level;

(g) Analyse the results to generate motion intensity vs. inter-storey drift curves and probability of collapse vs. motion intensity curves.

4.2 MODEL DEVELOPMENT

Since all BTFs in one single building were identical and placed symmetrically in the floor plan, a two-dimensional modelling approach [3] was adopted in this study using the OpenSees computer software. All braced frame models were 2-D and had a single bay. All member and connection details were obtained from the S-Timber design output. Columns were continuous elastic beam-column elements, pinned at the base. Elastic truss elements were used for the beams and braces connected to the columns using pin connections. Bolted connections were simulated using one zero-length element at one end of the brace. The load-deformation properties of both connections were included in the single zero length element. A typical 4-storey braced frame model used in the analysis is shown in Figure 6.

A typical timber connection model – Pinching4 (Figure 7), was utilised to simulate the stiffness, strength, plastic deformation, and hysteretic behaviour of the connections at the ends of braces. This model has 39 different parameters that define the connection behaviour. Parameters for this model for various connections were derived by fitting the hysteresis loops obtained from the cyclic tests. The hysteresis loops that represent the average response of each configuration were used for the parameter identification. Example of the modelled behaviour vs the behaviour obtained from testing is shown in Figure 8.

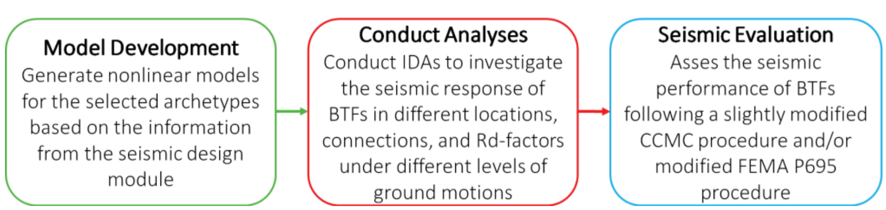

Figure 5. Main parts of the analysis engine developed for seismic evaluation of various BTF archetypes

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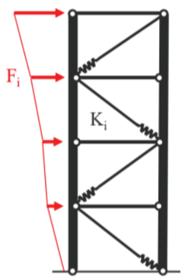


Figure 6. Schematics of a 4-storey BTF model

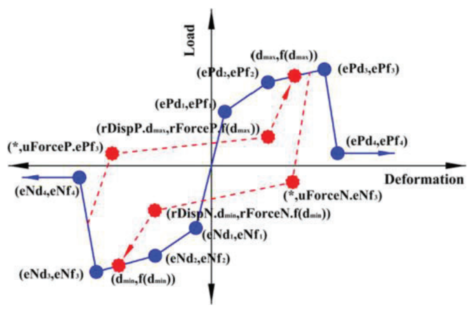


Figure 7. The Pinching4 model in OpenSees

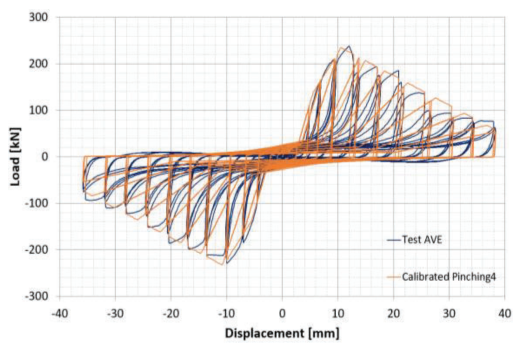


Figure 8. Comparison of the hysteretic response of two bolted connections between the testing and modelling

In addition, connection model updating rules were developed. The updating rules established the relationship between the model parameters and some of the most important connection properties such as the material of the brace, number and diameter of the bolts used, different bolt slenderness ratios, connections with and without STS reinforcement, etc. The developed updating rules were implemented in the analysis engine (Figure 5) for the development of nonlinear BTF models in OpenSees.

4.3 GROUND MOTIONS

The seismic response of the BTFs with different connections and R_d-factors was analysed using a series of IDA with different earthquake motions. Montreal was chosen to represent the spectrum of seismic hazards for SC3 in Eastern Canada. A set of eleven far-field ground motions (Figure 9) from PEER NGA-West2 database was selected and scaled for Montreal, Site Class D (Vs30 = 290 m/s), according to Method A of Appendix J of NBC for the period range of the structures being analysed.

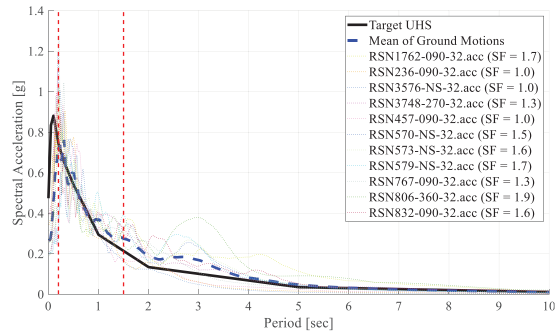


Figure 9. Selected ground motions for 2%/50 years hazard level scaled between 0.2 s and 1.5 s

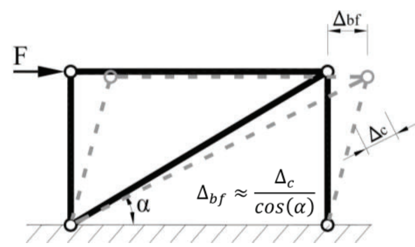


Figure 10. Relationship between the total deformation in two brace connections ($\triangle_c$) and the braced frame lateral deformation ($\triangle_{bf}$)

Table 2: Inter-storey drift limits

Tier Aspect Ratio	100% UHS Drift Limit [%]			200% UHS Drift Limit [%]		
	Bolt Diameter [mm]			*Bolt Diameter [mm]*		
	9.5	*12.7*	*15.9*	*9.5*	*12.7*	*15.9*
2:3	0.8	1.0	1.0	1.5	2.0	2.0
1:1	0.9	1.2	1.2	1.7	2.3	2.4
3:2	1.1	1.5	1.5	2.2	2.9	3.0

4.4 EVALUATION CRITERIA

The CCMC guide [7], is a simplified version of FEMA P-695, that was developed in Canada to assess the seismic performance of different building archetypes and determine the appropriate R_d-factors. This guide was used as a basis for the evaluation of the seismic response of the BTFs. The CCMC guide requires that inter-storey drifts (ISDs) under earthquake motions scaled to 100% of the Uniform Hazard Spectrum (UHS) per NBC, should not exceed 2.5%. For response to ground motions scaled to 200% of the UHS, the ISDs from the suite of analyses should not exceed 4.5% for 50% or more of the motions.

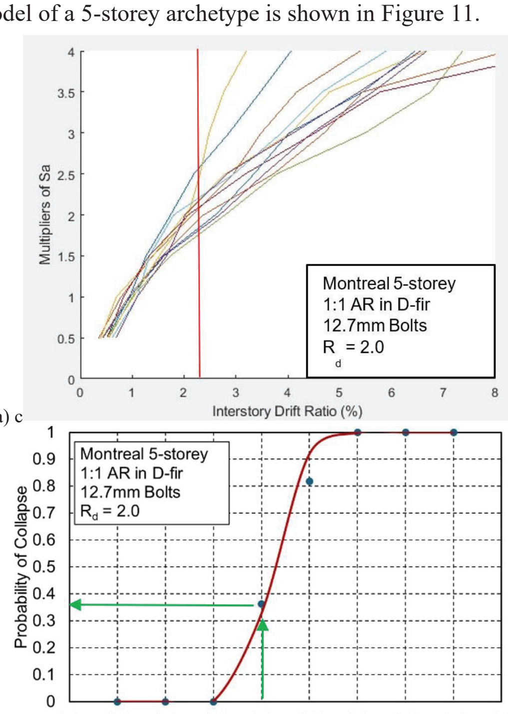
However, some modifications to the CCMC guide requirements were made to better accommodate BTFs as a system. Due to the stiff characteristics of BTFs and the available connection deformation, collapse occurs at a much lower ISD than the maximum of 4.5% mentioned in the CCMC Guide. The ISD limits set were different depending on the connection deformability and the aspect ratio of the tier (Figure 10). Table 2 lists the ISD limit criteria for BTFs with bolted connections and different aspect ratios that were used in the study, along with the necessary parameters for deriving the criteria.

5 – RESULTS AND DISCUSSIONS

According to NBC 2020, the design spectral accelerations of Montreal (City Hall area, Site Class D) at different periods were: Sa(0.2) = 0.744, Sa(0.5) = 0.542, Sa(1.0) = 0.294, Sa(2.0) = 0.134, Sa(5.0) = 0.035 (Figure 9), which is the maximum possible upper bound of seismic category 3 (SC3). The 1-in-50-year ground snow load and associated rain load were taken as 2.6 kPa and 0.4 kPa, respectively. The dead loads on the roof and the floor were 1.4 kPa and 2.4 kPa, respectively, while the live load was 4.8 kPa (commercial occupancy) for the floor and 1.0 kPa for the roof. It should be noted, however, that the results presented in this paper apply to residential occupancy as well as some of the gravity loads used are not taken as part of the seismic weight.

The seismic response of BTFs was investigated by conducting IDAs in OpenSees, where the nonlinear models were subjected to 11 ground motions from 0.5 to 4 times the design Sa (0.5 S_a to 4.0 S_a). Maximum ISDs obtained from the analyses were considered as the primary seismic performance indicator. Based on the data obtained from the analyses two types of curves were developed for each analysed building model: spectral acceleration factor versus ISD ratio curves, and fragility (probability of collapse versus spectral acceleration factor) curves. Example of these two types of curves for a numerical model of a 5-storey archetype is shown in Figure 11.

(a) c



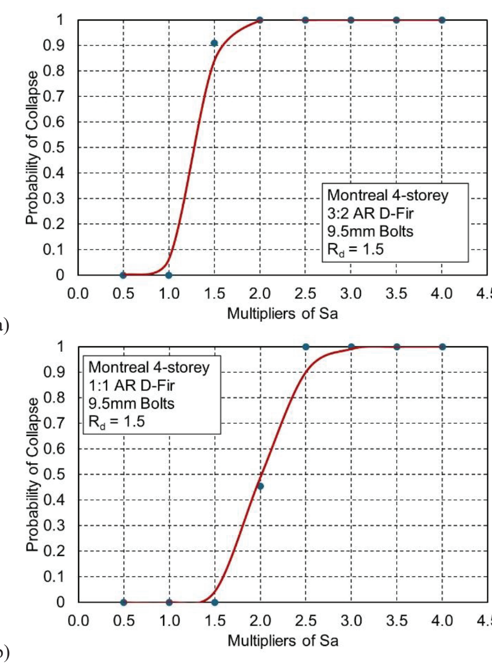
(b)

Figure 11. Selected spectral acceleration factor vs ISD ratio curves for a 5-storey archetype in D-Fir glulam with a tier aspect ratio of 1:1, using 12.7 mm bolts, designed with R_d = 2.0 (a); and the archetype fragility curve (b)

As can be seen in the Figure 11 (b), the probability of collapse at 2.0S_a is just above 35% for the archetype, which is less than the 50% performance requirement. This means that this design can be assigned an R_d-factor of 2.0, as per CCMC guidelines.

5.1 INFLUNECE OF TIER ASPECT RATIO

As expected, the tier aspect ratio (AR) of the frames had a significant influence on the frame behaviour. Frames with the highest aspect ratio of 3:2 (the narrowest frames), were much more flexible and had much more pronounced bending behaviour during the response, making the braces less efficient. In addition, the 3:2 AR frames had the highest deformation demand on the brace connections and in most cases this demand exceeded the deformation capability of the brace connections. Consequently, the results from the analysis have shown that these frames had the highest probability of failure of the three aspect ratios analysed, and in most cases, they were not able to satisfy the probability of collapse criteria at 200% S_a. An example of this is shown in Figure 12(a), which illustrates the fragility curve for a 4-storey braced frame archetype with 3:2 aspect ratio in D-Fir with 9.5 mm bolts designed with R_d = 1.5. As can be seen, this archetype had almost 100% probability of collapse at 200% of S_a, thus not satisfying the acceptance criteria for adequate performance for the chosen brace connections and an R_d = 1.5.



(a)

(b)

Figure 12. Examples of fragility curves for braced frame archetypes with different aspect ratios: 4-story frame with 3:2 tier aspect ratio designed with R_d = 1.5 (a); the same frame with 1:1 tier aspect ratio (b)

Braced frames with a tier AR of 1:1 showed deformation behaviour reminiscent of structures with a combination of bending and shear deformations. While braced frames with 2:3 tier AR (the widest frames) showed behaviour

https://doi.org/10.52202/080513-0171

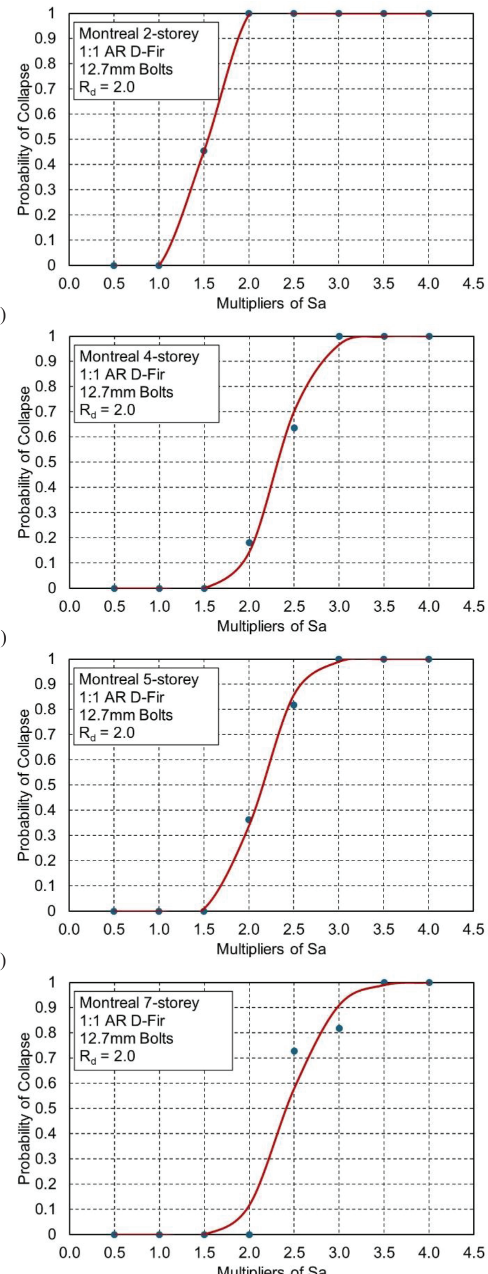
that is most reminiscent to that of shear type structures. The latter two frames were able to satisfy the probability of collapse criteria at 200% S_a in most cases. An example of this is shown in Figure 12(b) where the same archetype as the one shown in Figure 12(a) with a different AR (1:1 vs. 3:2) was able to have a probability of failure below 50% at 200% of S_a, thus satisfying the acceptance criteria for that design.

For the reasons mentioned above, frames with a tier AR of 3:2 were not allowed in the proposed design provisions for BTFs in CSA O86, and only frames with ARs between 1:1 and 2:3 were allowed. These frames make excellent use of the braces and the brace connections leading to efficient designs. Frames with ARs close to 1:1 were recommended, because in these cases the braces becoming shorter, more efficient, and less prone to buckling.

5.2 INFLUECE OF BUILDING HEIGHT

Results from the IDAs also showed that the number of stories (the building height) had a significant effect on the archetype performance. This also had a direct effect on the acceptable R_d-factors for the design of BTFs. Results showed that the ISD demands for 2-storey archetypes were higher than those with more stories. Consequently, most taller frames had lower probability of failure than 2-storey frames and were able to satisfy the performance criteria. Although this is a counter-intuitive finding, it is believed that this can be attributed to the predominant period of the motions with respect to the period of the buildings. This can also be attributed to the so-called "short period paradox" as covered in detail in FEMA P-2139 documents. It has been found over the past few decades that in many cases, low-rise, short-period, buildings tend to show higher probability of failure when analysed numerically, while no such performance is observed during past earthquakes.

Figure 13 shows the fragility curves for BTFs with four different heights. All archetypes had an AR of 1:1, were made of D-Fir glulam, and used brace connections with 12.7 mm bolts. Figure 13 (a) shows the fragility curve for a 2-storey frame while the curves for the 4-, 5-, and 7-storey frames are shown in Figure 18 (b), (c) and (d), respectively. As can be seen in the Figure, the 2-storey archetype did not satisfy the performance criteria at 200% Sa, while all other archetypes were able to satisfy the performance criteria.



(a) (b) (c) (d)

Figure 13. Fragility curves for BTFs with different heights, AR of 1:1, made of D-Fir glulam using 12.7 mm bolted connections: (a) 2-, (b) 4-, (c) 5-, and (d) 7-storey frames

5.3 INFLUENCE OF BOLT DIAMETER

Figure 14 shows the fragility curves for four other archetypes with different heights. These archetypes were the same as those shown in Figure 13, except that they used 15.9 mm bolts in the brace connections compared to the 12.7 mm bolts used in BTFs shown in Figure 13. As can be seen in Figure 14, all archetypes were able to fulfil the performance criteria at 200% S_a, including the 2-storey ones, that failed when designed with 12.7 mm bolts.

By comparing the results shown in Figures 13 and 14 it can be noticed that although the BTFs used bolts with the same slenderness ratio, the frames used larger diameter bolts have lower probability of collapse, especially for the 2-storey buildings. This is attribute to the wedge effect caused by a smaller bolt diameter. Although there were only few archetypes of all bolt diameters that did not meet the performance criteria, to be on the conservative side, the acceptable R_d-factor for all BTFs in D-Fir with bolted connections was chosen to be 1.5, based on the building height criteria only.

5.4 INFLUENCE OF GLULAM SPECIES

The choice of material for the glulam also had an impact on the performance of the archetypes. Generally, frames made of D-Fir have a lower probability of collapse compared to those made of SP. An example of comparison can be made between Figure 14c and Figure 15.

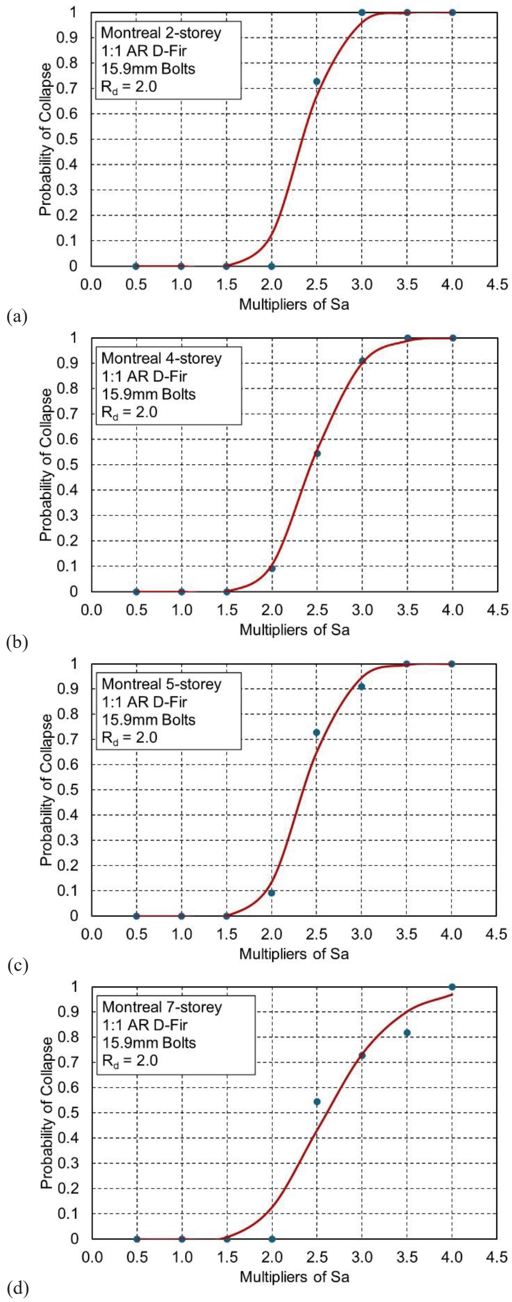


Figure 14. Fragility curves for BTFs with different heights, AR of 1:1, made of D-Fir glulam using 15.9 mm bolted connections: (a) 2-, (b) 4-, (c) 5-, and (d) 7-storey frames

Most archetypes made from D-Fir glulam were able to satisfy the limited ductility (R_d = 1.5) performance requirements, e.g., Figure 12(b). Many of them were even able to meet the moderately ductile (R_d = 2.0) performance criteria, e.g., Figure 14. Many frames made of SP glulam were able to meet the performance criteria for R_d = 1.5, with some of them, especially the taller ones, were able to meet even the R_d = 2.0 criteria (Figure 15). In other cases, however, the archetypes made of SP glulam were not able to satisfy the performance requirements even for the limited ductility (R_d = 1.5).

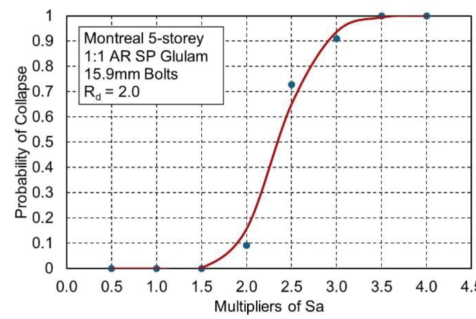


Figure 15. Examples of fragility curves for BTF archetypes in SP glulam that met the Rd = 2.0 performance criteria

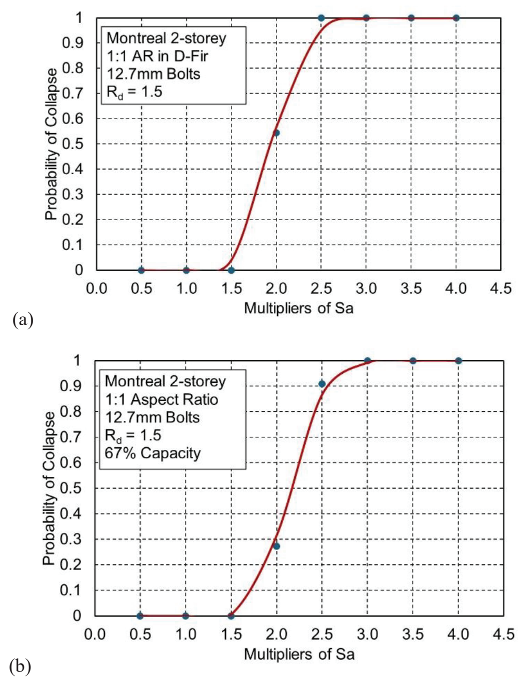


Figure 16. Fragility curves for 2-storey D-Fir frames with bolted connections, designed with R_d = 1.5 with 100% of the factored resistance for the connections as per CSA O86 (a); 67% of the connection factored resistance as per CSA O86 (b)

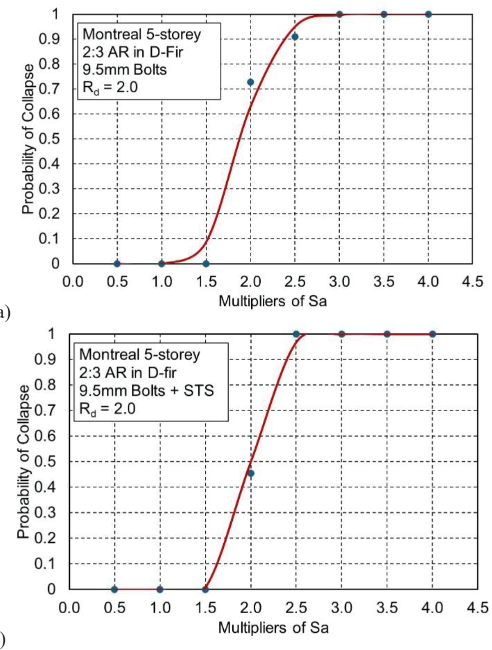
https://doi.org/10.52202/080513-0171

Only three of the BTF archetypes with unreinforced bolted connections in D-Fir were not able to meet the performance criteria for $R_d = 1.5$. All three of them were short period, 2-storey buildings, with the three different bolt diameters used. Similarly, a number of archetypes in SP did not meet the performance criteria for $R_d = 1.5$. An example of the fragility curve obtained for one of the archetypes in D-Fir is shown in Figure 16a. To meet the requirements for $R_d = 1.5$, these archetypes needed to be designed with a lower factored resistance of the connections equal to only 67% (2/3) of the original factored resistance for bolted connections provided in the CSA O86 standard. In such case all archetypes were able to meet the performance criteria with the reduced factored resistance. An example of the fragility curve obtained for one of these redesigned archetypes is shown in Figure 16b. Based on these findings, the design guidelines were modified to state that in the case of BTFs with non-reinforced connections, the factored resistance of the bolted connections shall be taken as 67% of the resistance provided in Clause 12.4 in the CSA O86.

5.5 INFLUENCE OF CONNECTION REINFORCEMENT WITH STS

Reinforcing the D-Fir bolted connections with STS provided significant improvement to the performance of the frames. All frames with reinforced connections were able to satisfy the $R_d = 2.0$ performance criteria. Figure 17 shows examples of the fragility curves for 5-storey frames designed with $R_d = 2.0$ with and without STS reinforcement. Based on these findings, the proposed design guidelines for CSA O86 will state that BTFs made of D-Fir glulam will need the bolted connections to be reinforced with STS in order to satisfy the $R_d = 2.0$ (moderately ductile) performance requirements.

It should be noted that testing of reinforced connections in SP glulam was not conducted since SP is much less prone to splitting then D-Fir. Use of STS to reinforce bolted connections in SP glulam may also improve their performance, however, since test results are not available at this point, no analyses were made using such archetypes and SP glulam was not included in the draft design guidelines.



(a)

(b)

Figure 17. Fragility curves for 5-storey D-Fir CBTFs located in Montreal, designed with Rd = 2.0 with regular (non-reinforced) bolted connections (a), and reinforced connections with STS (b)

Table 3: Acceptable R_d-factors

AR		2:3				1:1				3:2			
# of storeys		2	4	5	7	2	4	5	7	2	4	5	7
Height [m]		6	12	15	21	6	12	15	21	6	12	15	21
9.5 mm	SP	1.5*	1.5	1.5*	1.5	1.5*	1.5*	1.5*	1.5	1.0	1.0	1.0	1.0
	D-Fir	1.5*	1.5	1.5	1.5	1.5*	1.5	1.5	1.5	1.0	1.0	1.0	1.0
	D-Fir R	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	1.0	1.0	1.0	1.0
12.7 mm	SP	≥1.5*	≥1.5*	≥1.5*	1.5	1.5*	1.5*	1.5*	1.5	1.0	1.0	1.0	1.0
	D-Fir	1.5	≥2.0	≥2.0	≥2.0	1.5*	2.0	2.0	2.0	1.0	1.5	1.5	1.5
	D-Fir R	2.0	≥2.0	2.0	≥2.0	2.0	2.0	2.0	2.0	1.0	1.5	1.5	1.5
15.9 mm	SP	≥2.0	≥2.0	≥2.0	≥2.0	2.0	2.0	2.0	2.0	1.0	1.0	1.0	1.5
	D-Fir	≥1.5*	≥2.0	≥2.5	≥2.0	1.5*	2.0	2.5	2.0	1.0	2.0	2.0	2.0
	D-Fir R	≥2.0	≥3.5	≥4.0	≥4.0	2.0	3.5	4.0	4.0	≤2.0	≤3.5	≤4.0	≤4.0

* Results based on reducing factored design capacity to 67% of Clause 12.4 in the CSA O86;

≥ The R_d-factor is expected to be equal or higher than the number shown based on findings from previous analyses;

≤ The R_d-factor is expected to be equal or lower than the number shown based on findings from previous analyses.

5.6 ACCEPTABLE R_D-FACTORS

Table 3 summarize the acceptable R_d-factors for different designs of BTFs based on the results from the IDA conducted on archetypes located in Montreal. Based on the information presented, frames in D-Fir with bolted connections reinforced with STS can be used to satisfy the R_d = 2.0 (moderately ductile) performance category, while to meet the performance requirements for R_d = 1.5, (limited ductility category), BTFs can use unreinforced connections in D-Fir or SP designed with a factored resistance that is equal to 67% (2/3) of the original factored resistance for bolted connections provided in the CSA O86 standard.

6 – CONCLUSION

FPInnovations is leading a multi-year research project to study the seismic performance of BTFs as SFRSs and generate the technical information needed for the development of design guidelines for this system in CSA O86. This paper presents the results from a study on the seismic response of BTFs with bolted connections in Montreal, Canada.

To quantify the R_d-factors for different seismic designs of BTFs, a large number of archetypes were designed. A new module was developed in Altair S-TIMBER computer program in collaboration with Altair staff, to automatize the seismic design of the archetypes. Montreal, Quebec, was chosen for the archetypes as a representative location at the upper bound of seismic category 3 (SC3) in the Eastern seismic region of Canada.

The seismic response of the archetypes was evaluated using a series of IDAs using OpenSees computer program. A new engine was developed in MATLAB to automatize the modelling and nonlinear dynamic analyses of the archetypes. A suite of eleven ground motions were developed for the Montreal City Hall area and were used to analyse the designed archetypes.

Results from the IDAs have shown that the R_d factors for the analysed buildings were governed by the shortest (2-storey) archetypes in Montreal. Frames with the highest aspect ratio of 3:2 (the narrowest frames), were much more flexible and had much more pronounced bending behaviour during the response, making the braces less efficient, compared to frames with an aspect ratio of 1:1 or 2.3. For these reasons, frames with a tier aspect ratio of 3:2 were not included as an option in the proposed design provisions for BTFs in CSA O86, and only frames with the aspect ratios between 1:1 and 2:3 were allowed. These frames make excellent use of the braces and the brace connections leading to efficient designs.

Reinforcing the bolted connections in Douglas Fir with STS provided significant improvement to the performance of the frames. All frames with reinforced connections were able to satisfy the R_d = 2.0 performance criteria. A factor of 67% (2/3) needed to be applied to the factored design resistance of the unreinforced bolted connections in Douglas Fir and Spruce Pine for them to satisfy the R_d = 1.5 (limited ductility) performance requirement. IDAs for locations in Western Canada such as Vancouver and Victoria are needed to check if these design requirements can be used in these areas.

The work presented in this paper was instrumental in the development of the draft design provisions for BTFs that are proposed for acceptance in the upcoming supplement to the 2024 CSA O86. Implementation of BTFs in CSA O86 will allow designers in Canada to have one more choice for a structural system in mass timber construction in mid-rise residential and non-residential applications.

7 – ACKNOWLEDGMENTS

This project was financially supported by Natural Resources Canada and BC Forestry Innovation Investment. The in-kind contribution provided by Mr. Moein Ahmadipour from Altair in developing the modules for design of BTFs, and the Canadian Wood Council staff for designing a significant portion of the BTF archetypes used in the analyses, are greatly appreciated.

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