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SEISMIC PERFORMANCE PARAMETERS EVALUATION FOR TIMBER BRACED FRAMES

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ABSTRACT: This study focuses on timber braced frames (TBFs), which are recognized as a compelling alternative to conventional lateral systems for resisting seismic and wind loads. However, the absence of specific seismic performance factors (i.e., response modification coefficients, system overstrength factors, deflection amplification factors, etc.) in building codes has limited their widespread adoption in the United States. This study investigates the potential of TBFs as a lateral force-resisting system by utilizing the FEMA P-695 methodology to determine seismic performance factors that can be used in equivalent lateral force and modal response spectrum procedures in building codes in the United States. Several topics are discussed including existing test data on timber brace dowel connections, modelling methods of the connection response for incorporation into the analysis model, development of TBF building archetypes, and results from an application example subjected incremental dynamic analysis. The nonlinear response history analysis results indicate that the deformation capacity of the brace connection is critical in assessing the seismic performance factors.

KEYWORDS: dowel connection, dynamic analysis, FEMA P-695, seismic performance factors, timber braced frames.

1 – INTRODUCTION

The use of timber as a renewable resource in the building industry has gained increasing attention due to growing interest on designing buildings for enhanced sustainability [1]. Traditionally, timber members were primarily used for carrying gravity loads, while lateral loads such as seismic and wind forces were resisted by light-frame shear walls, steel, or concrete elements. Mass timber lateral force-resisting systems have recently gained significant traction, including laminated veneer panels [2][3], cross-laminated timber (CLT) and mass plywood panel (MPP) shear walls [4], hybrid timber-steel buckling-restrained braces (BRBs) [5], and other hybrid systems where timber columns and beams are combined with steel braces [6].

Among these systems, timber braced frames—where all members are timber and connected with steel connectors—have garnered particular attention as an architectural appealing and quickly constructed lateral force-resisting system [7]. Fig. 1 illustrates an example of a timber braced frame building built in Canada [8]. This four-story building consists of timber columns, beams, and braces in a chevron configuration.

Figure 1. *Timber braced frame building example, UBC Earth Sciences Building. Credit: Construction Canada [8].*

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Over the past two decades, extensive research has been conducted to understand the behaviour of timber braced frame buildings under extreme events, such as earthquakes. Timber brace systems, especially those utilizing mass timber materials, have been proposed and studied in Canada [9]. These frames have demonstrated the ability to resist both seismic and wind loads and have been implemented in some buildings in Canada and Europe, e.g.[8] and [10].

Despite advancements listed above, timber braced frames have yet to be fully integrated into structural design codes in the United States. The primary barriers to adoption are the absence of seismic performance factors in building codes and the lack of a standardized design guide for this system. This conference paper presents initial work being developed by the research team on both topics.

2 – BACKGROUND

2.1 BRACE CONNECTIONS

Significant research has been conducted on the properties of bolted connections in timber; however, most studies have focused on maximum strength to estimate force capacity, with limited attention given to deformation capacity or the cyclic response and hysteretic behaviour of these connections. While few studies have specifically examined the ductility of bolted timber brace connections, experimental tests have shown that these connections exhibit relatively low ductility due to brittle failure modes of timber, such as splitting, at large deformations.

To enable bolted connections to achieve large deformation capacities, preventing the wood from splitting is a crucial factor. Popovski [9] and Chen et al.[11] investigated the performance of bolted connections in mass timber braces through experimental testing, utilizing two-sided plate bolted connections with self-tapping screw reinforcement perpendicular to the grain. The bolt diameters range from 9.5 mm (3/8 in.) to 25.4 mm (1 in). Baird et al. [12] designed bolted connections for glulam timber braces with a focus on connection ductility, conducting experimental tests on 12 specimens. That study included parameters such as the number of slotted-in steel plates, bolt size and spacing, and the presence of self-tapping screw reinforcement perpendicular to the grain. These existing test results highlight that dowel spacing and reinforcement perpendicular to the grain are critical parameters influencing connection ductility. In the test, an 18-bolt connection with a diameter of 12.7 mm (1/2 in.) and self-tapping screw reinforcements reached a maximum load of 777 kN (175 kips). The ultimate deformation capacity of 27.1 mm (1.0 in.) was observed at the bottom connection, where splitting failure occurred.

Fig. 2 presents the relationship between connection ductility and the ratio of bolt spacing to bolt diameter in existing test data [9][11][12], where values are taken from each paper, and connections reinforced with screws are represented by solid dots. When comparing results within the same test series, a moderate trend emerges: increasing the spacing-to-diameter ratio of bolts and/or providing reinforcing screws generally enhances ductility.

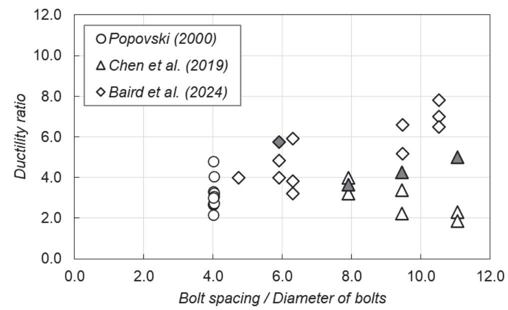

Figure 2. Relationship between connection ductility and the ratio of bolt spacing to bolt diameter in existing test data [9][11][12].

2.2 SEIMIC PERFORMANCE FACTORS AND FEMA P-695 METHODOLOGY

The seismic performance factors, such as the response modification coefficient (R), system overstrength factor (Ω_0), and deflection amplification factor (C_d), are commonly used in the U.S for designing buildings using the equivalent lateral force or modal response spectrum procedures described in ASCE 7-22 [13]. The Canadian building code, CSA086, also includes these seismic performance factors with some modifications [14]. Although some initial efforts have been made to estimate these factors [15] —such as by assessing system ductility based on the diagonal brace connection ductility or through 2D non-linear push-over analyses involving timber braced frames with varying numbers of stories, story heights, and aspect ratios—these factors for TBFs have yet to be fully validated.

The standardized way to assess these factors is through the FEMA P-695 methodology [16], which evaluates trial values against specific acceptance criteria. This approach involves a collapse assessment of various archetypes with the structural system of interest. In this study, we utilize this methodology by developing a three-story office archetype building with timber braced frames and performing incremental dynamic analysis.

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3 – ANALYSIS OF TEST RESPONSE

3.1 EXPERIMENTAL TEST OF BOLTED CONNECTOIN

In this research, a reference brace bolted connection of test specimen 1P-12-75-s from Baird et al. [12] is used as the basis to assess the seismic performance of a three-story TBF building. Fig. 3 illustrates the connection. The braces are made from Grade 24f-E spruce-pine glulam with a cross-section of 265 mm x 305 mm (10.5 in. x 12 in). The bolted connection configuration consists of a 10 mm (0.39 in.) thick single inserted plate, 18 bolts of 12.7 mm (0.5 in.) diameter, and 8 mm (0.31 in.) diameter reinforcement screws driven perpendicular to the grain.

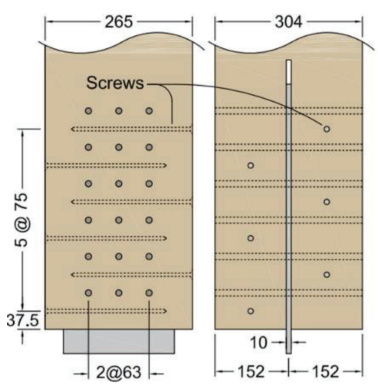

Figure 3. *Connection of 12.7mm bolts and self-tapping screws [12]. Dimensions in millimetres.*

Fig. 4 shows the force versus displacement response of the top and bottom end connection in 1P-12-75-s [12]. The results shown can be used to determine the connection ductility of 6.57 taken at the displacement at which 20% loss of peak strength is reached. Further details of the test can be found in [12].

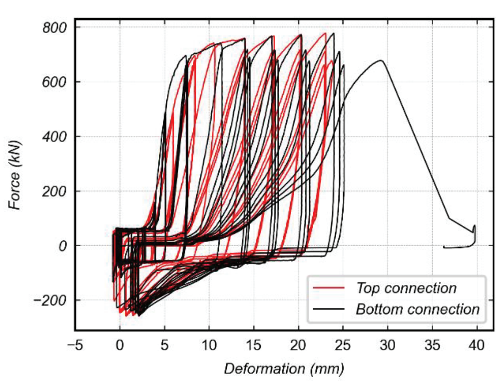

Figure 4. *Force versus displacement response of the top and bottom connection in 1P-12-75-s [12].*

3.2 BACKBONE CURVE

A backbone curve is a representation of the force-deformation envelope of a structural component, reflecting the structural properties of the tested connection, such as its initial stiffness, yielding point, peak strength, ultimate strength, and deformation capacity. The ASCE 41 trilinear curve [17], using methods described in Bora et al. [18], is used to establish the backbone model. Before modelling, however, the test data is first horizontally shifted to account for initial slip in the test setup. The shift is determined at the intersection of the x-axis and the initial stiffness line, which is derived from the points of 10% to 40% of the maximum strength. In the method, the yielding strength is determined using the offset method. The initial stiffness is shifted back by 0.05 times the bolt diameter, and the reference yield deformation is identified as the point where the offset line intersects the original response curve. The reference deformations at maximum strength and at ultimate strength—the later defined as the point where peak strength decreases to 80% of the maximum strength—are then computed by subtracting a displacement equal to 0.05 times the bolt diameter. The resulting backbone curves are shown in Fig. 5. In the figure, the ultimate point of the top connection is not modelled since only the bottom connection failed at the end of the test. The modelled backbone curves at the top and bottom connections are similar, particularly in terms of the yielding strength and the maximum strength point, demonstrating the connection behaviour is stable.

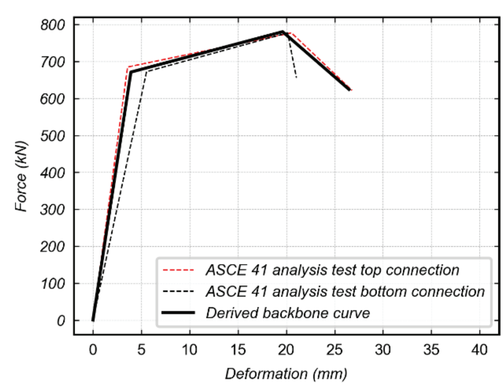

Figure 5. *Backbone curve and modelling results of 1P-12-75-s [12].*

3.3 BACKBONE CURVE MODELLING

An approach is proposed to develop normalized backbone curves for various connection configurations. Fig. 6 illustrates the generalized parameters that define the backbone curve. In addition to the initial stiffness, (K), and yield strength, P_y, four key parameters are used to estimate the normalized backbone curve. These parameters include the yield and maximum strength ratios relative to the design value (R_y and R_{max}), as well as the deformation ratios at maximum and ultimate strengths relative to the yield deformation (μ_{max} and μ).

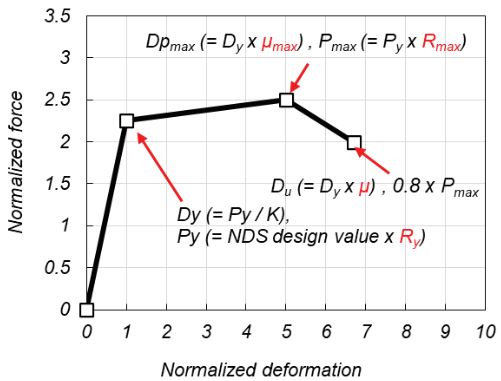

Figure 6. *Normalized backbone curve.*

The first calculated value is the initial stiffness, K. The stiffness of dowel connections varies significantly depending on several factors, such as species and specific gravity of wood, moisture content, the presence of defects, and dowel type and configuration. which make it challenging to obtain accurate stiffness values. Eurocode 5 [20] provides a simplified method to estimate the stiffness of dowel connections based on the dowel diameter and specific gravity of wood material, and is given by:

$$K_{conn} = n * \frac{G^{1.5} * D}{23} * n_{shear} \tag{1}$$

where K_{conn} = stiffness of brace connection at one end of timber brace, N/mm; n = number of fasteners in timber brace connection; G = density of wood (kg/cm^3); D = fastener diameter, mm; n_{shear} = number of shear planes. For the 1P-12-75-s connection, the estimated stiffness of is 171.1 kN/mm, and the obtained fitted mean stiffness is 156.6 kN/mm.

Second, the yield strength of the connection is determined using the U.S. National Design Specification for Wood Construction (hereafter, NDS) [19], which provides yield mode equations for dowel connections accounting for various dowel types and failure mechanism yield modes. The adjusted design value for 1P-12-75-s is determined to be 312.2 kN while the yield strength from the test data is 677.4 kN, which corresponds to the mean of the yield values for the top and bottom connection. The ratio of the test mean yield strength to the NDS adjusted design value is approximately 2.17.

The additional four parameters that define the backbone curve were determined based on data available in [12]. Based on analyses of the test data, the mean values are listed in the second column in Table 1. For simplicity and generalization, in this study, we use the proposed values in the third column in Table 1 for subsequent analyses.

Using the NDS design value, initial stiffness determined using Equation 1, and the proposed values in Table 1, a force-displacement relationship can be derived. Fig. 5 shows the backbone curves obtained using this method and overlays it with the backbone curve derived from the test data. Since the constructed backbone curve falls within the top and bottom connection data, this figure demonstrates that the proposed approach produces a reasonable backbone curve.

Table 1. *Modelling parameters and calculation results*

Parameter	Test Data	Proposed Values
R_y	2.17	2.15
R_{max}	2.49	2.50
μ_{max}	4.46	5.00
μ	7.48*	6.75

*ultimate deformation parameter, μ is calculated only based on the bottom connection.

3.4 HYSTERESIS MODEL

To conduct nonlinear response history analyses, connection hysteresis models must also be defined in addition to the backbone curve. In this study, the Pinching 4 model [21] in the Open System for Earthquake Engineering Simulation (OpenSees) [22] was investigated to identify the best-fitting parameters for the connection.

To calibrate multiple parameters for the connection model, a uniaxial force-deformation analysis was performed. The experimentally measured deformation history was applied to the connection model. The hysteretic energy dissipation per loading cycle was determined for the first quadrant of the response since the test was primarily performed under tension only strains. Fig. 7 illustrates the response of the numerical model for the connection as well as the corresponding test data. Fig. 8 compares the energy dissipation per loading cycle for the bottom connection. These figures demonstrate that the Pinching4 model, calibrated based on the test data, effectively captures the characteristics of the bolted connection and suggests its suitability for inclusion in analyses of a full frame.

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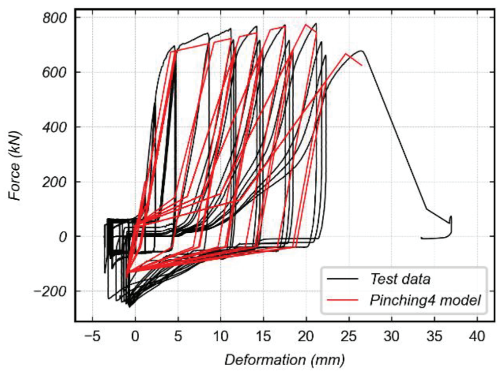

Figure 7. *Hysteretic response for the numerical model (red) and the test data for the 1P-12-75-s bottom connection [12].*

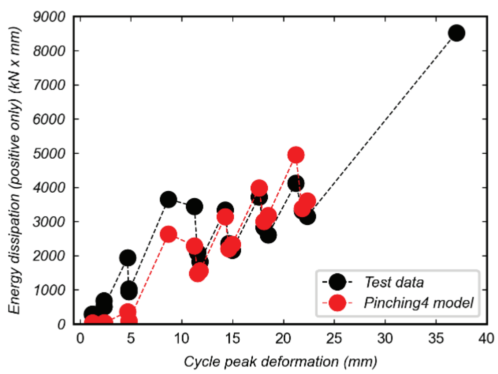

Figure 8. *Energy dissipation for the numerical model (red) and test data for the 1P-12-75-s bottom connection.*

3.5 TRIAL SEISMIC PERFORMANCE FACTORS

Trial seismic performance factors are determined based on the ductility of structural systems. In TBFs, the ductility relies on the brace connection. The test results showed that the ductility of the bolted connection was 7.5 at the point of maximum force, while a strength ratio of the test analysis relative to the NDS calculation value was greater than 2.0. However, system ductility is lower than connection ductility in brace connections, as Chen et al. [11] reported. Considering these factors, we assumed a trial seismic response modification coefficient (R) of 3.0. The system over-strength factor should reflect the ratio of the maximum connection strength relative to the calculated nominal value. The over-strength factor (Ω_0) was therefore set to 2.0. Moreover, following the FEMA P-695 methodology, the deflection amplification factor (C_d) is recommended to be the same value as the R factor. Therefore, this study adopts a deflection amplification factor of 3.0.

4 – ARCHETYPE MODEL ANALYSIS

4.1 ARCHETYPE MODEL DESIGN

To verify the trial seismic performance factors, FEMA P-695 [16] requires a collapse assessment on a wide range of archetypes fulfilling a design space. As an initial attempt to apply the FEMA P-695 procedure to gain a better understanding of the structural properties of TBFs, a single archetype model was developed.

A three-story office building incorporating timber braces was developed. Fig.9 shows perspective elevation views of the archetype building. The archetype features a 4 × 8-bay layout, 30 ft (9.14 m) spans, and a 12 ft (3.66 m) story height, with six timber-braced bays in a chevron configuration in both directions. The structural system consists of glulam columns, beams, braces, and CLT diaphragms. Details of each member are presented in Table 2. The brace connections incorporate inserted knife plates, bolts, and self-tapping screw reinforcement perpendicular to the grain. Connection details are provided in Table 3.

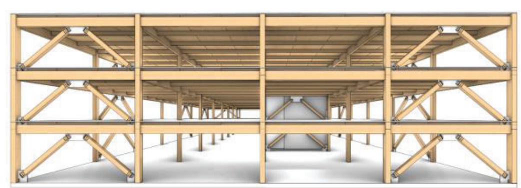

Figure 9. *Perspective elevation view of 3-story office archetype from the short span side.*

Table 2. *Member configurations (unit: mm).*

	Level	Grade	Depth	Width
Column	1st - 3rd	DF L2	343	311
Beam	2nd - Roof	DF L2	610	273
Braces	1st - 3rd	DF L2	305	273

Table 3. *Bolted connection configurations (unit: mm).*

Floor	Material	Diameter	Number	Spacing
1st	A307 bolts	12.7	27	76
2nd	A307 bolts	12.7	24	76
3rd	A307 bolts	12.7	15	76

The building is designed aligned with the ASEC 7 [13] and FEMA P-695 [16]. The seismic category, which corresponds to the level of design earthquake ground motion, is D_{min}/C_{max}. Thus, the design level earthquake spectral accelerations of the short-period and 1-second are 0.50 and 0.2, respectively [16]. Seismic weight is assumed to be 70 psf for the dead load based on the weight of the structure and 65 psf for the live load [13]. As discussed in the previous section, the seismic performance factors, $R = 3.0$, $\Omega_0 = 2.0$, and $C_d = 3.0$, are used in the structural design.

4.2 ANALYSIS MODEL

OpenSees is used for the numerical analysis of the archetype. In order to simplify the analysis model while keeping all the important structural characteristics, one braced frame is modelled with a single P-delta column accounting for the rest of the gravity loads tributary to the braced frame. Images of the modelled frame and the modelling area are shown in Fig. 10 and Fig. 11, respectively.

In this model, 1.05 times the dead load and 0.25 times the live load are considered as a gravity load and a seismic mass [16], applied to the beam-to-column connection in the frame and each node of the P-delta column. Also, the model accounts for the self-weight of the brace at the brace end nodes.

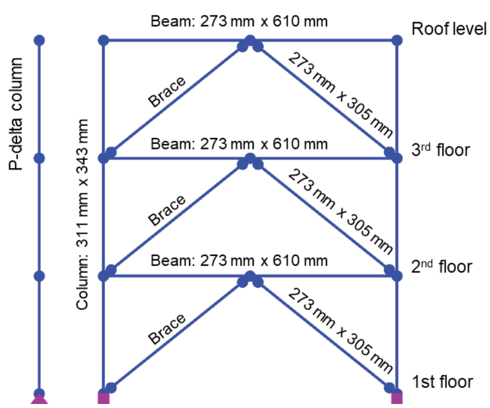

Figure 10. *Analysis model.*

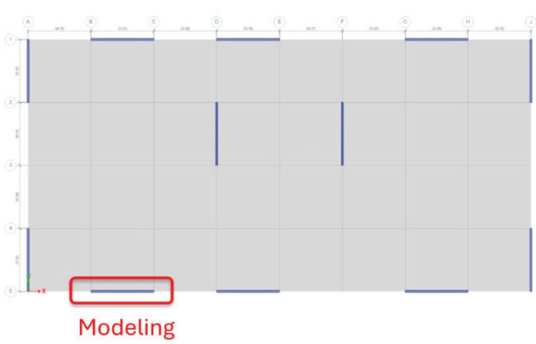

Figure 11. *Modelling area.*

In the frame, the columns are assumed to be continuous. The beam-to-column connections are modelled as pins. Although the base boundary conditions are fixed, rotational springs with neglectable stiffness are placed between the bases and the bottom of the columns, effectively treating them as pin connections. Knife plates are modelled as elastic elements in the axial direction of the brace to reflect the axial stiffness of the entire brace system with a length of 1 foot (304.8 mm). Inelastic springs discussed in the previous section are applied to both brace ends. The P-delta column and the frame are connected using an equal degree of freedom (DOF) constraint in the horizontal direction at each floor level. Viscous damping of 5.0% is applied.

The fundamental periods of the structure obtained modal analysis are as follows: 0.673 seconds for the 1st mode, 0.259 seconds for the 2nd mode, and 0.164 seconds for the 3rd mode. Because of the 2D modelling, all modes are translational.

4.3 ANALYSIS RESULTS AND DISCUSSIONS

In this study, we conducted a pushover analysis and incremental dynamic analyses using the FEMA-P695 far-field record set scaled to the Seismic Design Category C_{min} spectrum according to the FEMA P-695 methodology. Fig. 12 and Fig. 13 show example of dynamic analysis results using the earthquake NORTHR/MUL009. Fig. 12 shows the response history of the inter-story drift ratios at each story while Fig. 13 shows the force-deformation response of the top and bottom connections in the right brace at the first story (i.e., the location where the maximum deformation was recorded in any brace connection).

The results show that the inter-story drift at the first story is the largest, with a maximum response of about 0.012 radians. The maximum deformation response of the top connection was approximately -16.3 mm, while the ultimate deformation capacity of the connection is 22.9 mm. Although the demand deformation does not exceed the deformation at the maximum strength point, the connection does experience yielding. Collapse criteria is defined based on the maximum inter-story drifts. For reference, the allowable story drift for timber structures is defined as 2.0% [13], while the FEMA P-695 suggests 5% as a collapse-level drift ratio. The results herein indicate that the failure of the brace connections is likely to be more critical than the inter-story drift in defining the collapse of TBFs. In this study, brace connection failure is addressed as a non-simulated collapse limit state [16].

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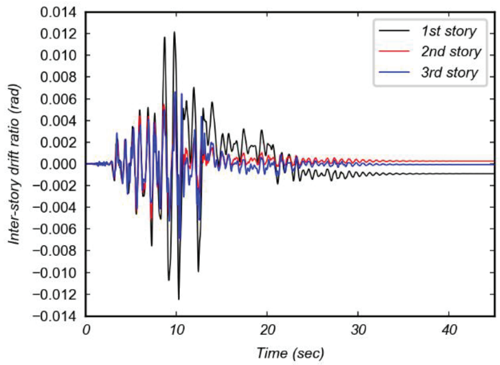

Figure 12. Inter-story drift ratio time hysteresis responses.

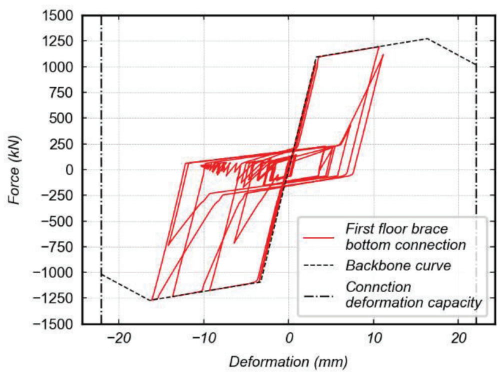

Figure 13. Hysteretic response for the bottom brace connections

FEMA P-695 requires that the collapse probability of structures due to the Maximum Considered Earthquake (MCE) be less than 10% on average for all archetypes within a performance group and be limited to 20% for any archetype. Instead of calculating these probabilities directly, the collapse margin ratio (CMR) is utilized to assess seismic performance factors in FEMA P-695. The CMR is defined by a median collapse intensity, where half of the earthquake record causes the collapse of the structure, divided by the MCE intensity. The CMR must be adjusted to account for the effects of spectral shape using the spectral shape factors (SSFs) of FEMA P-695, after which it is denoted the adjusted collapse margin ratio (ACMR) The ACMR is compared with acceptable ACMRs corresponding to 10% and 20%, probabilities of collapse which are derived based on assigned uncertainties. In this study, all uncertainties are assumed to be "Good," leading to a total system uncertainty of 0.52. The details of the calculation method and theory can be found in FEMA P-695[16]. Table 4 shows the CMR, SSF, ACMR, and acceptable ACMR 10% and 20%. The ACMR is significantly less than the acceptable ACMR 10% and 20% values. Therefore, the results indicate that a TBF with an R factor of 3.0 and the given connection deformation capacity does not meet the FEMA P-695 criteria.

Table 4. *Adjusted collapse margin ratio and acceptable values.*

CMR	SSF	ACMR	Acceptable ACMR 10%	Acceptable ACMR 20%
1.206	1.081	1.30	1.94	1.56

5 – CONCLUSION AND FUTURE WORK

Despite the potential benefits of timber braced frames, the use of this structural systems remains limited due to the absence of seismic performance factors in the U.S. building codes. Through assessment of existing test data and numerical analysis, the conclusions of this study are as follows:

1. Experimental test data on a bolted connection at the timber brace ends were calibrated without losing structural characteristics, using a trilinear curve and *Pinching4* hysteresis model. The connection model was incorporated into a full frame analysis model for collapse assessment.
2. A three-story office archetype building incorporating timber braced frames was developed with trial seismic performance factors, $R = 3.0$, $\Omega_0 = 2.0$, and $C_d = 3.0$.
3. Incremental dynamic analysis revealed that the deformation capacity of the brace connection in the archetype studied would lead to structural collapse before other collapse modes.
4. Collapse assessments in accordance with FEMA P-695 indicate that the archetype studied, which is based on a specific connection design, does not meet the FEMA P-695 criteria.

This study is a first step in the FEMA P-695 process for qualifying timber braced frames for U.S. building codes. Future work will involve a comprehensive assessment of the structural system, including uncertainty evaluation and collapse margin calculations for additional archetypes. This process may include modifying trial seismic performance factors (e.g., lowering R) and/or exploring other brace connections that have higher deformation capacities in order to meet FEMA P-695 requirements.

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REFERENCES

[1] UNECE and FAO (Food and Agriculture Organization of the United Nations). “Forest products annual market review 2023–2024“. Geneva: UNECE and FAO. 2024.

[2] Palermo, A., Pampanin, S., Buchanan, A., Newcombe, M. (2005). Seismic design of multi-storey buildings using laminated veneer lumber (LVL). In Proceedings of the New Zealand Society of Earthquake Engineering Annual National Conference, Wairakei, New Zealand.

[3] Sarti, F. (2015). Seismic Design of Low-Damage Post-Tensioned Timber Wall Systems (Doctoral dissertation). University of Canterbury.

[4] van de Lindt, J. W., M. O. Amini, D. Rammer, P. Line, S. Pei, and M. Popovski. “Seismic Performance Factors for Cross-Laminated Timber Shear Wall Systems in the United States.” Journal of Structural Engineering, (2020), 146 (9): 04020172.

[5] Murphy, C., Pantelides, C. P., Blomgren, H. E., & Rammer, D. “Development of timber buckling restrained brace for mass timber-braced frames”. Journal of Structural Engineering, (2021), 147(5), 04021050.

[6] Dong, W., Li, M., Lee, C.-L., MacRae, G., & Abu, A. "Experimental testing of full-scale glulam frames with buckling restrained braces." Engineering Structures, (2020), 222, 111081.

[7] Gleason, T. M. “Evaluation of structural systems for mass timber buildings in low to moderate seismic regions” Master dissertation, Massachusetts Institute of Technology. 2019.

[8] Karsh, Eric. "Building the Earth Sciences Building at the University of British Columbia." Construction Canada, 1 Apr. 2013, https://www.constructioncanada.net/building-the-earth-sciences-building-at-the-university-of-british-columbia/img_1613/.

[9] M. Popovski, "Seismic performance of braced timber frames," Ph.D. dissertation, Dept. of Civil Eng., Univ. of British Columbia, Vancouver, BC, 2000.

[10] Malo, K. A., Abrahamsen, R. B., & Bjertnæs, M. A. "Some structural design issues of the 14-storey timber framed building “Treet” in Norway." European Journal of Wood and Wood Products, (2016), 74, 407–424.

[11] Z. Chen, M. Popovski, and P. Symons, "Solutions for Upper Mid-Rise and High-Rise Mass Timber Construction: Seismic Performance of Braced Mass Timber Frames - Year 1," Project No. 301013067, FPInnovations, Vancouver, BC, 2019.

[12] Baird, Z., Woods, J. E., Viau, C., & Doudak, G. Cyclic “Behavior of Bolted Glued-Laminated Timber Brace Connections with Slotted-In Steel Plates.” Journal of Structural Engineering, (2024), 150(7), 04024068.

[13] American Society of Civil Engineers (ASCE). “Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-22).” ASCE, Reston, VA. 2022.

[14] CSA Group. (2019). Engineering Design in Wood (CSA O86-19). Canadian Standards Association.

[15] Chen, Z., and M. Popovski. “Seismic Response of Braced Heavy Timber Frames with Riveted Connections.” J. Perform. Constr. Facil., (2021), 35 (5): 04021051.

[16] FEMA. “Quantification of building seismic performance factors. FEMA P695.” Washington, DC: FEMA. 2009.

[17] American Society of Civil Engineers. “Seismic evaluation and retrofit of existing buildings.” American Society of Civil Engineers. 2017.

[18] Bora, S., Sinha, A., & Barbosa, A. R. “Effect of wetting and redrying on performance of cross-laminated timber angle bracket connection.” Journal of Structural Engineering, (2021), 147(9), 04021121.

[19] AWC (American Wood Council). “National Design Specification (NDS) for Wood Construction (ANSI/AWC NDS-2018)”. American Wood Council. 2018.

[20] “CEN 1995-2 EN 1995-2:2004: Design of timber structures. Part 1–1: General - Common rules and rules for buildings”, Brussels, Belgium. 2004.

[21] Lowes, Laura Nicole. Finite element modeling of reinforced-concrete beam-column bridge connections. University of California, Berkeley, 1999.

[22] S. Mazzoni et al. “OpenSees command language manual” Pacific Earthquake Engineering Research (PEER) Center 2006.

[23] REACTS (Research on Engineering, Architecture & Construction of Timber Structures) website. https://tallwoodinstitute.org/reacts-consortium/ (accessed: March 30, 2025).

https://doi.org/10.52202/080513-0349