

Seismic Performance of Timber Frames with Timber Buckling-Restrained Braces

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Abstract: The building industry requires the development of a ductile lateral force-resisting system made of mass timber to allow mass timber to continue to grow in popularity as the primary structural building material. A timber frame with mass ply lam (MPL) beams and columns with a timber buckling-restrained brace (TBRB) was subjected to a quasi-static cyclic loading protocol defined according to the qualification procedure of conventional buckling-restrained braces (BRBs). Two of the TBRBs in this study included a carbon fiber-reinforced polymer (CFRP) wrap to help confine the two ends of the mass ply panel (MPP) casing and improve the performance of the brace. The experiments showed that the TBRB without any CFRP wrap met the requirements for qualification of conventional BRBs, and the mass timber braced frame was able to reach a drift ratio of 3.0%. The TBRB with CFRP wraps met the requirements for qualification of conventional BRBs; the confinement provided by the CFRP wraps postponed failure of the TBRB by enabling the steel core to reach a maximum strain of 3.1% in both tension and compression and the mass timber braced frame to reach a maximum drift ratio of 4.5%. DOI: [10.1061/JSENDH.STENG-12877](https://doi.org/10.1061/JSENDH.STENG-12877). © 2024 American Society of Civil Engineers.

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Introduction

A recent resurgence in the interest in building with wood as the primary structural material has been largely caused by its sustainable characteristics, construction benefits, and architectural appeal. Moreover, engineers are able to take advantage of the high strength-to-weight ratio of timber due to the development of engineered wood products such as laminated veneer lumber (LVL), cross-laminated timber (CLT), glue-laminated timber, mass ply lam (MPL), and mass ply panel (MPP). US building codes have addressed the growing interest in timber by defining three new building types that allow mass timber construction in taller buildings up to 18 floors (ICC 2021).

A primary challenge for designing mass timber buildings in regions with high seismic risk is the lack of a well-defined ductile timber lateral force-resisting system (LFRS) in the building codes. One popular wood LFRS is shear walls; while light-frame wood shear walls are commonly used in residential buildings, they are not a practical solution for tall wood buildings because they require a low height-to-length ratio to effectively transfer lateral loads to the foundation without significant building damage. There has been

significant effort toward improving the ductility of CLT shear walls through post-tensioning, U-shape flexural plates, and improved base connections (Fitzgerald et al. 2020; Ganey et al. 2017; Morrell et al. 2020; Popovski and Gavric 2015). Furthermore, van de Lindt et al. (2020, 2022) recently completed a study to define the overstrength (Ω), deflection amplification (C_d), and response modification (R) factors for two generic CLT shear wall connections, and seismic design procedures have been developed for CLT shear walls (Lo Ricco et al. 2021; Busch et al. 2022).

Another lateral system traditionally used in wood buildings is the conventionally braced timber frame. This system resists lateral loads with large timber diagonal members connected to the frame at beam-column joints using steel plates and screws or bolts. The connections in this system are the primary energy dissipators and therefore provide limited ductility and displacement capacity (Cao et al. 2021; Xiong and Liu 2014; Popovski et al. 2003). Because damage to the wood in these connections is irrecoverable, a large earthquake would likely require replacement of the building, which is costly for building owners.

The limited number of defined LFRSs for wood structures in building codes leads designers to combine wood gravity systems with structural steel or reinforced concrete lateral systems. One type of structural steel LFRS that has been used in combination with a timber gravity system is the buckling-restrained brace frame (BRBF). The conventional buckling-restrained brace (BRB) consists of a steel core restrained by a concrete-filled steel tube. The steel core has a yielding region with a reduced cross section to ensure the inelastic response of the brace occurs within the restraining element, which controls the local and global buckling of the brace. By restraining the steel core, the casing allows the BRB to achieve approximately equal yield strengths in both tension and compression (Kersting et al. 2015). Extensive studies on BRBs in recent years show they can withstand strong earthquakes by efficiently dissipating energy and achieving high strains (Black et al. 2004; Fahnestock et al. 2007; Wu et al. 2014; Xu and Pantelides 2017). A challenge with

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integrating materials in this manner is that it can lead to complex and expensive buildings because of the different material performances and increased coordination efforts from independent material trades. These compatibility issues can be mitigated when one material is used as the primary structural material for the LFRS and the gravity system; therefore, it is desirable to develop a tall mass timber LFRS as a solution for tall mass timber buildings.

To develop a mass timber LFRS with similar strength, ductility, and energy-dissipation capabilities as the traditional BRB, researchers have been developing a timber BRB (TBRB). Murphy et al. (2021) completed some of the first successful TBRB component tests, which utilize MPP blocks sandwiched around a steel yielding core as the restraining element. In those tests, the TBRB achieved high strains (approximately 3.9%) and exceeded the cumulative inelastic ductility requirements established by AISC 341 (AISC 2016) for conventional BRBs. Takeuchi et al. (2022) completed a series of cyclic tests on TBRBs with glulam restrainers and presented more detailed design methods. Other studies of TBRBs include the use of carbon fiber-reinforced polymer (CFRP) composites to fasten the assembly together (Song et al. 2022) and with cross-shaped cores (Luo et al. 2022). Tests of full-scale mass timber frames with conventional BRBs showed that mass timber frames with a BRB can achieve almost twice the ductility compared to a timber frame with a conventional timber brace (Dong et al. 2020). Recently, cyclic tests of timber beam-column connections with steel dowels were carried out by Williamson et al. (2023b). Results from these tests were integrated into the design of a single-story timber braced frame specimen with a TBRB; a series of four tests using TBRBs in a single-story timber frame was recently completed by Williamson et al. (2023a), who found that the TBRB frame (TBRBF) is able to achieve 2.8% story drift before TBRB failure.

The purpose of this paper is to provide a more comprehensive understanding of the TBRBF system performance according to seismic provisions for conventional BRBs provided in AISC 341 (AISC 2016). This is achieved by completing a series of TBRBF quasi-static cyclic tests, including some with TBRBs utilizing CFRP wraps at the casing ends to provide additional strength in the perpendicular-to-grain direction. In addition, the timber frame was tested on its own as a bare frame under quasi-static cyclic loads to provide improved understanding of the toughness of the subassembly system.

Experimental Program Description

The first testing series and design of the TBRBF was described in Williamson et al. (2023a). The timber frame is shown in Fig. 1(a), with a story height of 2.29 m (7 ft 6 in.) and column spacing of 3.05 m (10 ft); these frame dimensions were selected based on the dimension limitations of the testing facilities and the frame is considered to be two-thirds scale. The frame had a single diagonal TBRB [Fig. 1(b)] designed to have an axial yield strength of 178 kN (40 kip). New classes of CLT and LVL known as MPP and MPL were used for the TBRB casing and the frame members, respectively. These types of materials result in members of repeated 25.4-mm (1.0-in.) layers consisting of veneers with varying cross-grain directions. The material properties and dimensions of the components of the frame are specified in Table 1. The design approach used for the dowelled connection utilizes the dowel-type connection equations from National Design Specification (NDS), AWC (2018a, b) and the equivalent model presented in Fan et al. (2011) to ensure the connections could resist the expected forces from the TBRB (Williamson et al. 2023b). A strain hardening factor, ω , of 1.45 and a compression adjustment factor, β , of 1.25

(Murphy et al. 2021) were used to determine the expected forces from the TBRB brace. To ensure the connections could meet the tight construction tolerance required for the efficient performance of the frame, the beams, columns, MPP casing, and steel connecting plates were manufactured with a computer numerical control (CNC) machine.

Two of the TBRBF tests utilized enhanced TBRBs to improve the performance of the TBRBF. The TBRBs were enhanced by wrapping the ends of the MPP casing with one layer of CFRP wrap 635 mm (25 in.) wide [Fig. 1(c)], with the cured composite properties in Table 1. The CFRP wrap was added to confine the MPP (Moran et al. 2019) and increase its tensile strength in the perpendicular-to-grain direction and avoid the premature splitting of the casing observed during previous tests (Williamson et al. 2023a). The width of the CFRP wrap was selected to provide confinement of the MPP casing through the transition zone of the steel yielding core, as shown in Fig. 1(b). The design of the CFRP wrap was determined based on the expected bulging forces applied on the MPP casing by the steel yielding core when it experiences weak-axis buckling; due to the short length of the TBRB used in the present tests, only weak-axis buckling was considered. The equations for determining the bulging force, $P_{d,w}$, were presented in Takeuchi et al. (2022) as

$$P_{d,w} = \frac{4N_{cu}(2s_{rw} + \nu_p t_c \varepsilon_t)}{l_{pw} - 4N_{cu}/K_r} \quad (1)$$

where N_{cu} = maximum core compression force; s_{rw} = weak-axis gap between the core and the MPP casing; l_{pw} = weak-axis wavelength estimated to be approximately from 9 to 11 times t_c (Wu et al. 2014; Murphy et al. 2021); t_c = thickness of the steel core; and K_r = restrainer stiffness, defined in Eqs. (2)–(5). The tensile strain, ε_t , is the maximum strain expected in the TBRB steel core. For the design of the CFRP wrap, ε_t was assumed to be 2.0% and N_{cu} was assumed to be 267 kN (60 kip) based on the maximum strains and tensile forces achieved by the TBRBs in the testing series performed by Williamson et al. (2023a)

$$K_r = \frac{1}{1/K_w + 1/K_b} \quad (2)$$

$$K_b = \frac{E_b A_{\text{eff}}}{D_r/2} \quad (3)$$

$$K_w = \frac{E_w l_{pw} B_r}{D_{rh}} \quad (4)$$

$$A_{\text{eff}} = 2 \times \frac{\pi d^2}{4} \left(\frac{l_{pw}}{l_b} \right) \quad (5)$$

where K_b = stiffness of the bolt; K_w = stiffness of the wood; E_b and E_w = elastic moduli of the bolt and MPP, respectively; D_r = depth of the TBRB assembly; B_r = width of the restrainer; D_{rh} = depth of one MPP casing block; A_{eff} = effective area of the bolts; and l_b = spacing of the bolts.

Once the bulging force, $P_{d,w}$, was determined, it was used to find the number of layers of the CFRP wrap required to provide adequate tensile strength and confinement using the following equation:

$$n = \frac{P_{d,w}}{2F_t w_f t_f} \quad (6)$$

where n = number of required layers; F_t = design tensile stress of the CFRP composite; w_f = width of the CFRP wrap; and t_f = thickness of a single layer of the CFRP wrap. The dimensions and

Table 1. Material properties

Component	Dimensions	Material properties
Beams and columns	Width = 389 mm (15–5/16 in.) Depth = 381 mm (15 in.)	F16 MPL ^a $E = 11 \text{ GPa}$ (1,600 ksi)
Dowels	Diameter = 13 mm (1/2 in.) Length = 381 mm (15 in.)	$F_y = 248 \text{ MPa}$ (36 ksi)
Connection plates	Thickness = 13 mm (1/2 in.)	$F_y = 248 \text{ MPa}$ (36 ksi)
TBRB steel core	Total length = 2,946 mm (116 in.) Yielding zone = 1,549 mm (61 in.) $13 \times 51 \text{ mm}$ (1/2 $\times$ 2 in.) Ends = $13 \times 178 \text{ mm}$ (1/2 $\times$ 7 in.)	$F_y = 310 \text{ MPa}$ (45 ksi) $\nu_p = 0.5$
TBRB casing	$254 \times 152 \times 2,426 \text{ mm}$ (10 $\times$ 6 $\times$ 95.5 in.)	F10 MPP ^a
TBRB bolts	Yielding zone = $13 \times 356 \text{ mm}$ (1/2 $\times$ 14 in.) Spacing = 152 mm (6 in.) Ends = $6 \times 356 \text{ mm}$ (1/4 $\times$ 14 in.) Spacing = 76 mm (3 in.)	$F_y = 248 \text{ MPa}$ (36 ksi) $F_y = 248 \text{ MPa}$ (36 ksi) $E = 199,948 \text{ MPa}$ (29,000 ksi)
CFRP wrap	Ply thickness (t_f) = 1 mm (0.04 in.)	Tensile strength = 1,248 MPa (181 ksi) Tensile modulus = 71.7 GPa (10,400 ksi)

^aAPA (2022).

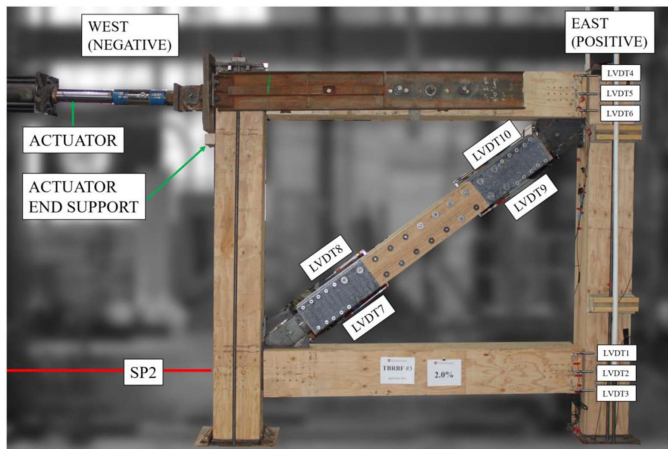


Fig. 2. TBRBF loading and instrumentation.

after the 50 cycles, the loading protocol returned to increasing displacement steps until failure.

- Incremental displacement: In this loading protocol, shown in Fig. 3(b), the peak displacement was increased until failure.

The intention of the incremental displacement loading protocol was to determine the maximum strains the TBRB is able to achieve, and the intention of the fatigue loading protocol was to use the cumulative inelastic deformation (CID) factor from these tests to show the TBRB can meet the requirements defined by the AISC seismic provisions (AISC 2016) for conventional BRBs. TBRB1 and TBRB3 were subjected to the fatigue loading protocol, while TBRB2 was subjected to the incremental displacement loading protocol.

In addition to the three TBRB frame tests, the timber frame was tested as a bare frame without a brace according to the incremental displacement loading protocol. Table 2 summarizes the completed tests including the brace enhancement details and the loading protocols. The bare frame test was completed to determine its contribution to the subassembly system and its lateral force and drift capacity.

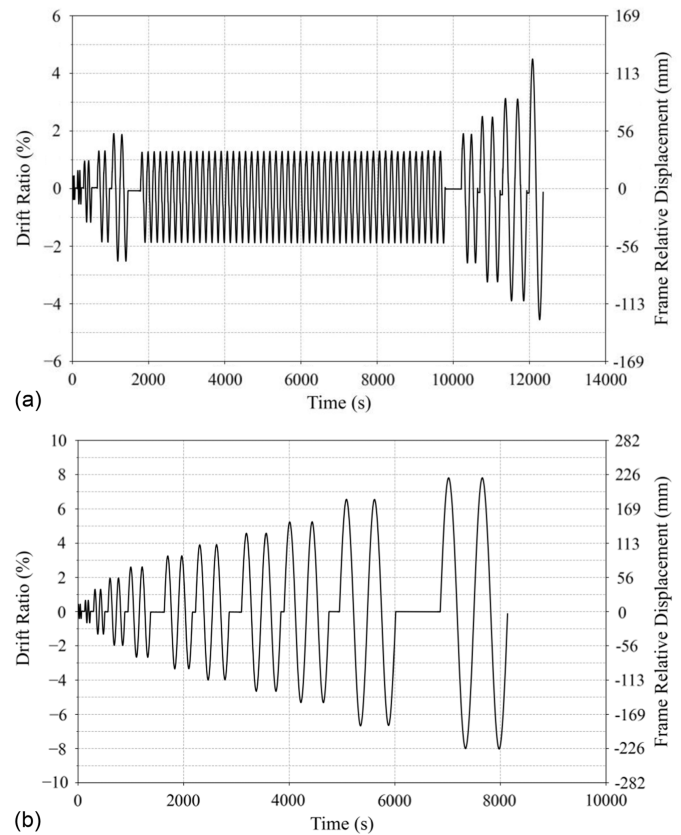


Fig. 3. Loading protocols: (a) fatigue; and (b) incremental displacement.

Table 2. Test matrix

Test name	Test type	Brace enhancements	Loading protocol notes
TBRB1	TBRBF	None	Fatigue
TBRB2	TBRBF-enhanced	CFRP wrap	Incremental displacement
TBRB3	TBRBF-enhanced	CFRP wrap	Fatigue
BF	Bare frame	N/A	Incremental displacement

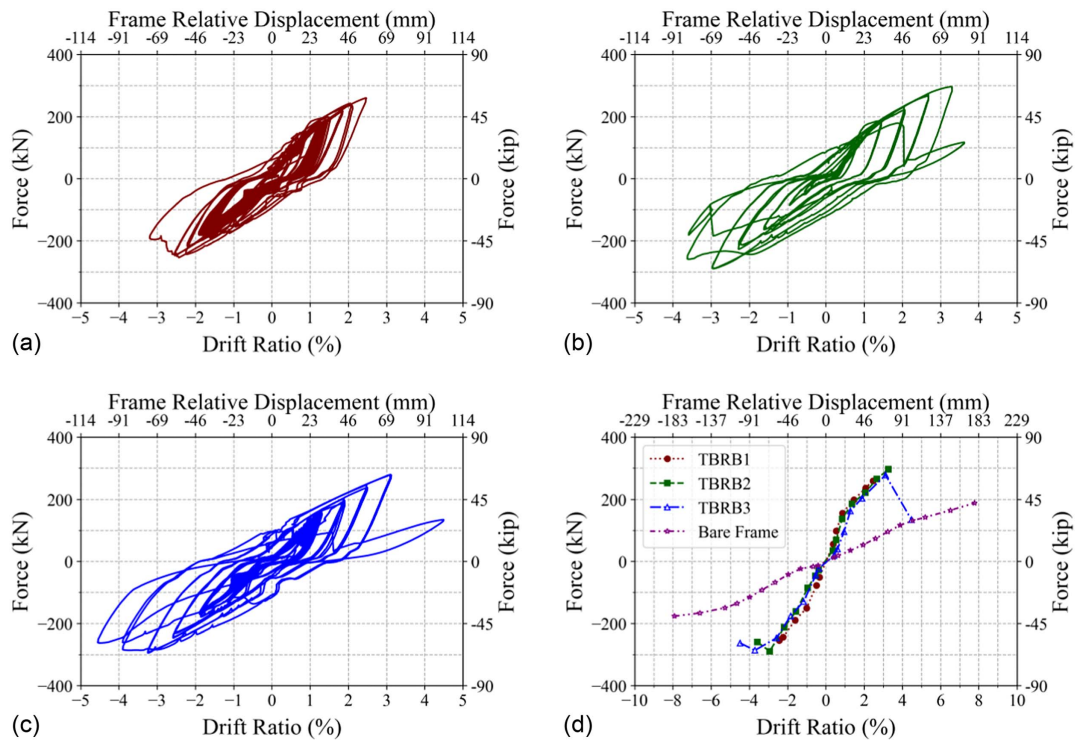


Fig. 4. Experimental test results: (a) TBRB1 hysteresis; (b) TBRB2 hysteresis; (c) TBRB3 hysteresis; and (d) force–displacement envelopes of all tests.

TBRBF Experimental Results

A timber frame was tested with three different TBRBs utilizing two quasi-static cyclic loading protocols: the fatigue loading protocol and the incremental displacement loading protocol. Two of the TBRBs were enhanced by wrapping the casing ends with one CFRP layer. The story drift ratio was calculated by dividing the relative displacement of the frame by the story height of 2.29 m (7 ft 6 in.). Moreover, the relative displacement of the frame was determined by subtracting the horizontal displacement measured by the string potentiometer, SP2 (Fig. 2), from the horizontal displacement measured by the hydraulic actuator.

Hysteretic Performance

Test TBRB1 was subjected to the fatigue loading protocol [Fig. 3(a)]; the TBRBF reached a maximum story drift ratio of 3.1% [Fig. 4(a)]; failure of the TBRB was due to splitting of the timber casing as shown in Fig. 10(a). Test TBRB2 was subjected to the incremental displacement loading protocol [Fig. 3(b)]; the TBRB failed during the second cycle defined by a maximum story drift ratio of 3.6% [Fig. 4(b)]. The TBRB in Test TBRB2 experienced fracture of the steel core in two locations; first, when the brace was in tension, in a tensile fracture [marked “Core Fracture #1” in Fig. 8(b)], and second, when the brace was in compression, in a compressive fracture [marked “Core Fracture #2” in Fig. 8(b)]. TBRB3 was subjected to the fatigue loading protocol; the TBRB failed during the first cycle defined by a maximum story drift ratio of 4.5% during the failure cycle [Fig. 4(c)]. The failure mode of the TBRB in TBRB3 was also tensile fracture of the steel core, which is indicated in Fig. 8(c).

The *Hazus Earthquake Model Technical Manual (FEMA 2022)* defines the drift limit for various levels of damage to a building after an earthquake. For wood buildings for commercial use greater than 465 m² (5,000 ft²), 1.2% and 4.0% interstory drift ratios are

associated with moderate and extensive damage to the building, respectively. Test TBRB3 was able to achieve a drift ratio in the range of the extensive damage state, while Tests TBRB1 and TBRB2 achieved story drifts more closely related to the moderate damage state.

Minimal pinching was observed in the hysteresis loops of all TBRBF tests, caused by the slip in the system as the dowels in the beam-column connections shift within the holes when the load is reversed. A drop in lateral force before TBRB failure was observed; this is caused by fatigue cycles affecting the connections of the frame to the strong floor, such as welds breaking during Tests TBRB1 and TBRB2 and loosening of bolts attaching the TBRBF to the strong floor during all three tests.

The lateral force–displacement envelopes for each test are illustrated in Fig. 4(d) and the maximum forces in tension and compression are summarized in Table 3. Conventional BRBs typically develop higher compressive forces compared with tension forces. The maximum forces in tension and compression are similar for the first two tests, while Test TBRB3 achieved a slightly higher force in compression compared with tension. The first two tests achieved more balanced forces in tension and compression because they are experiencing lower compressive strains than the third test and are therefore unable to experience the full effects of strain hardening. Moreover, some damage to the connections of the TBRBF to the strong floor observed during Tests TBRB1 and TBRB2, under the east column, may have prevented the frame from developing the full compressive force.

Cumulative hysteretic energy for each test is plotted in Figs. 5(a and b). Test TBRB2 and the bare frame used the incremental displacement protocol and are plotted together in Fig. 5(a); Tests TBRB1 and TBRB3 used the fatigue loading protocol and are plotted in Fig. 5(b). The sequence of testing was as follows: Test TBRB1, followed by Test TBRB2, then Test TBRB3, and finally the test of the bare frame. Test TBRB1 was able to dissipate slightly

Table 3. TBRBF and bare frame test results

Test name	Maximum story drift ratio (%)	Maximum positive force [kN (kip)]	Maximum negative force [kN (kip)]	Maximum tensile strain ratio	Maximum compressive strain ratio	CID
TBRB1	3.1	258 (58)	254 (57)	10.3	8.9	315
TBRB2	3.6	298 (67)	289 (65)	20.0	10.0	112
TBRB3	4.5	280 (63)	294 (66)	20.0	20.0	333
BF	8.0	187 (42)	178 (40)	N/A	N/A	N/A

Note: Maximum tensile strain ratio = maximum tensile strain/yield strain; and maximum compressive strain ratio = maximum compressive strain/yield strain.

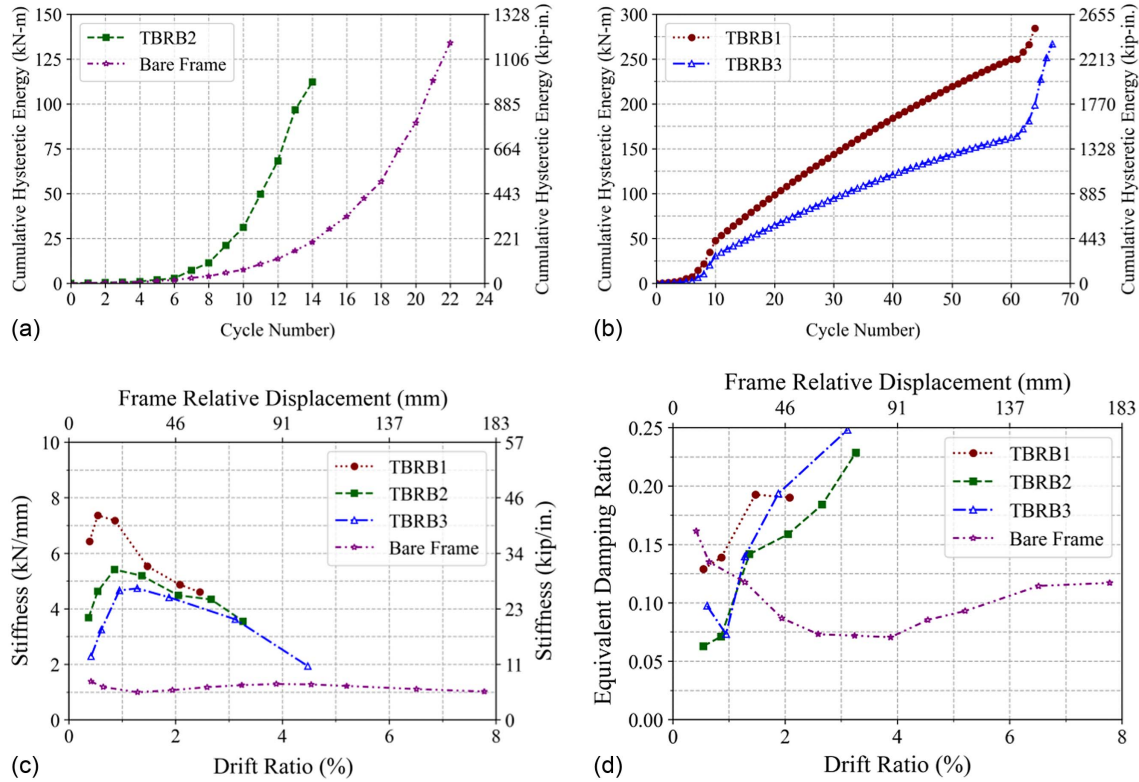


Fig. 5. Test results: (a) cumulative hysteretic energy–incremental displacement; (b) cumulative hysteretic energy–fatigue; (c) stiffness degradation; and (d) equivalent damping ratio.

higher hysteretic energy during the fatigue cycles than Test TBRB3 because the stiffness of the system at that drift level was slightly higher.

Stiffness

The stiffness degradation of the system was studied by calculating the secant stiffness for the first cycle of each displacement step [Fig. 5(c)]. In all three tests, the TBRBF started with a lower stiffness and reached a peak stiffness between the 0.5% and 1.0% drift ratios. It is likely that the low initial stiffness of the frame was caused by the manufacturing tolerances of the timber members; this type of low initial stiffness is common for dowelled timber connections (Williamson et al. 2023b; Dong et al. 2021). The subassembly in TBRB1 showed the highest stiffness values because it was the first test conducted. The same timber frame was used for all tests, so the connections were slightly damaged in Tests TBRB2 and TBRB3; this is the reason the subassembly in the second and third tests started with a lower stiffness and was not able to achieve the same stiffness as Test TBRB1. Additionally, there was a small construction defect in assembling the TBRB for Test TBRB3, which created a slightly larger gap between the steel core and the MPP casing and likely contributed to the lower stiffness of the TBRBF in Test TBRB3.

Equivalent Damping Ratio

Equivalent viscous damping, ξ_{eq} , provides insight on the relative energy dissipation capacity when the frame is subjected to cyclic loads; it considers both the strain energy of the cycle, E_{so} , and the hysteretic energy of the cycle, E_D . The strain energy is calculated as

$$E_{so} = \frac{K_{TBRBF} u_o^2}{2} \quad (7)$$

where K_{TBRBF} = stiffness of the TBRBF calculated for the cycle as described previously; and u_o = maximum displacement achieved during the cycle. The equivalent viscous damping ratio can then be calculated as follows (Chopra 2007):

$$\xi_{eq} = \frac{E_D}{4\pi E_{so}} \quad (8)$$

The equivalent viscous damping ratio was evaluated for the first cycle of each displacement step beginning at 0.5% story drift ratio [Fig. 5(d)]. At the beginning of the test, the equivalent damping ratio ranged from 6% to 12% of critical damping. As the test progressed, the damping ratio increased to 19%, 23%, and 35% for Tests TBRB1, TBRB2, and TBRB3, respectively. The increase of the equivalent damping ratio to approximately 20% during Tests TBRB1

and TBRB2 is in good agreement with the equivalent damping ratio determined for other timber structures (Filiatrault et al. 2003).

TBRBF Damage Observations

Upon the completion of each TBRB frame test, the frame was inspected to ensure no significant damage existed in the gusset plates and connections. The rotation of the connections in the TBRB frame tests in Fig. 6 was monitored with LVDTs attached to the connections on the east side of the frame (Fig. 2). The rotations in the present three tests are consistent with the rotations observed during the previously completed TBRB frame tests (Williamson et al. 2023a).

Throughout the entire testing series, no damage was observed in the gusset plates or the pins used to connect the TBRB to the frame.

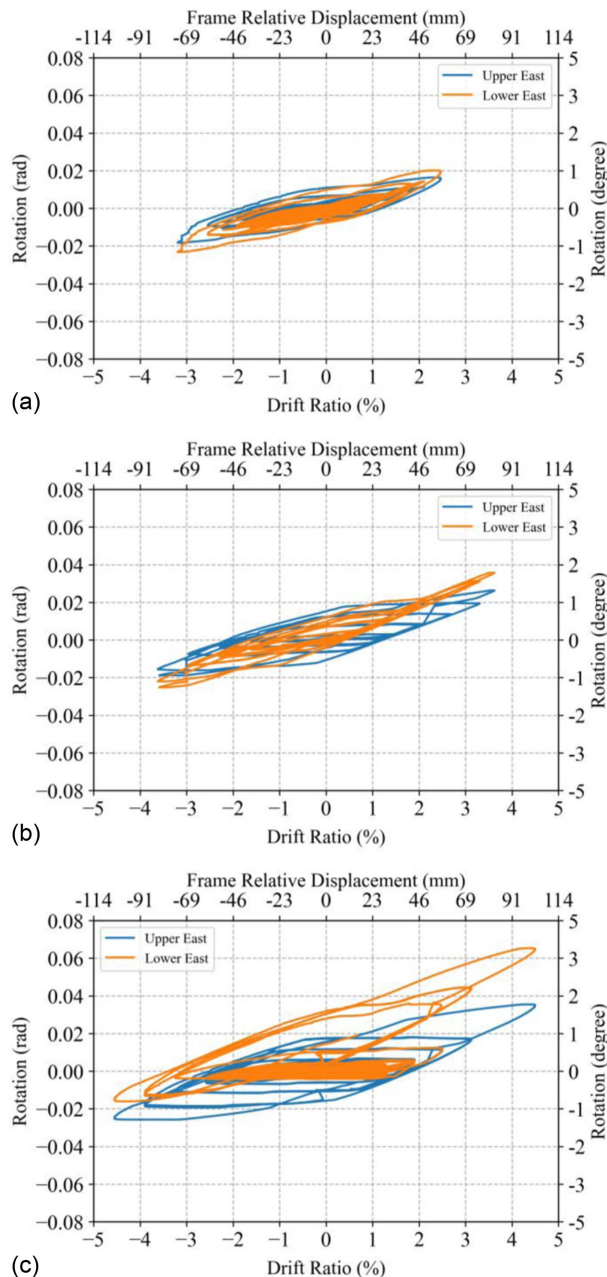


Fig. 6. East connection joint rotations during TBRB frame tests: (a) TBRB1; (b) TBRB2; and (c) TBRB3.

Additionally, no external damage to the connections such as splitting or crushing of the MPL was observed.

TBRB Experimental Results

TBRB Strain and CID Factor

Extension due to tension and shortening caused by compression of the steel yielding core was visible throughout the loading sequence (Fig. 7). The strain in the TBRBs was monitored throughout the experiment with two LVDTs on each end of the TBRB, as shown in Fig. 2. To determine the average strain in the brace, an average value of displacement was calculated for the top and bottom pairs of LVDTs; these values were then added together and divided by the length of the yielding core. The strain in the TBRB for each test is depicted in Figs. 8(a–c) and the maximum strain ratios are summarized in Table 3. The TBRBs in Tests TBRB2 and TBRB3, which had the CFRP wrap at the two TBRB ends, were able to achieve twice the tensile strain compared with TBRB 1; moreover, the TBRB in Test TBRB2 experienced a slight increase in compressive strain compared with the TBRB in Test TBRB1, while the TBRB in Test TBRB3 achieved twice the compressive strain of the TBRB in Test TBRB1. TBRB3 was able to achieve a balanced maximum strain in both tension and compression equal to 20 times the yield strain. TBRB3 likely recorded a larger compressive strain compared with TBRB2 because of the second fracture of the steel yielding core that occurred during Test TBRB2. This second core fracture caused the steel yielding core to dig into the MPP casing instead of shortening in compression. Conversely, the core in Test TBRB3 did not fracture in compression; instead, the steel core developed a high-amplitude wave near the end of the yielding zone and therefore recorded higher compressive strains than Test TBRB2. The increase in strain recorded by the TBRBs with the CFRP wrap at the casing ends occurred because the confinement and tensile capacity provided by the CFRP wrap prevented the premature splitting of the casing observed for the TBRBs used in previous TBRBF tests (Williamson et al. 2023a), as well as the present Test TBRB1. Therefore, the timber casing was able to continue to restrain the steel core, which developed higher strains in both tension and compression and allowed the subassembly to achieve higher drift ratios. Additionally, the strains observed in tests with the CFRP wrap are consistent with the strains reported by Takeuchi et al. (2022) when the timber restrainer was similarly reinforced with external steel plates near the ends.

The AISC 341 seismic specifications (AISC 2016) require that conventional BRBs must achieve a CID factor (η) of at least 200 to



Fig. 7. Extension of TBRB core during Test TBRB1.

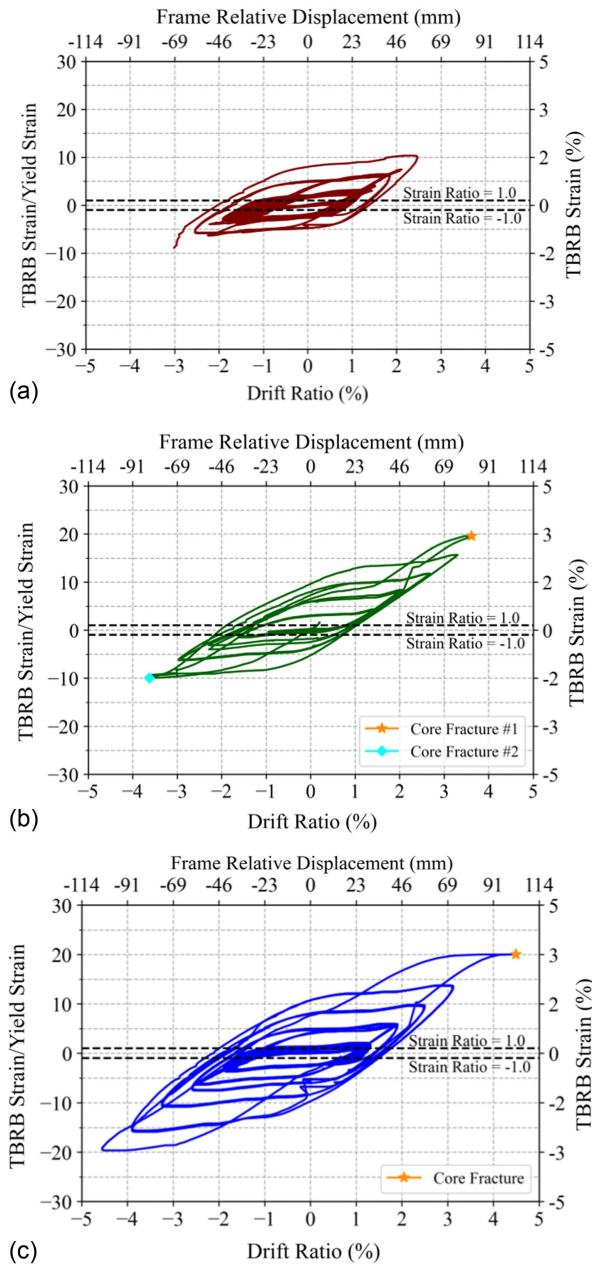


Fig. 8. TBRB strain versus frame relative displacement: (a) TBRB1; (b) TBRB2; and (c) TBRB3.

satisfy the requirements for BRB qualification. The CID is calculated as follows:

$$\eta = \frac{\sum |\Delta_{pi}^{\max} - \Delta_{pi}^{\min}|}{\Delta_y} \quad (9)$$

where $\Delta_{pi}^{\max}$ and $\Delta_{pi}^{\min}$ = maximum and minimum plastic deformations of the brace for each cycle; and Δ_y = yield deformation of the TBRB.

The CID for the three TBRBF tests is listed in Table 3. The TBRBs in Tests TBRB1 and TBRB3 surpass the qualification requirement with CIDs approximately 1.5 times the required value. In the component tests of TBRBs with a similar design to the TBRBs tested in the present research, a CID from 2.4 to 7.9 times the required CID for conventional BRBs was achieved (Murphy et al. 2021). It is likely the TBRBs tested in the present research did not

reach the same level of CID because the TBRB to beam-column connections introduced eccentric forces compared with the TBRBs in Murphy et al.'s tests, which were tested uniaxially. TBRB2 did not reach the 200 CID threshold because the incremental displacement protocol was not intended for the qualification of the TBRB. Although TBRB2 is lower than the 200 CID threshold, this brace would have likely met the requirement if it had been subjected to repeated cycles at a lower drift ratio similar to the manner in which the TBRBs in Tests TBRB1 and TBRB3 were tested.

In addition to the CID, the axial force in the TBRB in Test TBRB2 was estimated by subtracting the lateral force resisted during the bare frame test from the lateral force resisted during Test TBRB2. The axial forces were then used to determine the design factors for determining the expected forces of the brace: the compression adjustment factor, β , and the strain hardening adjustment factor, ω . Factor β was found to be 1.0 and factor ω was found to be 1.3; these values are in a similar range to the values determined during the timber BRB component tests completed by Murphy et al. (2021).

TBRB Failure Modes and Frame Observations

The TBRB frame at the conclusion of each test is shown in Fig. 9. The MPP casing of the TBRB in Test TBRB1 [Fig. 9(a)] is clearly damaged, while the damage to the TBRBs in Tests TBRB2 [Fig. 9(b)] and TBRB3 [Fig. 9(c)] is not visible because failure was due to fracture of the steel core within the TBRB. The failure mode for the TBRB in Test TBRB1 was weak-axis buckling of the steel core, which caused the MPP casing to split between the lamina [Fig. 10(a)]. The steel core developed eight waves along the yielding region with the largest amplitude being 48 mm (1 7/8 in.). This failure mode is consistent with the previously completed TBRBF tests (Williamson et al. 2023a); moreover, it is also consistent with the failure mode observed by Takeuchi et al. (2022) during the test of a 2.0 m (6 ft 7 in.) TBRB with just a bolted timber restrainer.

Tensile strengthening of the timber casing at the two ends with CFRP wraps eliminated the premature splitting of the timber casing and shifted the failure mode of the TBRB to steel core fracture. The TBRB in Test TBRB2 fractured twice [Fig. 10(b)]; first in a tensile fracture, during the tension phase of the cycle near the end of the transition zone annotated with a “1” in Fig. 10(b), and subsequently in a compressive fracture at the valley of the wave annotated with a “2” in Fig. 10(b); the amplitude of the wave at this location was 38 mm (1 1/2 in.), which created a large indentation in the MPP casing at this location [Fig. 10(c)]. The steel core of the TBRB in Test TBRB3 also fractured in tension [Fig. 10(d)], but it did not experience any core fracture in compression as observed in Test TBRB2; the amplitude of the wave near the fracture location was 76 mm (3.0 in.), which left a sizable indentation in the MPP casing [Fig. 10(e)]. Although the buckled steel core created significant damage to the inside of the MPP casing, the CFRP wrap provided the additional tensile strength at the ends to prevent the splitting of the entire casing and allowed the TBRBF to achieve higher drift ratios.

Consistent with the tests completed in an earlier testing series (Williamson et al. 2023a), the TBRBs in the present testing series experienced failure of the TBRB in the upper east end of the brace and are compared in Fig. 10(f). The main reason for this is the rigid nature of the upper east beam-column connection compared with the lower west connection. Because the base of the west connection is a roller support, the horizontal component of the force from the brace on that end is reduced because the column is allowed to displace while the upper east end must resist all the force.

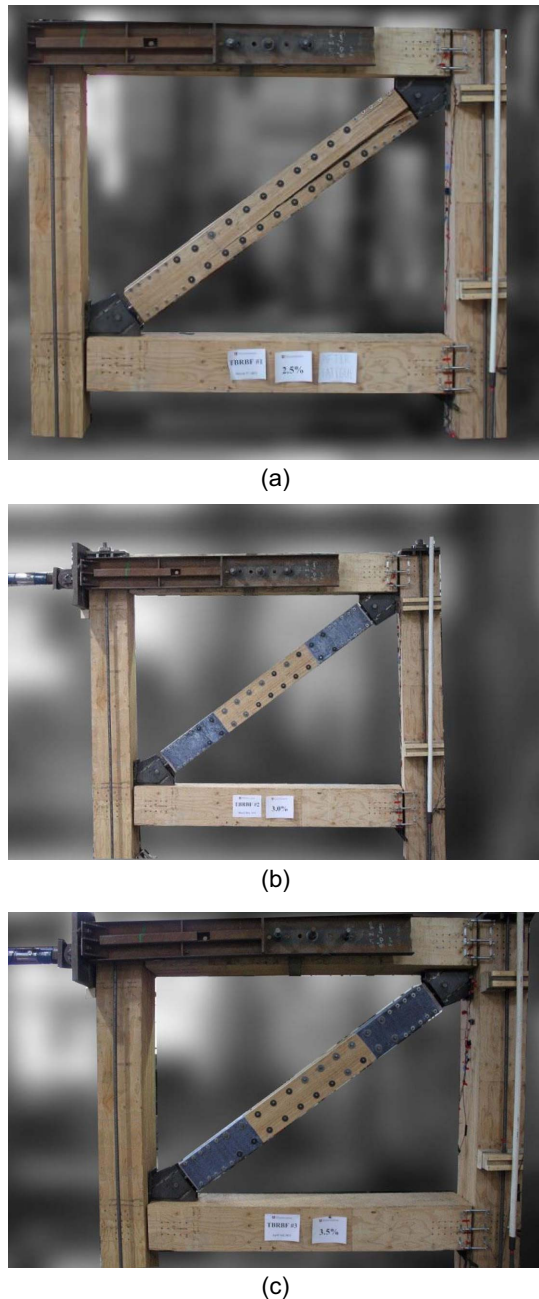


Fig. 9. TBRBF at end of test: (a) TBRB1; (b) TBRB2; and (c) TBRB3.

Bare Frame Experimental Results

Following the completion of the TBRBF testing series, the timber frame was tested by itself as a bare frame to provide more insight on its toughness. The bare frame test followed the incremental displacement loading protocol; the maximum horizontal displacement of the hydraulic actuator imposed on the frame was equivalent to a story drift ratio of 8.0%. The hysteresis of the timber bare frame (Fig. 11) resists slightly more force in the positive direction than the negative direction (Table 3). During Test TBRB2, which was subjected to the same loading protocol, the lateral force reached 298 kN (67 kip) in the cycle before the TBRB failed at 3.6% story drift ratio. At a similar drift ratio, the bare frame has a lateral force capacity of 95 kN (21 kip); this lateral force capacity is approximately 31% of the total lateral force resisted by the TBRBF with

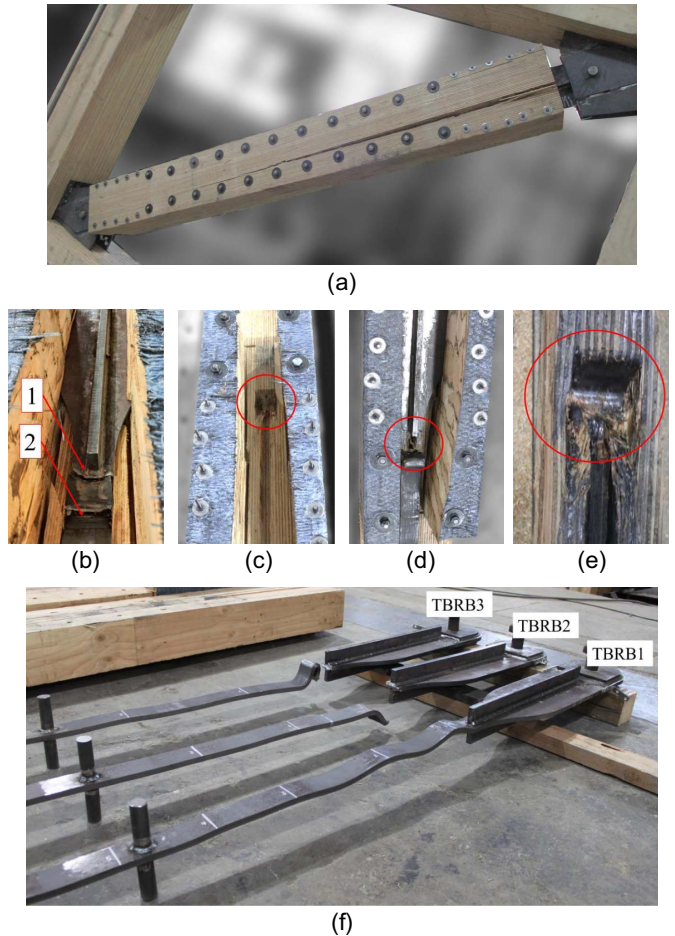


Fig. 10. TBRBF failure modes: (a) TBRB1 casing rupture; (b) TBRB2 core fracture; (c) TBRB 2 indented casing; (d) TBRB3 core fracture; (e) TBRB3 indented casing; and (f) buckling of cores.

the TBRB. This ratio is similar to the contribution made by the frame action of the beams and columns in a structural steel BRBF observed during biaxial cyclic tests completed by Palmer et al. (2013), which reached 28%. The contribution of the timber frame can also be seen through the force–displacement envelope of the bare frame [Fig. 4(d)] because the last positive envelope point in Test TBRB3 is after the TBRB yielding core had fractured. At this point the timber bare frame is resisting lateral force on its own and it intersects with the force–displacement envelope of the bare frame test.

The cumulative hysteretic energy dissipated by the bare frame [Fig. 5(a)] is 23 kN-m (203 kip-in.) compared with a cumulative hysteretic energy dissipation of 112 kN-m (992 kip-in.) by Test TBRB2 after the same number of 14 cycles. The stiffness [Fig. 5(c)] started out significantly lower than the TBRBF tests at 1.4 kN/mm (7.9 kip/in.). The low stiffness is partially because the connections had already experienced some local crushing of the timber during the previous tests; however, the stiffness of the frame remained relatively constant throughout the entire loading protocol, indicating that the timber bare frame remained somewhat elastic even at high drifts. The equivalent damping ratio of the timber bare frame [Fig. 5(d)] remains relatively low compared with the TBRBF tests, indicating that it is not as effective at dissipating hysteretic energy as the TBRBF subassembly. Similar to the TBRBF tests, the stiffness degradation of the timber bare frame was calculated.

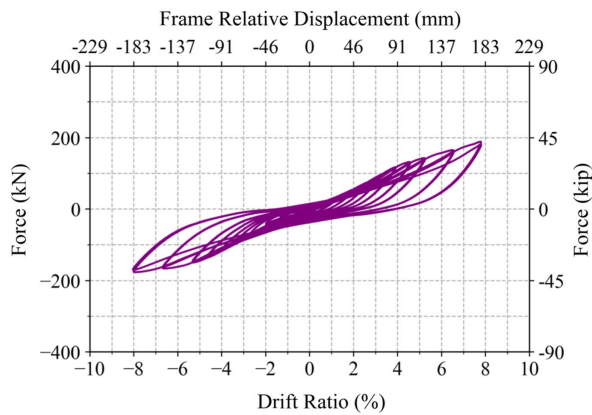
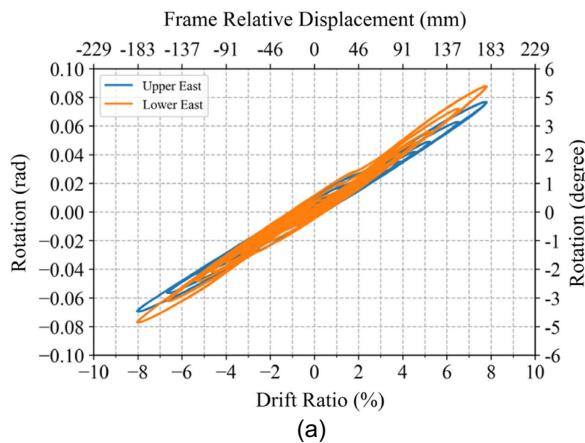


Fig. 11. Timber bare frame hysteresis.

The rotations of the beam-column connections in the east side of the frame were monitored with LVDTs (Fig. 2). The upper and lower connection achieved maximum rotations of 0.08 and 0.09 rad, respectively [Fig. 12(a)]. Despite achieving significant rotations, the connections did not exhibit any external damage to the MPL beam and column members [Fig. 12(b)]. This is consistent with the findings from a previously completed beam-column connection testing series with similar details in which the connections achieved 0.11 rad of rotation without external damage to the timber (Williamson et al. 2023b). Moreover, following the completion of the test, the distance



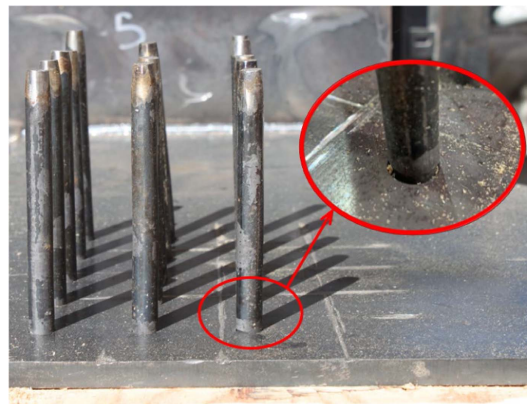
(a)



(b)



(c)



(d)

Fig. 12. Bare frame performance: (a) joint rotation; (b) bare frame at maximum negative drift; (c) dowel damage to top east column joint; and (d) dowel damage to bottom east column joint.

from the face of the timber column to the tip of select dowels was measured and indicated that the dowel ends on the south face of the frame had moved inward approximately 5.0 mm (0.2 in.). This movement indicates that the dowels had begun to yield and were likely exhibiting Yield Mode IV behavior, as observed in previous tests (Williamson et al. 2023b). Following the completion of this testing series, the top [Fig. 12(c)] and bottom [Fig. 12(d)] east connections were dissected to observe the internal damage of the connections. Both connections showed indications of dowel bending [Fig. 12(c)]; moreover, both connections had dowels with a reduced diameter at the steel plate interface [Fig. 12(d)]. The internal damage to the connections was relatively minor and occurred mostly in the corner dowels located within the column in both the top and bottom connections.

Practical Application Discussion

The results of the present study as well as previously completed research (Murphy et al. 2021; Takeuchi et al. 2022; Williamson et al. 2023a) show how TBRB can meet the requirements defined by AISC 341 (AISC 2016) for conventional BRBs. The stable hysteretic loops, significant hysteretic energy dissipation, and high strains achieved by TBRB show how it is a viable ductile lateral force-resisting system for new timber structures. Moreover, a capacity-based design approach, which is used by structural engineers for the design of conventional BRBs, was utilized for the design of the TBRB and the frame connections. This approach resulted in

a ductile response from the TBRBF and a conservative design for the connections; the connections experienced very limited damage throughout the experimental program. The authors recommend designing the timber frame as if the connections are pinned and the TBRB resists the lateral load on its own, similar to the design of conventional BRBFs. Further testing of full-scale TBRBs with various end conditions is also planned to help confirm the results of this study.

Conclusions

A series of TBRB frame quasi-static cyclic tests were completed to evaluate their seismic performance. Two of the tests included TBRBs with a CFRP wrap at the two ends of the casing to confine the MPP and improve the strength in the perpendicular-to-grain direction, thus preventing premature splitting of the casing observed in previous tests. Based on the results of the present tests, the following conclusions can be made:

- The fatigue loading protocol, which was adopted to show that TBRBs could meet the requirements for conventional BRBs, showed that both the original TBRB design and the TBRB enhanced with CFRP wraps can exceed the cumulative inelastic deformation value of 200 set for the qualification of conventional BRBs by the AISC 341 seismic specifications.
- The CFRP wrap alters the failure mode of the TBRB by preventing splitting of the MPP casing, thus allowing the subassembly to achieve interstory drift ratios high enough to fracture the steel core after significant yielding and large strains are achieved.
- Although internal crushing of the MPP by the steel yielding core still occurs in TBRBs with the CFRP wrap, the MPP casing does not split apart. This allows the TBRB frame to achieve higher interstory drift ratios.
- The TBRB frame tested with the fatigue loading protocol using a TBRB enhanced with a CFRP wrap at its two ends was able to achieve a maximum strain of 3.1% before tensile fracture, which is 20 times the yield strain of the steel core in both tension and compression, thus exhibiting balanced ductile performance.
- The TBRB frame tested with a TBRB enhanced with a CFRP wrap at its two ends was able to increase the achieved interstory drift ratio from 3.1% to 4.5%.
- The TBRB frame tested with a TBRB and subjected to the incremental displacement loading protocol resists approximately three times more lateral force compared with the timber bare frame on its own at the same interstory drift ratio of 3.6%.
- The TBRB improves the cumulative hysteretic energy capabilities of the subassembly by approximately five times compared with the timber bare frame after completing the same number of cycles of the incremental displacement loading protocol.
- Although the timber bare frame on its own has a relatively low stiffness and lateral force capacity compared with the TBRB frame, it is able to achieve high interstory drifts up to 8.0% without visible damage to the connections. This demonstrates the timber bare frame's toughness and indicates that even if a TBRB fractures within a TBRB frame in a large seismic event, it can be replaced following the earthquake while the timber frame remains intact.

Data Availability Statement

Some or all data, models or code that support the findings of this study are available from the corresponding author upon reasonable request.

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