

Design and qualification of timber buckling restrained brace for mass timber-braced frames

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ABSTRACT

A ductile lateral force resisting system (LFRS) for mass timber frames is proposed utilizing Mass Timber Buckling-Restrained Braces (MT-BRBs), with mass timber as the primary casing material. Compressive tests were conducted on Mass Plywood Panels (MPP) with and without horizontal self-tapping screws inserted perpendicular to laminations to assess differences in stiffness, peak load, and maximum displacement. Screws perpendicular to lamination enhanced performance, increasing the peak load by nearly four times and peak displacement by more than three times. Withdrawal tests were also performed on timber blocks with screws installed parallel or perpendicular to laminations. The withdrawal force when screws were placed parallel to lamination was 0.68 times that of the force when screws were placed perpendicular to lamination. Seven MT-BRBs 5.5 m (18 ft) long with a targeted tensile yield strength of 293.5 kN (66 kip) were designed and tested using two loading protocols; a fatigue protocol and a high strain protocol. Six braces were built using steel bolts to assemble the mass timber casing and screws were utilized perpendicular to lamination; the seventh brace used screws to assemble the mass timber casing and screws were utilized perpendicular to lamination. For the fatigue loading protocol, the cumulative inelastic demand (CID) achieved by the TBRBs was 2.9–6.9 times the value required by AISC 341. The maximum strain in the steel core for all six tests exceeded 13.3 times the yield strain, indicating excellent ductility. The MT-BRB assembled with screws did not achieve stable hysteresis curves and failed prematurely. The MT-BRBs assembled with bolts exhibited stable hysteretic behavior with a maximum strain of 3.1 %, which corresponds to an inter-story drift of 4.9 % for the prototype mass timber braced frame. The novelty introduced in this paper is the elimination of external steel plates or carbon fiber wraps used to postpone splitting of the timber casing; instead, screws perpendicular to lamination were utilized that were successful in postponing premature splitting failure of the MPP casing. In addition, the MPP casing was built without any hardwood spacers, or internal steel plates by incorporating a profile in the MPP casing thus simplifying construction. The fastener design of the MT-BRB was developed for both bolts and screws specifically for an MPP casing, and the overall MPP casing dimensions were checked against the NDS specifications for laminated built-up columns.

1. Introduction

Mass timber exists as large engineered wood in the form of glue-laminated and cross-laminated timber, and it is growing as a primary building material. The advantages of using timber in buildings include low cost, sustainability, light weight and high speed of construction. A mass timber buckling restrained brace (MT-BRB) is a hysteretic energy dissipator which adds ductility to mass timber braced frames and resists

lateral forces arising from earthquakes. The advantages of using timber as the casing, or restrainer, include its light weight, aesthetics, and ease of fabrication.

A cross-laminated timber (CLT) shear wall is one of the lateral force resisting systems (LFRSs) considered for mass timber buildings [1,2]. Successful shake table tests have been conducted on mass timber buildings with CLT walls as LFRS [3,4]. Timber frames can reduce seismic demands in areas of high seismicity because of their reduced

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weight; however, they possess lower stiffness compared to steel and concrete. An efficient LFRS is currently lacking for mass timber braced frame buildings. In an attempt to develop a LFRS for mass timber, post tensioning of mass timber systems was proposed [5]. A solution using conventional buckling-restrained boundary elements (BRBs) with mass timber walls detailed to pivot about a pinned base has also been proposed [6]. The application of resilient slip friction joint dampers for energy dissipation in both timber frames and CLT shear walls has also been explored [7]. Bolted glulam brace connections with internal steel plates have been studied to achieve system-level ductility in mass timber structures [8].

Many of the proposed systems achieve low ductility compared to timber buckling restrained braces which are well suited for high seismic regions because of their large deformation capacity beyond the yield strain of the steel core. A mass timber buckling restrained brace using glulam as the casing was tested for seismic regions [9]; it exhibited high axial load and stiffness and stable axial deformation in compression and tension. Extension of this work was carried out by testing MT-BRBs comprising of a steel core with a mass timber casing [10]; the hysteresis curves for all six MT-BRBs were stable, demonstrating excellent energy dissipation, while cumulative inelastic deformation exceeded two times the AISC 341 requirement [11]. Undesirable failure modes such as splitting along the glue line of the timber casing were observed during a shake table test [12] on a glulam frame structure with T-BRB.

Other LFRSs for mass timber frames include a timber-steel hybrid system [13], which consists of glulam frames and conventional steel-concrete BRBs. Mass timber frames with MT-BRBs or conventional BRBs have also been explored [14–16]. Experiments have demonstrated the viability of MT-BRBs to enhance the seismic performance of glulam frames [17]. The casing of a MT-BRB consists of two mass timber parts connected using bolts [10,14,18]. The behavior of glulam casing with self-tapping screws as a casing has been studied [19]; it was found that premature splitting of the glulam casing occurred and the MT-BRB did not meet the cumulative inelastic demand specified in AISC 341–16 [11].

Different failure modes associated with a timber casing were investigated [18]; weak-axis and strong-axis buckling was observed in timber BRBs with a different bolted casing, boundary conditions, steel core plate clearances, and reinforcing steel plate arrangements; design methods were developed to avoid these failure modes. Brittle failure of the timber casing was observed and efforts to prevent this failure mode are continuing.

Premature splitting of the timber casing along the glue line is one of the failure modes of MT-BRBs [14,18]; this is attributed to out-of-plane high mode buckling of the steel core. Studies describe how to prevent premature splitting failure; one of the methods uses short steel plates to connect the bolts on the exterior and interior of the timber casing [18]; another method uses a carbon fiber wrap at the two MT-BRB ends [15]. Additional research is required to prevent the splitting failure mode since the casing fails once this occurs and the MT-BRB loses its compression capacity.

This paper discusses the design and full-scale testing of MT-BRBs as ductile lateral force resisting elements for mass timber braced frames. Small-scale compressive tests were performed on mass timber specimens with and without self-tapping screws placed perpendicular to lamination. The goal was to determine the stiffness, load capacity, and deformation limits of the timber blocks, while also evaluating the effectiveness of self-tapping screws in preventing premature splitting of the timber casing; an alternative to internal steel plates or a carbon fiber wrap. Withdrawal tests of self-tapping screws were also carried out on mass timber specimens, both parallel and perpendicular to lamination. Subsequently, seven 5.5 m (18-ft) long MT-BRBs with a 293.5 kN (66 kip) tensile yield capacity were tested under cyclic load. Design elements include the mass timber material, the number of bolts or screws required to connect the two parts of the timber casing, and the number of screws placed perpendicular to lamination to prevent splitting. Six

tests utilized bolts and the seventh utilized screws as an alternative to bolts for connecting the two parts of the mass timber casing.

2. MT-BRB casing study

The strength and ductility of a MT-BRB largely depends on the mechanical performance of the casing material which forms the restrainer. Premature splitting of the casing leads to global buckling of the brace, compromising its load-carrying capacity. Selecting an appropriate casing material is critical. The compressive strength of various engineered wood products, including laminated veneer lumber (LVL), parallel-strand lumber (PSL), and mass plywood panel (MPP), was investigated for creating a MT-BRB casing [10]. These materials were tested under compressive load applied both parallel and perpendicular to the lamination direction—referred to as the X- and Y-direction, respectively. MPP loaded in the X-direction exhibited superior strength and stiffness. Consequently, MPP oriented parallel to lamination was chosen as the casing material for the present MT-BRB. Splitting of the timber casing along the lamination direction is a common failure mode of MT-BRBs. Splitting failure of MT-BRB casing occurs due to the high mode buckling of steel core. Fig. 1(a) illustrates the contact region between the steel core and the casing component and how the steel core exerts outward force on a timber casing. To study the mitigating measures for the splitting failure of timber casing, small scale tests were conducted on MPP blocks (Fig. 1(b)). The width (B_r) of a single MPP block was 254 mm (10 in.), the thickness (D_{th}) was 152 mm (6 in.) and the length was 305 mm (12 in.). Screws were installed perpendicular to lamination to evaluate their effectiveness to prevent splitting. Two grades of MPP material were explored in the experiments.

2.1. Short beam tests of timber blocks

Fig. 2(a) illustrates the experimental setup used for the short beam tests. The timber blocks measured $305 \times 254 \times 152$ mm ($12 \times 10 \times 6$ in.). A constant loading rate of 1.0 mm/min (0.04 in./min) was applied using a steel platen measuring 76×76 mm (3×3 in.); the timber block span was equal to 254 mm (10 in.). Monotonic tests were conducted on timber specimens with and without self-tapping screws. Mass

plywood panels from two different grades were used, namely MPP-1 with a modulus of elasticity of 6205 MPa (900 ksi) designated as F10, and MPP-2 with a modulus of elasticity of 11031 MPa (1600 ksi) designated as F16, as per APA Product Report PR-L325 [20]. The specimens were first tested as short beams without screws to determine their flexural strength. The test results suggest that the flexural strength for both grades was approximately the same so MPP-2 was adopted for further study. In subsequent tests, shown in Fig. 2(b), screws were inserted into the specimens perpendicular to lamination to reinforce the timber casing. Simpson Strong Tie strong drive screws – SDCFC, fully threaded with rolled heads were used. For a single screw, the head diameter was 15.0 mm (0.59 in.), the thread outer diameter was 8.0 mm (0.315 in.), the shank diameter was 5.9 mm (0.23 in.), the screw length was 220 mm (8.66 in.), and the specified yield strength was 1241 MPa (180 ksi). The allowable strength in tension was 8.9 kN (2.0 kip) and the allowable strength in shear was 6.2 kN (1.4 kip).

During the short beam test, the steel platen tended to bear into the wood casing, often leading to splitting along the lamination. Table 1 summarizes the peak load and actuator displacement for each configuration. Fig. 2(c) shows the splitting failure mode when screws are absent (set A in Table 1) and the bearing failure mode when screws are present (set B in Table 1). Fig. 2(d) shows that the actuator force for the blocks with screws was 3.2–3.4 times that of the blocks without screws; additionally, the actuator displacement for the blocks with screws was 2.8 times that of the blocks without screws. Inclusion of screws enhanced the resistance of the timber casing to splitting failure, suggesting that the casing could withstand a more localized global buckling of the steel core

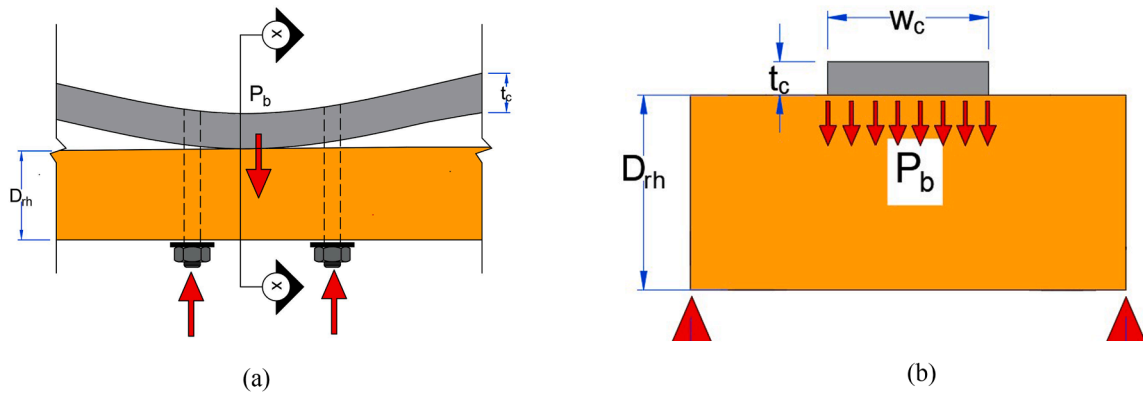


Fig. 1. Illustration of contact region between the steel core and steel buckling wave: (a) longitudinal direction; (b) cross-section at x-x.

before any splitting would occur. For the tests involving screws, a comparison was made between a 152 mm (6.0 in.) and a 254 mm (10.0 in.) screw spacing. The two screw spacings were examined to optimize the design of the MT-BRB; the aim was to maintain structural performance while improving cost efficiency. As shown in Fig. 2(c), the strength of the timber blocks was similar in both cases; however, the displacement capacity was higher for the 152 mm (6.0 in.) spacing.

2.2. Withdrawal strength of screws

This section presents the withdrawal test results for screws located parallel or perpendicular to lamination. Screws parallel to lamination could be used for connecting the two parts of the timber casing together, while screws perpendicular to lamination could be used to prevent splitting of the timber casing. Rothoblaas VGS fully threaded screws with countersunk head were used in both cases; the screws were selected due to their availability in various lengths and head diameters. A screw with a head diameter of 16.0 mm (0.63 in.), thread outer diameter (nominal diameter) of 9.0 mm (0.35 in.) and shank diameter of 5.9 mm (0.23 in.) was adopted, which is similar to the screws used in the short beam tests. Properties of VGS screws include the yield strength of 1000 MPa (145 ksi), tensile strength of 25.4 kN (5.7 kip) and yield moment of 27.2 N-m (0.24 kip-in.). The countersunk head thickness for a VGS 9 mm screw was 6.5 mm (0.25 in.). To prevent splitting VGS Φ -9mm/240 mm screws were used, and to connect the two timber casing parts of the casing VGS Φ -9mm/300 mm screws were used.

To evaluate the pullout capacity of the screws, withdrawal tests were conducted on MPP F-16 specimens in the parallel and perpendicular to lamination directions. According to ASTM D1761–20 [21], screws used in withdrawal tests must be embedded at a depth of at least nine times the diameter.

With a screw diameter of 9 mm, the required minimum embedment was 81 mm (3.2 in.); in the present tests, a screw embedment of 127 mm (5 in.) was used in both directions. The loading rate was set at 2.5 mm/min (0.1 in./min), according to ASTM D1761 guidelines. Fig. 3(a) presents the test setup for the screw withdrawal tests, while Fig. 3(b) provides a schematic drawing of the test configuration.

Three tests were conducted with screws parallel to lamination and three with screws perpendicular to lamination. Fig. 4(a) shows the withdrawal force versus displacement curves for screws loaded parallel and perpendicular to lamination. The screw head was clamped to the top circular plate during the test. At the inception of the test, the initial slip represents the engaging time between the top circular plate and the screw. Once the screw was fully engaged, the actuator force was resisted by the screws, trying to pull it apart and the slip disappeared. The failure modes for each test are shown in Fig. 4(b). In the parallel-to-lamination tests, the screws caused localized failure by pulling timber fibers around their edges. In contrast, in the perpendicular-to-lamination tests there was more significant damage, with screws tearing through the veneers.

Table 2 summarizes the results obtained in these tests. Withdrawal force parallel to lamination was 0.68 times the withdrawal force perpendicular to lamination. This value is within the range of the National Design Specification for Wood Construction (NDS) [22] recommended value of 0.75 which is based on a large number of tests. The withdrawal displacement parallel to lamination was 0.43 times the withdrawal displacement perpendicular to lamination. The results from the withdrawal tests show that screws are less effective when installed parallel to lamination in terms of withdrawal force and displacement. The peak withdrawal force was used to find the screw spacing for the MT-BRB design.

3. Layout and design of MT-BRB specimen

A MT-BRB consists of a steel core plate and the two parts of the timber casings that are held together by either bolts or screws that form the restrainer. A MT-BRB with a length of 5.5 m (18 ft) was

designed, and a total of seven specimens were built and tested. Six specimens were assembled using bolts to connect the two parts of the mass timber casing and reinforcing self-tapping screws perpendicular to lamination. Three of the six specimens were subjected to a fatigue loading protocol, while three underwent a high-strain loading protocol. The seventh MT-BRB was constructed using self-tapping screws to assemble the mass timber casing; self-tapping screws were also applied perpendicular to lamination and this MT-BRB was tested under the fatigue loading protocol. In order for the MT-BRB to meet the same standards as a conventional BRB, it must satisfy the CID requirement provided by AISC 341–16 [11]. In addition, according to ASCE/SEI 41–23 [23], a well-designed BRB should sustain plastic strains 10 times the yield strain for the life safety performance level, and 13.3 the yield strain times for the collapse prevention level.

3.1. Steel core plate layout

Fig. 5(a) illustrates the plan view of the steel core plate with a yield strength of 303 MPa (44 ksi), as determined from tensile coupon tests. The total length of the core measured 5.69 m (224 in.), including a 3.86 m (152 in.) yield zone. A transition zone, with a 5:1 horizontal to vertical taper, connected the yield zone to the end regions intended to remain elastic. The width of the core was 76.2 mm (3 in.) in the yield zone and 177.8 mm (7 in.) at the ends, with a thickness of 12.7 mm ($\frac{1}{2}$ in.). To prevent buckling at the ends, steel stiffener plates measuring 648 $\times$ 50.8 $\times$ 12.7 mm (25.5 $\times$ 2 $\times$ 0.5 in.) were added at both ends, as shown in Fig. 5(b,c). Additionally, two 9.5 mm (3/8 in.) thick circular steel cheek plates were welded to each end to prevent pin tear-out (Fig. 5(b,c)). A 76 $\times$ 25 mm (3 $\times$ 1 in.) round steel dowel bar was welded to both sides at mid-length of the steel core plate to ensure proper alignment and symmetrical distribution of buckling forces, as shown in Fig. 5.

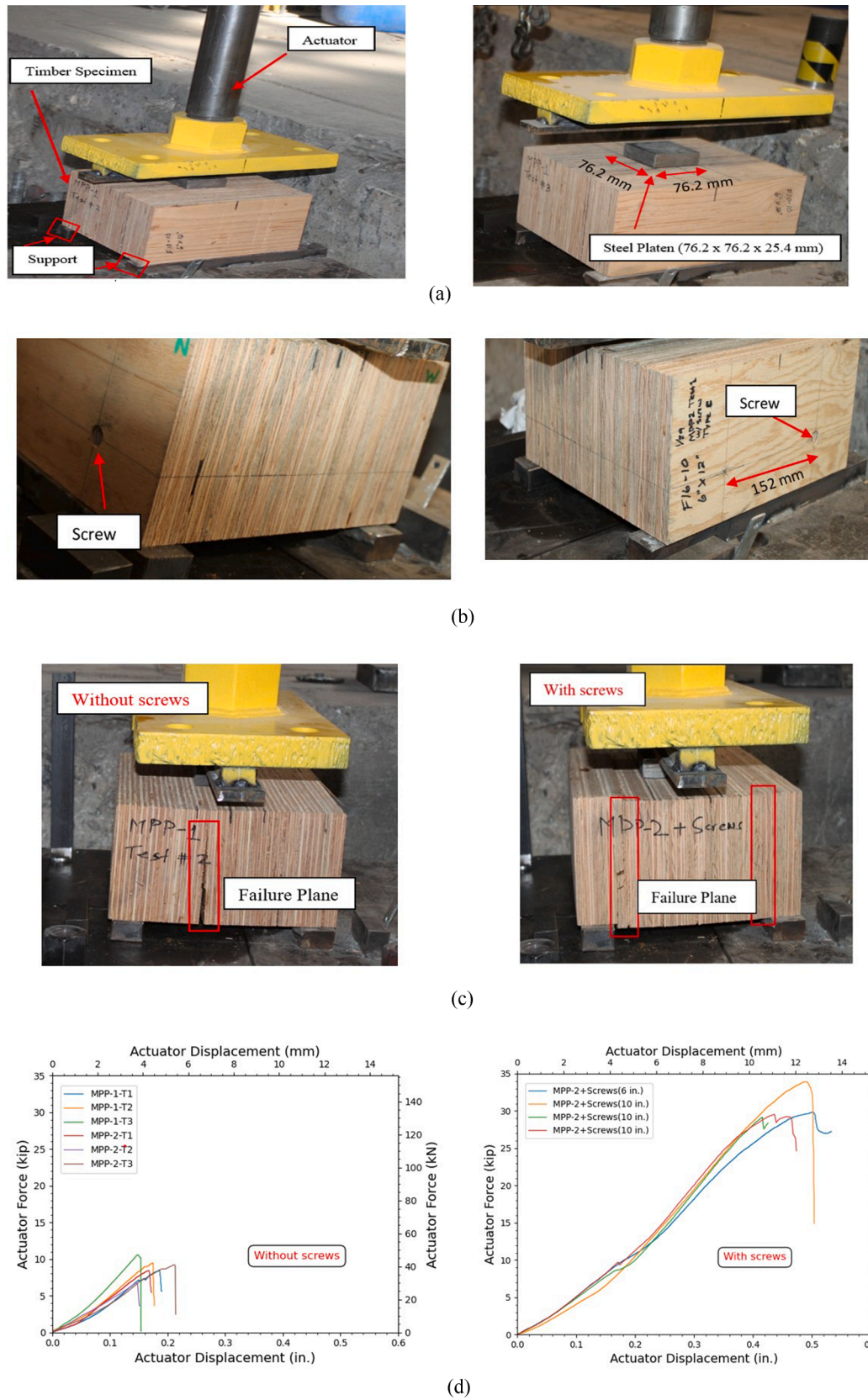


Fig. 2. Small-scale short beam tests: (a) specimen setup; (b) specimen setup for MPP2-F16 with screws; (c) failure modes – without screws (left) and with the screws (right); (d) short beam test results – without screws (left) and with the screws (right).

Table 1

Actuator load and displacement for small-scale short beam tests without (set A) and with (set B) screws.

Specimen Set	Specimen Type	Actuator load [kN (kip)]	Actuator Displacement [mm (in.)]
A	MPP1-F10-Test 1	37.9 (8.5)	4.82 (0.19)
A	MPP1-F10-Test 2	42.5 (9.6)	4.57 (0.18)
A	MPP1-F10-Test 3	46.7 (10.5)	3.81 (0.15)
A	MPP2-F16-Test 1	37.6 (8.5)	4.32 (0.17)
A	MPP2-F16-Test 2	30.7 (6.9)	3.81 (0.15)
A	MPP2-F16-Test 3	41.2 (9.3)	5.59 (0.22)
	Statistical Mean of A	39.6 (8.9)	4.57 (0.18)
B	MPP2 + screws [152.4 mm (6 in.) spacing]	134.5 (30.2)	15.8 (0.62)
B	MPP2 + screws [254 mm (10 in.) spacing]	150.8 (33.9)	12.7 (0.50)
B	MPP2 + screws [254 mm (10 in.) spacing]	130.2 (29.3)	10.9 (0.43)
B	MPP2 + screws [254 mm (10 in.) spacing]	131.8 (29.6)	11.9 (0.47)
	Statistical Mean of B	136.8 (30.7)	12.7 (0.50)

3.2. MPP casing layout

A detailed layout of the MT-BRB is shown in Fig. 5(c,d,e). The two parts of the mass timber casing were made using MPP-2 panels grade F16; each casing was 5080 mm (200 in.) long, 286 mm (11.25

in.) wide, and 159 mm (6.25 in.) thick. A computer numerical control (CNC) machine was used to cut a 6.35 mm (0.25 in.) deep profile into each of the two parts of the timber casing to fit the 12.7 mm (0.5 in.) thick steel core, thus eliminating the need for hardwood spacers, which were used in previous tests [10,14,15]. Zero clearance was provided between the steel core plate and mass timber casing at the top and bottom, whereas a side clearance of 3.8 mm (0.15 in.) was maintained along each edge of the casing. Fastener details are provided in the section on fastener design.

3.3. Mass timber casing design

The mass timber casing prevents global buckling of the brace. The length of the casing was 5.08 m (200 in.). The adjusted brace strength in compression (P_{cr}) is found using Eq. (1). The design yield strength of the steel core was 303 MPa (44 ksi), the cross-sectional area of the steel core plate was 968 mm² (1.5 in.²), and the tensile yield force (P_{ysc}) was 293 kN (66 kip). The adjusted brace strength involves two adjustment factors, the compression adjustment factor (β), and the strain hardening adjustment factor (ω). The β factor is obtained from Eq. (2) where, P_{max} is the maximum compression load and T_{max} is the maximum tension load for each loading cycle. According to AISC 341 [11], this ratio must remain below 1.5 to minimize demand at the connection of the BRB to the braced frame. Similarly, the ω factor defines the ratio of maximum tensile load (T_{max}) per cycle to yield force (P_{ysc})

(Eq. (3)). The yield strength adjustment factor (R_y) accounts for variations in the yield strength of the core and is the ratio of expected to specified yield stress.

$$P_{cr} = \beta \omega P_{ysc} \quad (1)$$

$$\beta = \frac{P_{max}}{T_{max}} \quad (2)$$

$$\omega = \frac{T_{max}}{R_y P_{ysc}} \quad (3)$$

Eq. (1) was used to calculate the adjusted brace strength and this

requires values for both the β and ω adjustment factors. For design purposes, the values of these adjustment factors were adopted from previous tests and taken as $\beta = 1.25$ and $\omega = 1.45$ [10]. The adjusted brace strength was calculated using Eq. (1) as 534 kN (120 kip).

Once the adjusted brace strength was determined the cross-sectional dimensions of the MT-BRB were obtained using Euler's buckling load:

$$P_{cr} = \frac{n\pi^2 EI}{L^2} \quad (4)$$

The end conditions were assumed as pinned so (n) was taken as 1.0. The modulus of elasticity of MPP2 grade F16 was taken as 11031 MPa (1600 ksi) for compression along the joist orientation in accordance with APA Product Report PR-L325 [20]. Utilizing Eq. (4), the minimum moment of inertia required for the MT-BRB casing was obtained as 12, 653 cm⁴ (304 in.⁴). The width of the casing was fixed at 286 mm (11.25 in.), which resulted in the minimum height of the casing as 175 mm (6.87 in.). The final dimensions of the casing were selected as 286 mm (11.25 in.) by 317 mm (12.5 in.). This conservative selection aimed to ensure good performance and prevent global buckling.

After finalizing the casing dimensions, a strength check parallel to lamination was conducted for the MT-BRB in accordance with the National Design Specification for Wood Construction (NDS) [22]. The MT-BRB was analyzed as a mechanically laminated built-up column. The parallel to grain basic compression design value ($F_{c||}$) for MPP-2 was obtained from the APA report as 16.5 MPa (2.4 ksi). This basic value was modified to determine the parallel to grain compression design reference value (F_c^*) using Eq. (5) by incorporating the load duration factor (C_D), wet service factor (C_M), and temperature factor (C_T). Subsequently, the column stability factor was calculated separately for the width and depth directions using Eqs. (6–8). The slenderness ratio in each direction, as determined by Eq. (6), should not exceed 50. The reference compression design value was further adjusted using the column stability factor, (C_p) as shown in Eq. (9), resulting in the final adjusted design strength parallel to grain. The smaller of the two values obtained from the width and depth directions was taken as the limiting strength value of the column parallel to grain. This value was multiplied by the cross-sectional area of the casing to compute the allowable axial force, which should be less than the buckling load.

$$F_c^* = C_D \cdot C_M \cdot C_T \cdot F_{c||} \quad (5)$$

$$\text{slenderness ratio} = \frac{l_e}{d} \quad (6)$$

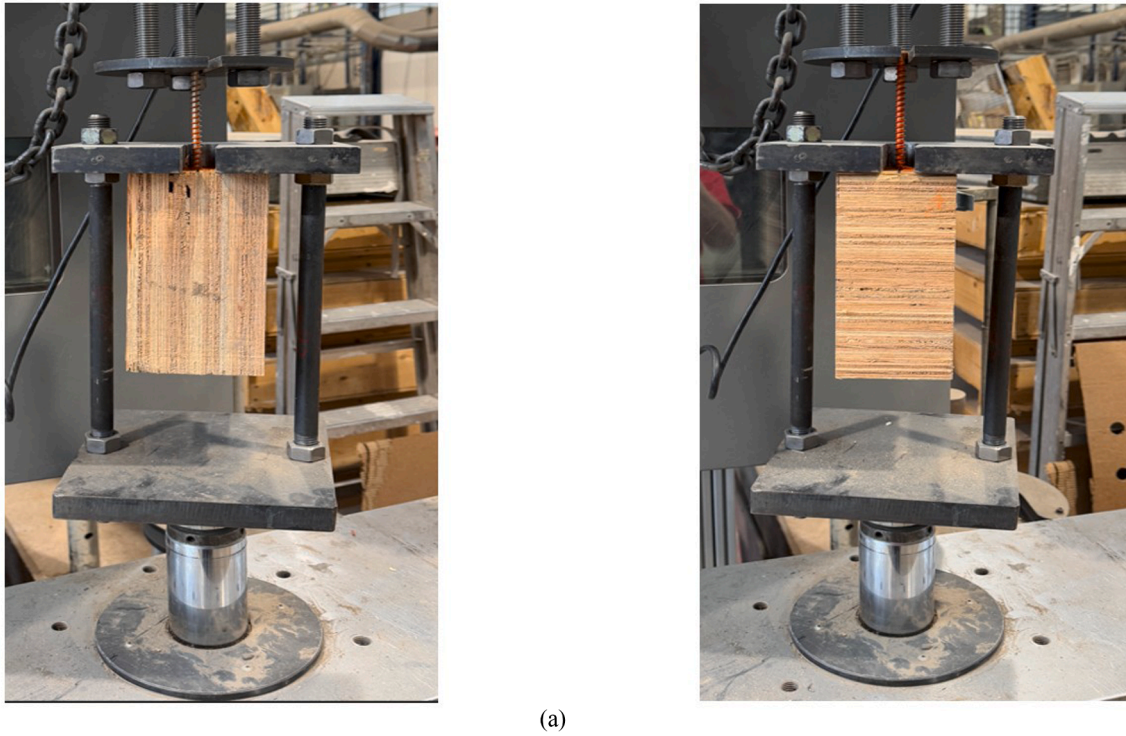
$$F_{cE} = \frac{0.822E'_{min}}{(l_e/d)^2} \quad (7)$$

$$C_p = K_f \left[\frac{1 + \left(\frac{F_{cE}}{F_c^*} \right)}{2c} - \sqrt{\left(\frac{1 + \left(\frac{F_{cE}}{F_c^*} \right)}{2c} \right)^2 - \left(\frac{F_{cE}}{F_c^*} \right)} \right] \quad (8)$$

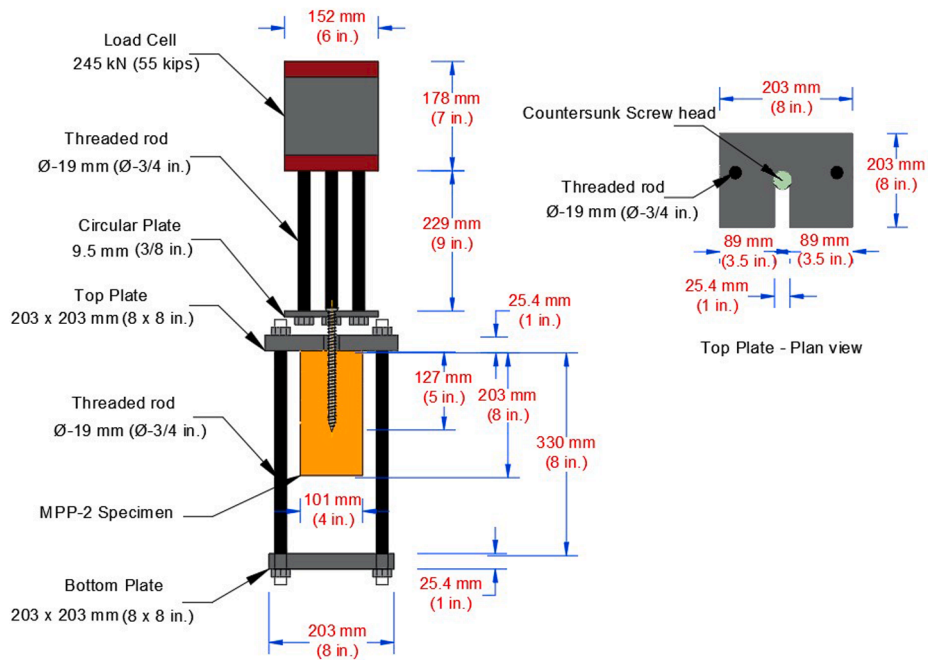
$$F'_{c||} = F_c^* \cdot C_p \quad (9)$$

3.4. Fastener design

In addition to the two components of the mass timber casing, the MT-BRB incorporated bolts and screws, whose design was essential for determining optimal spacing and achieving a cost-effective design. Bolts were employed to connect the two parts of the mass timber casing, while self-tapping screws were used to reinforce the casing to avoid splitting. Two rows of A449 bolts, having a diameter of 12.7 mm (1/2 in.), were used in the middle zone along the yielding length; the yield capacity of the bolt was 81 kN (18.4 kip). At the two elastic zone ends, the bolt diameter was reduced to 9.5 mm (3/8 in.). Fig. 6 shows the buckled



(a)



(b)

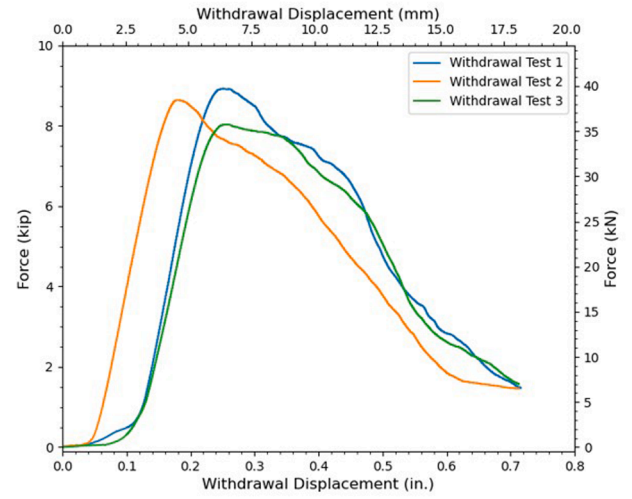
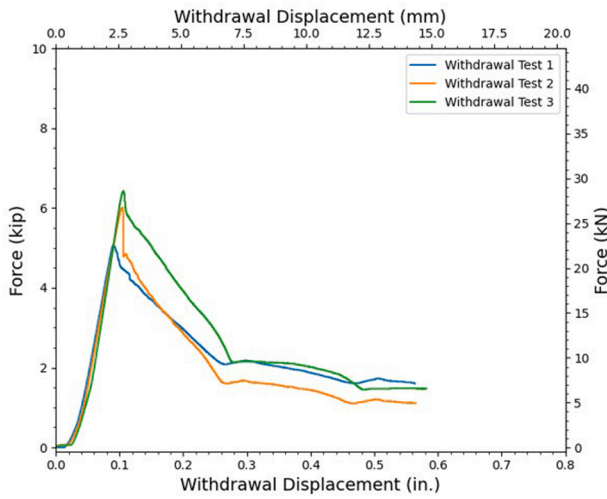
Fig. 3. Withdrawal test setup: (a) screw parallel to lamination (left) and screw perpendicular to lamination (right); (b) schematic drawing.

steel core free-body diagram. Weak axis buckling was considered for the bolt design (Fig. 6(a)). The resulting gap between the core and timber casing along the weak-axis due to buckling of the steel core is given in Eq. (10). The corresponding force per bolt was found using Eq. (11).

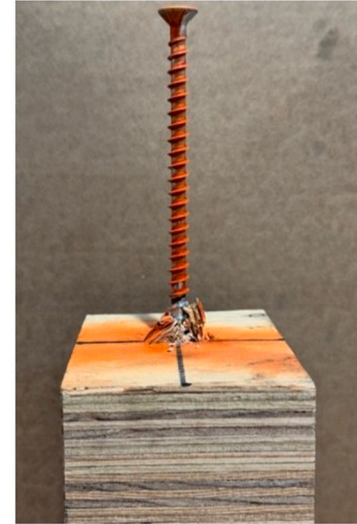
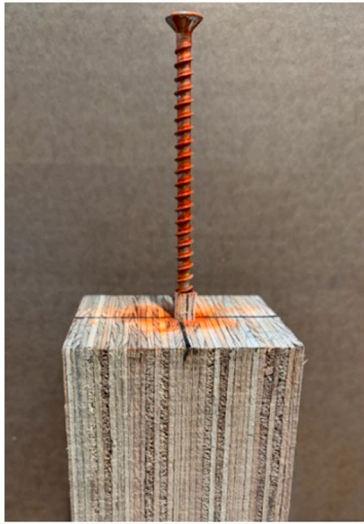
$$\Delta_{\text{gap}} = s_{\text{rw}} + \nu_p \epsilon_{\text{max}} t_f \quad (10)$$

$$P_b = \frac{4P_{\text{max}} \nu_p \epsilon_{\text{max}} t_f}{l_{pw}} \quad (11)$$

where Δ_{gap} = total weak-axis gap during buckling; s_{rw} = initial weak-axis gap which was taken as zero; ν_p = plastic Poisson's ratio taken as 0.5; ϵ_{max} = maximum design strain (3.5 %); t_f = thickness of steel core; l_{pw}



(a)



(b)

Fig. 4. Withdrawal tests: (a) screw parallel to lamination (left) and screw perpendicular to lamination (right); (b) failure – screw parallel to lamination (left) and screw perpendicular to lamination (right).

Table 2
Screw withdrawal test results.

Pullout test results	Parallel to lamination		Perpendicular to lamination	
	Withdrawal force [kN (kip)]	Withdrawal displacement [mm (in.)]	Withdrawal force [kN (kip)]	Withdrawal displacement [mm (in.)]
P – 1, 2	22.64 (5.09)	2.29 (0.09)	39.72 (8.93)	6.35 (0.25)
P – 3, 4	26.73 (6.01)	2.54 (0.10)	38.43 (8.64)	4.57 (0.18)
P – 5, 6	28.56 (6.42)	2.79 (0.11)	35.72 (8.03)	6.35 (0.25)
Statistical Mean	25.98 (5.84)	2.54 (0.10)	37.94 (8.53)	5.84 (0.23)

= weak-axis wavelength taken as $11t_f$ [24] and P_{max} = maximum force considered for bolt

design taken as 534 kN (120 kip) calculated using Eq. (1). Eq. (11) produced a bolt force, P_b , of 3.1 kN (0.7 kip), which was rounded to 4.5 kN (1.0 kip) for design purposes. With a weak-axis wavelength of 140 mm (5.5 in.), this force was applied 55 times along the yield zone length of the core, resulting in a total force of 245 kN (55 kip). Assuming an allowable capacity of 6.7 kN (1.5 kip) per bolt, 38 bolts of 12.7 mm

(1/2 in.) diameter were required in two rows, corresponding to a spacing of 203 mm (8 in.). A final bolt spacing of 178 mm (7 in.) was adopted, resulting in a total of 44 bolts arranged in two rows (Fig. 5(b)). The final bolt allocation is conservative to prevent bolt fracture.

Once the number and spacing of bolts were established, Eqs. (12–17) were used to calculate the outward bulging force [18]. The bulging force was used for designing the screws to prevent splitting of the timber casing. The design of the screws was based on the expected bulging force along the weak axis ($P_{d,w}$) and strong axis ($P_{d,s}$), respectively as calculated using Eqs. (12–17):

$$P_{d,w} = \frac{4N_{cu}(2s_{rw} + v_p t_f \epsilon_{max})}{l_{pw} - \frac{4N_{cu}}{K_r}} \quad (12)$$

$$P_{d,s} = \frac{4N_{cu}(2s_{rs} + v_p w_c \epsilon_{max})}{l_{ps}} \quad (13)$$

$$K_r = \frac{1}{\frac{1}{K_b} + \frac{1}{K_w}} \quad (14)$$

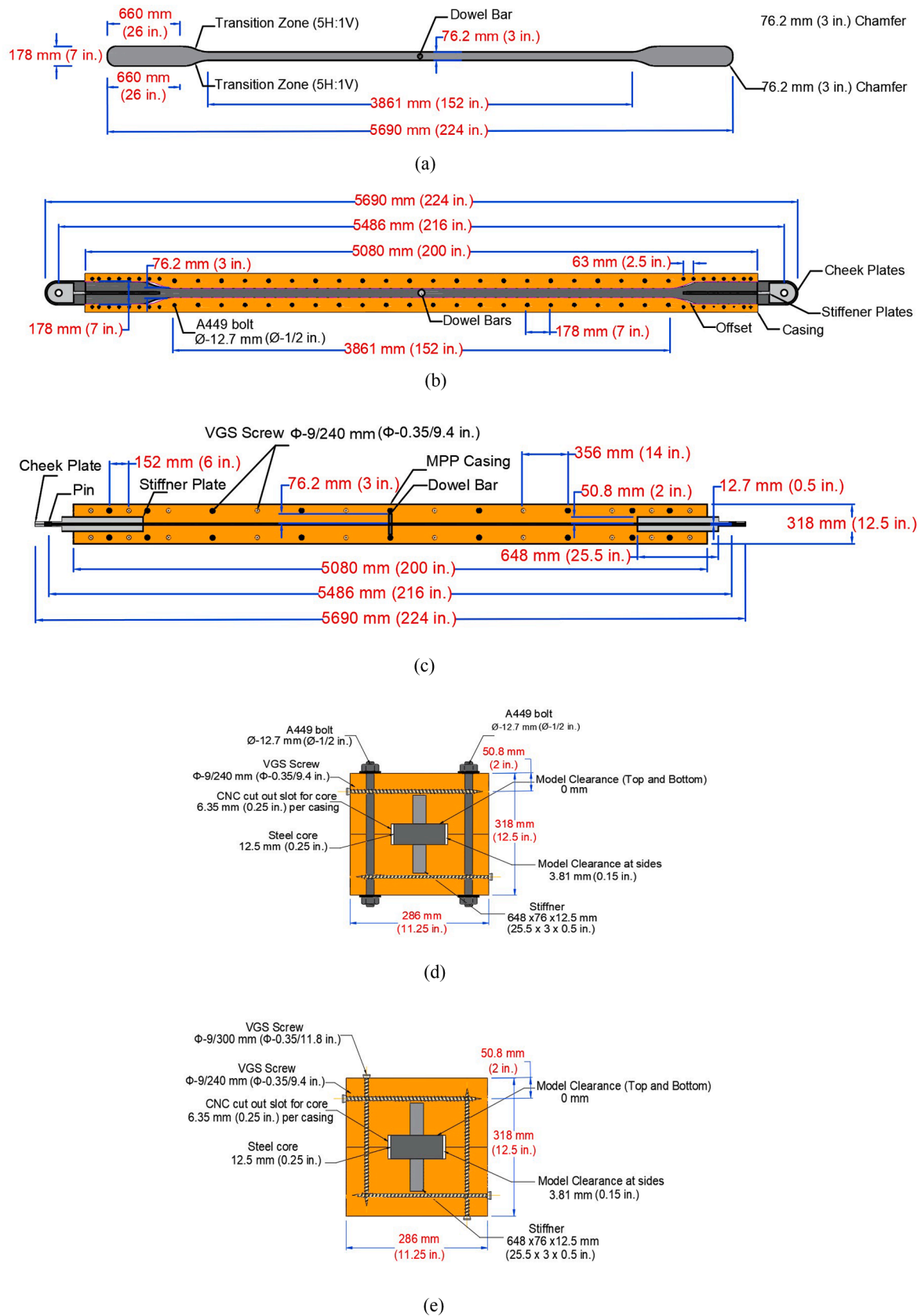


Fig. 5. MT-BRB layout: (a) steel core details; (b) plan view; (c) elevation; (d) cross-section with bolts; (e) cross-section with screws.

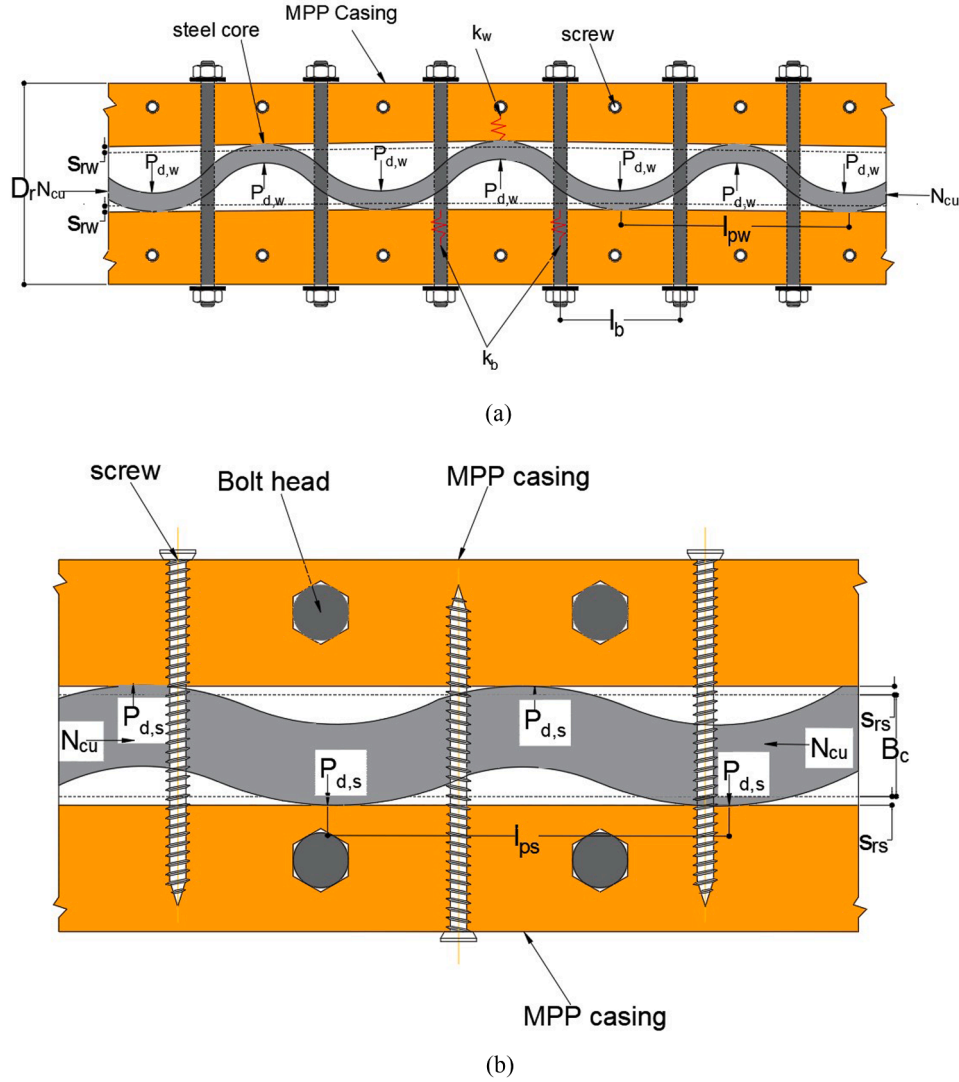


Fig. 6. Mechanics of steel core buckling: (a) weak-axis; and (b) strong-axis.

$$K_b = \frac{E_b A_{\text{eff}}}{D_r / 2} \quad (15)$$

$$K_w = \frac{E_w l_{pw} B_r}{D_{rh}} \quad (16)$$

$$A_{\text{eff}} = 2 \times \frac{\pi d^2 l_{pw}}{4 l_b} \quad (17)$$

where, N_{cu} = maximum core compression force; s_{rs} = strong-axis gap between the steel core and casing; l_{ps} = strong axis wave length, taken as $6w_c$ [24] where w_c = width of steel core; K_r = stiffness of the restraining mechanism given by the combined action of mass timber and bolt; K_b = stiffness of the bolts; K_w = stiffness of mass timber; A_{eff} = effective area of the bolts; E_b and E_w = the elastic moduli of bolts and timber respectively; D_r = depth of the MT-BRB assembly; B_r = width of the casing; D_{rh} = depth of one MPP block and l_b = spacing of the bolts. N_{cu} was taken 534 kN (120 kip) for the design obtained from Eq. (1). Construction tolerance of s_{rs} was specified between 2.54 and 5.08 mm (1/10–1/5 in.); therefore, an average value 3.8 mm (0.15 in.) was adopted for s_{rs} in the design and s_{rw} was taken as zero since no gap was allowed in the weak axis direction (Fig. 5(d)). Eq. (12) resulted in a weak-axis bulging force of 4.1 kN (0.93 kip) per wavelength, which was

rounded to 4.5 kN (1.0 kip). In comparison, Eq. (13) resulted in a strong-axis bulging force of 41.8 kN (9.4 kip) per wavelength, rounded up to 44.5 kN (10.0 kip) for design. In one of the MT-BRBs cyclic experiments where bolts were replaced by screws to connect the two parts of the timber casing, VGS screws Φ -9mm/300 mm were used, as shown in Fig. 5(e).

One of the objectives of this study was to ensure that the timber casing did not suffer premature splitting; thus, horizontal screws were utilized perpendicular to lamination. Figs. 6(a) and 6(b) show free body diagrams of the specimen under weak-axis buckling assembled with bolts, and strong-axis buckling reinforced with screws, respectively. Strong-axis buckling effects were deemed critical and were considered in the design. Since the strong-axis bulging force governed, it was used to determine the number of screws perpendicular to lamination to prevent splitting of the timber casing. This force was applied 18 times along the yielding zone length of the MT-BRB, corresponding to a 457 mm (18 in.) wavelength or $6w_c$ [24], resulting in a total outward force of 800 kN (180 kip). Assuming equal load distribution between the two parts of the timber casing, each part was subjected to a 400 kN (90 kip) force along the strong axis. Using an average screw withdrawal capacity of 38 kN (8.5 kip) from the present tests (Fig. 4(a)), a total of 11 screws were required along the yield zone length, spaced at 356 mm (14 in.). In the elastic zones, identified as critical zones prone to splitting [15],

screw spacing was reduced to 152 mm (6 in.), as shown in Fig. 5(c).

4. Full-scale tests of MT-BRB specimens

An experimental study of full-scale MT-BRBs was performed using seven specimens. Six specimens utilized bolts to secure the two parts of the timber casing and self-tapping screws perpendicular to lamination to prevent splitting. The seventh specimen employed self-tapping screws to connect the two parts of the casing and to prevent splitting; the latter was achieved by installing VGS Φ -9mm/240 mm screws; for connecting the two parts of the casing, VGS Φ -9mm/300 mm screws were used. Fig. 7 shows a MT-BRB specimen in the test frame, the schematic layout

of the test setup, as well as deformation of the prototype braced frame that relates the axial displacement to drift (Fig. 7b).

Two loading protocols were used: a fatigue protocol and a high-strain protocol (Fig. 8a). In the fatigue loading protocol, drift was increased by 0.5 % up to a maximum drift of 3.0 %. After reaching 3.0 %, the drift level was decreased to 2.0 % and maintained until the specimen failed. For the high-strain loading protocol, the displacement was increased in 0.5 % drift increments, until failure occurred. The brace yield deformation (Δ_{by}) and design story drift (Δ_{bm}) were calculated for establishing the loading protocol. The design story drift (Δ_{bm}) was set at 1.0 %, which was calculated as 32.5 mm (1.28 in.). The yield deformation of the BRB was determined using Eqs. (18–20). Since instrument details were

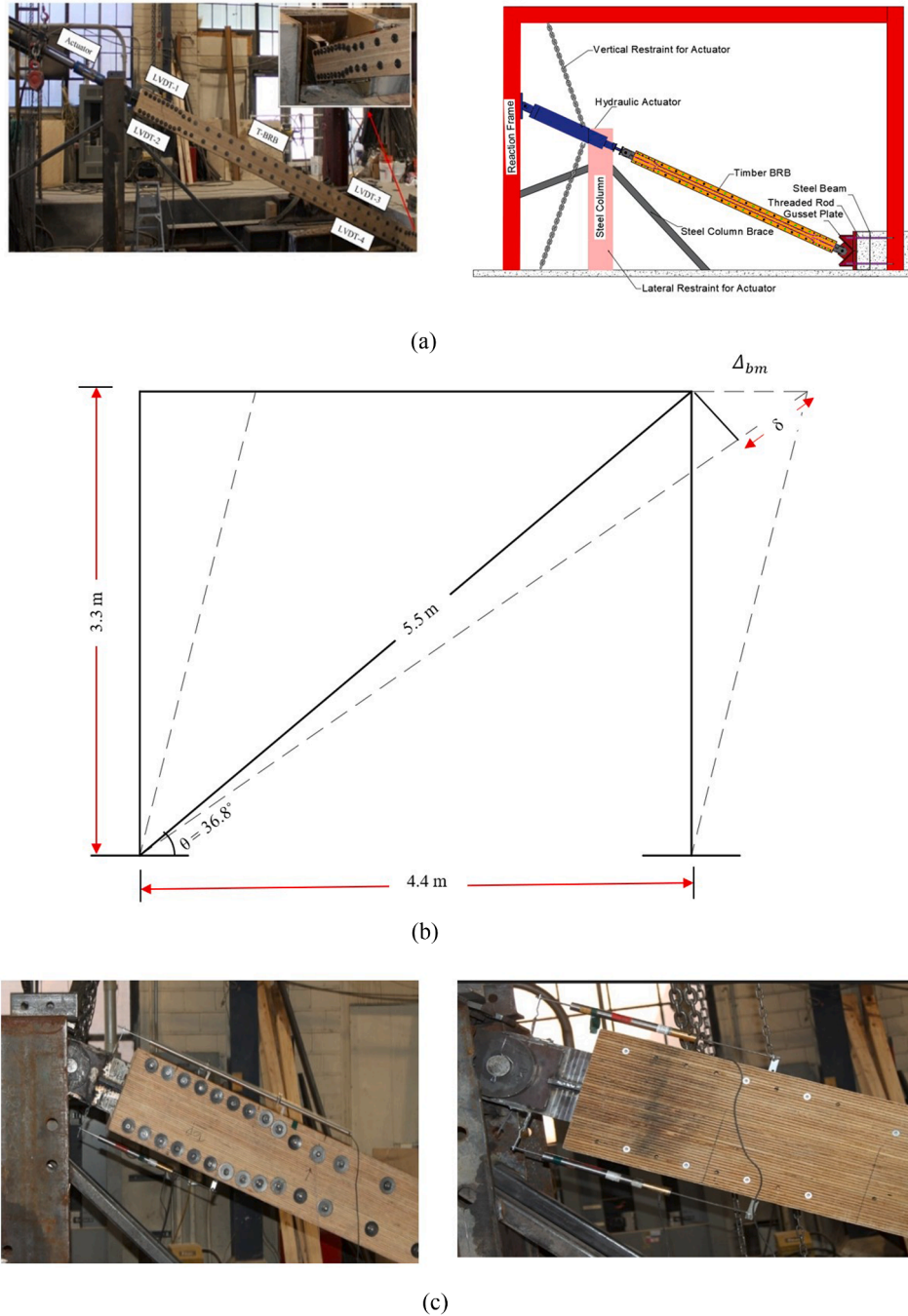


Fig. 7. Experimental cyclic test setup of MT-BRB: (a) picture (left) and schematic drawing (right); (b) deformation of prototype braced frame; (c) MT-BRB test setup for different connectors of two casings – with bolts (left) and with screws (right).

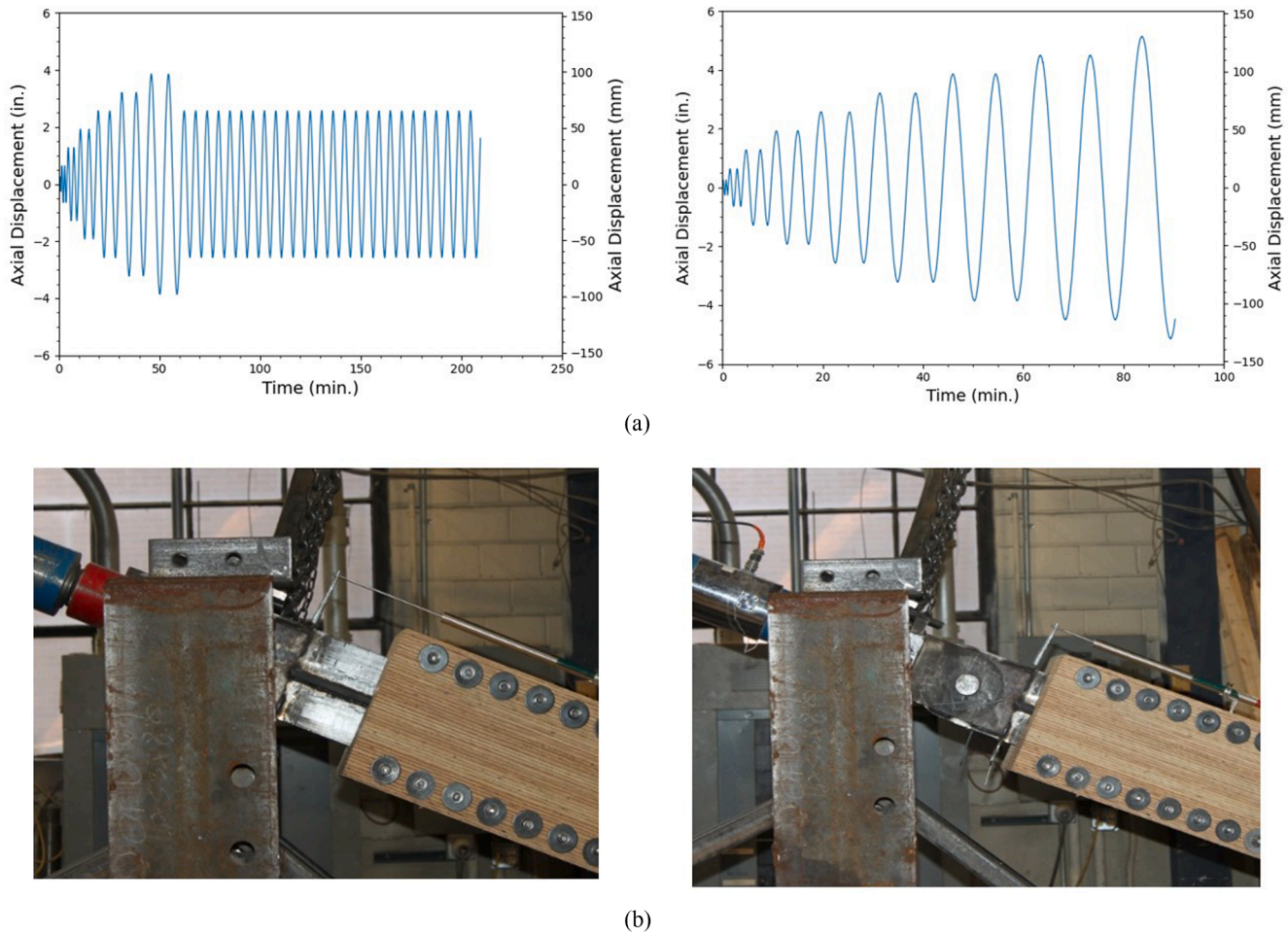


Fig. 8. Loading protocols and steel core displacement of MT-BRB: (a) fatigue protocol (left) and high strain protocol (right); (b) tension (left) and compression (right) of steel core during loading.

accounted for, $\Delta_{apparatus}$ was assumed to be zero. In Eqs. (19, 20), F_{ysc} represents the steel core tensile yield strength, L_{ysc} denotes the yield zone length, L_{end} is the length of the MT-BRB elastic zone; A_{sc} is the area of the steel core, and A_{end} is the area of the steel at the ends.

$$\Delta_{by} = \Delta_{b-ysc} + \Delta_{b-ends} + \Delta_{apparatus} \quad (18)$$

$$\Delta_{b-ysc} = \frac{F_{ysc} L_{ysc}}{E} \quad (19)$$

$$\Delta_{b-ends} = 2 \frac{A_{sc}}{A_{end}} \left(\frac{F_{ysc} L_{end}}{E} \right) \quad (20)$$

For each experiment, the MT-BRB compression adjustment factor (β) and strain hardening adjustment

factor (ω) were determined using Eqs. (2) and Eq. (3), respectively. Since a coupon test was conducted to determine the steel core yield strength, R_y was set equal to 1.0. Once the adjustment factors are established, the adjusted brace strength in both tension and compression can be determined using Eqs. (21) and Eq. (22), respectively.

$$T_{max} = \omega R_y P_{ysc} \quad (21)$$

$$P_{max} = \beta \omega R_y P_{ysc} \quad (22)$$

5. Results of MT-BRB cyclic tests

The following conditions were established as the acceptance criteria

for the MT-BRB tests as outlined in AISC 341-16 [11]: (1) hysteresis curves should be stable throughout the test; (2) a minimum cumulative inelastic demand (CID) of 200 Δ_{by} should be achieved; (3) the compression adjustment factor (β) shall not be greater than 1.5; (4) the strain hardening adjustment factor (ω) should be greater than 1.0. In the present case, the MT-BRB casing should not experience premature splitting; moreover, according to the ASCE/SEI 41-23 standard the ratio of strain at failure to yield strain should be at least 13.3 at the collapse prevention performance level [23]. The CID required for MT-BRB qualification was calculated using Eq. (23) and (24):

$$CID = \sum_{i=1}^n \frac{\Delta_{plastic,i}}{\Delta_{by}} \quad (23)$$

$$\Delta_{plastic,i} = \Delta_{max,i} - \Delta_{by} \quad (24)$$

where the index (i) is an individual half-cycle of tension or compression; $\Delta_{plastic,i}$ is the plastic deformation in each cycle calculated using Eq. (24) and (Δ_{by}) is the brace yield deformation.

5.1. MT-BRB hysteresis and failure modes

A total of seven MT-BRBs were tested. Table 3 shows the MT-BRB specimens and the type and spacing of fasteners used, both parallel and perpendicular to laminations. Table 4 shows that Specimens 1–3 were tested under a fatigue loading protocol, Specimens 4–6 were tested under a high strain loading protocol; Specimen 7 was assembled with screws only and tested under a fatigue loading protocol. The seventh

Table 3
MT-BRB specimens and fastener configurations.

Specimen	Parallel to lamination		Perpendicular to lamination	
	Fastener type	Fastener spacing [mm (in.)]	Fastener type	Fastener spacing [mm (in.)]
1	Bolts	178 (7)	Self-tapping screws	356 (14)
2	Bolts	178 (7)	Self-tapping screws	356 (14)
3	Bolts	178 (7)	Self-tapping screws	178 (7)
4	Bolts	178 (7)	Self-tapping screws	356 (14)
5	Bolts	178 (7)	Self-tapping screws	178 (7)
6	Bolts	178 (7)	Self-tapping screws	178 (7)
7	Self-tapping screws	178 (7)	Self-tapping screws	356 (14)

specimen was an exploratory test to study whether or not, the screws work parallel to laminations – an alternative to bolts. Rationale was to see if it becomes economical to use

only screws. Design philosophy was similar to the bolt design using the capacity of the single screw in withdrawal. A distinct tension and compression displacement behavior of the MT-BRB steel core was observed during each test (Fig. 8(b)). The hysteresis plots for all tested MT-BRBs as well as their associated failure modes are presented in Figs. 9–11. All six MT-BRBs assembled with bolts exhibited stable hysteretic response. The seventh MT-BRB assembled with screws did not achieve a stable hysteretic curve and it failed prematurely.

Specimens 1, 2 and 3 were tested using the fatigue loading protocol. Specimen 1 failed in tension during the 41st loading cycle (Fig. 9(a)). The steel core fractured in tension after high mode strong-axis buckling; minor cracks were observed at various locations along the steel core. No splitting of the timber casing was observed at failure, indicating that the screws used in the perpendicular-to-lamination direction effectively held the casing together. Specimen 2 failed during the 18th cycle in compression due to high-mode buckling of the steel core along its weak axis (Fig. 9(b)). The casing began to split only after reaching 97.5 mm (3.84 in.) of actuator displacement, at which point the cumulative inelastic demand (CID) was $372 \Delta_{by}$. The average wavelength was 116.8 mm (4.6 in.) with a wave amplitude of 38 mm (1.5 in.). Although specimen 1 and 2 had similar configurations of bolts and screws, the failure mode was different. This was because in the case of specimens 1, the steel core experienced strong axis buckling and tensile fracture during the last tension cycle. For specimen 2, the steel core experienced weak axis buckling which caused splitting failure of the casing just outside the elastic region during the last compression cycle. For Specimen 3, failure occurred during the 39th cycle due to tensile fracture of the steel core near the transition zone (Fig. 9(c)). Buckling at failure exhibited a combination of weak-axis and strong-axis waves. The timber casing did not experience any splitting. The average wavelength for weak-axis buckling was 135 mm (5.3 in.) and the wave amplitude at

failure was 38 mm (1.5 in.) as shown in Fig. 9(c) and Table 5; The wavelength for strong-axis buckling was 206 mm (8.1 in.) as shown in Table 5.

Specimens 4, 5 and 6 were tested using the high strain loading protocol. Specimens 4 and 5 failed at the 14th and 15th cycle in compression caused by strong-axis buckling of the steel core (Fig. 10(a) and Fig. 10(b)). Specimen 4 failed at an actuator displacement of 113.7 mm (4.48 in.) at which, a large-amplitude wave, measuring approximately 95.2 mm (3.75 in.), developed along the strong axis

of the steel core; this wave exerted pressure on the casing, ultimately leading to a splitting failure. Specimen 5 failed in a similar fashion as Specimen 4; the only difference was that it completed an extra cycle compared to Specimen 4 with an actuator displacement of 130 mm (5.12 in.). The strong-axis wave amplitude measured at failure was 2.5 in. (63.5 mm), as shown in Fig. 10(b). In both Specimen 4 and 5, the MT-BRB casing experienced splitting failure when the CID had reached $399 \Delta_{by}$ and $461 \Delta_{by}$, thus surpassing the code provision of $200 \Delta_{by}$. This clearly indicated that screws placed perpendicular to lamination remained effective in preventing premature splitting. Specimen 6 initially experienced a strength drop in compression at the 14th cycle due to weak-axis steel core buckling, followed by tensile core fracture during the subsequent cycle (Fig. 9(c)). A small crack on the MT-BRB casing was observed at failure, when the CID was $482 \Delta_{by}$, which indicated that screws perpendicular to lamination postponed early splitting of the timber casing.

Specimen 7 experienced an initial strength drop during the 6th cycle followed by compression failure at the 7th cycle due to high amplitude weak axis buckling of the steel core (Fig. 11). Failure initiated by splitting of the timber casing; the screws were inadequate in securing the casing together. As the loading cycles progressed and displacement increased, the gap between the two parts of the casing widened, enabling the steel core to form increasingly larger buckling waves. Eventually, a

prominent localized wave developed during compression at an actuator displacement of 65 mm (2.56 in.) (Fig. 11). This localized buckling occurred in the transition region of the steel core and was driven by a buckling wave along the weak axis whose amplitude measured 38 mm (1.5 in.). The resulting deformation exerted an outward pressure on the timber casing, leading to a large bulge (Fig. 11)).

5.2. MT-BRB strain, CID and adjustment factors

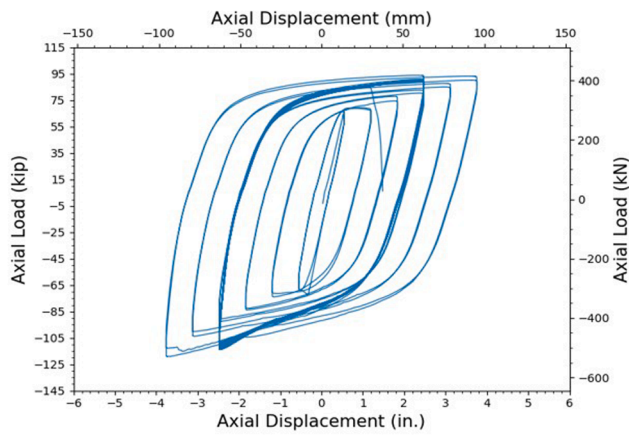
Results from the MT-BRB tests are presented in Table 4. Strain in the MT-BRB steel core was determined from two LVDTs positioned at each of the two ends to measure the axial deformation and

calculate the corresponding strain (Fig. 7(c)). The displacement at one end of the MT-BRB was determined by averaging the readings from the two LVDTs placed at that end. The average displacements from both ends were then summed and divided by the yield zone length of the steel core to calculate the axial strain. Fig. 12 shows peak strain at failure normalize by the yield strain for all tests. The MT-BRBs achieved a maximum strain equal to 3.1 % in the case of Specimen 5 under the high strain loading protocol. In all tests of MT-BRB with bolts the failure strain exceeded the collapse prevention performance level of 13.3 times

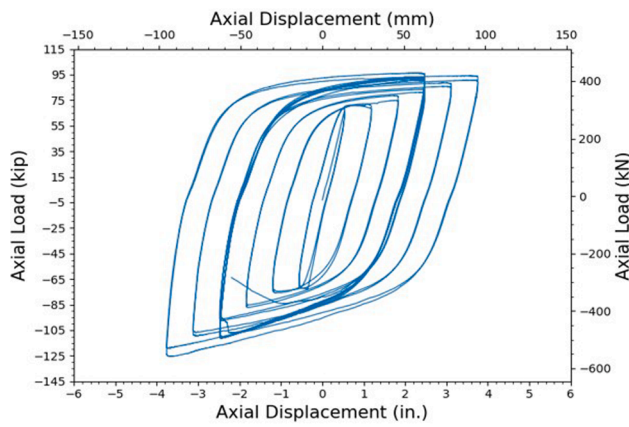
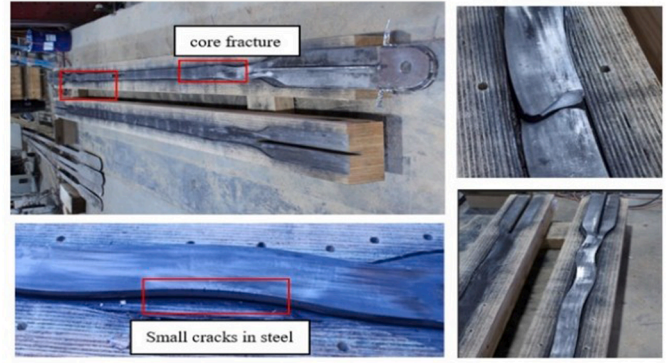
Table 4
MT-BRB specimens and test results.

Protocol	Specimen	Max strain (%)	Cycles to failure	β	ω	CID	Total hysteretic energy dissipation [kN-m (Kip-in.)]
Fatigue	1	2.4	41.0	1.27	1.42	1380	3161 (27976)
	2	2.4	18.5	1.30	1.46	588	1303 (11530)
	3	2.4	39.0	1.30	1.48	1238	3056 (27053)
High strain	4	2.7	13.5	1.32	1.47	399	1108 (9810)
	5	3.1	14.5	1.33	1.52	461	1353 (11978)
	6	2.7	14.0	1.31	1.52	482	1150 (10177)
Fatigue	7	1.5	6.5	1.44	1.24	151	229 (2026)

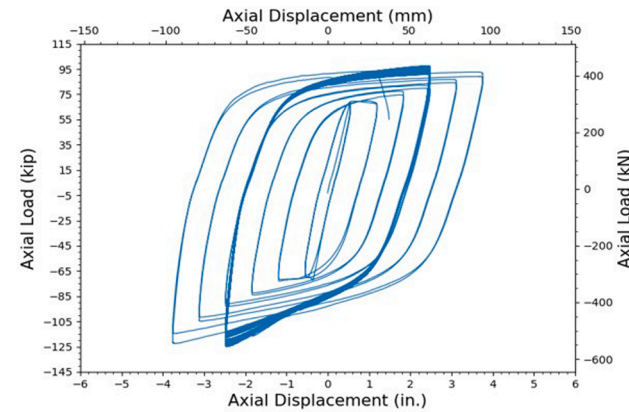
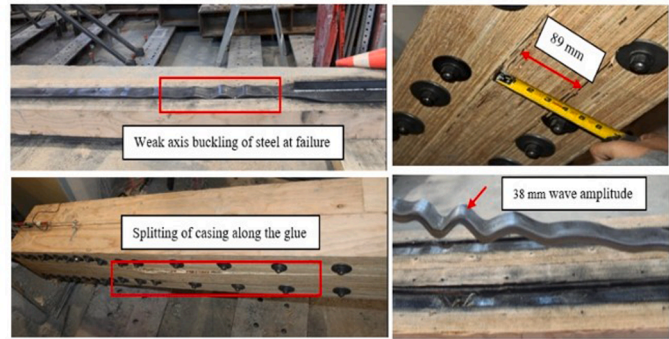
Note: β = compression adjustment factor; ω = strain hardening adjustment factor.



(a)



(b)



(c)

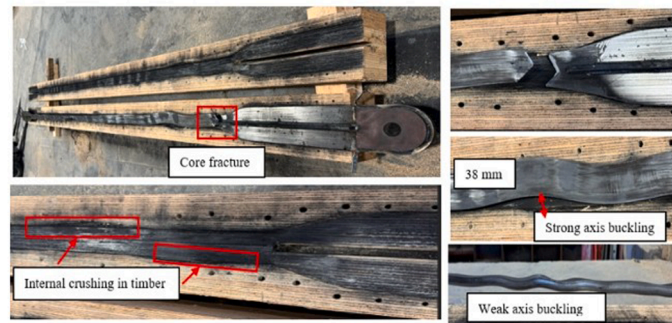
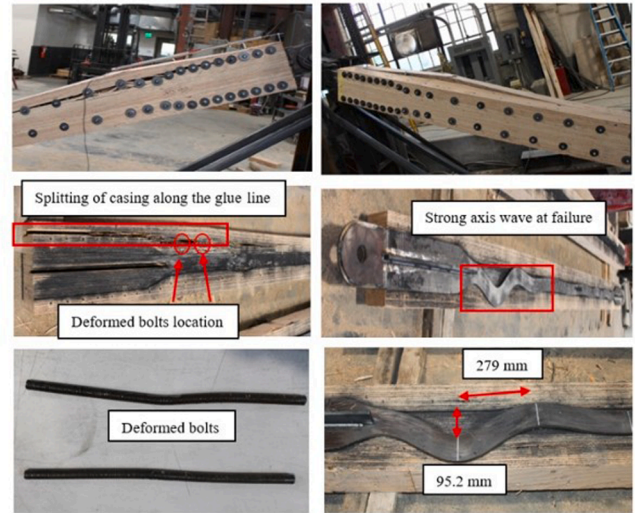
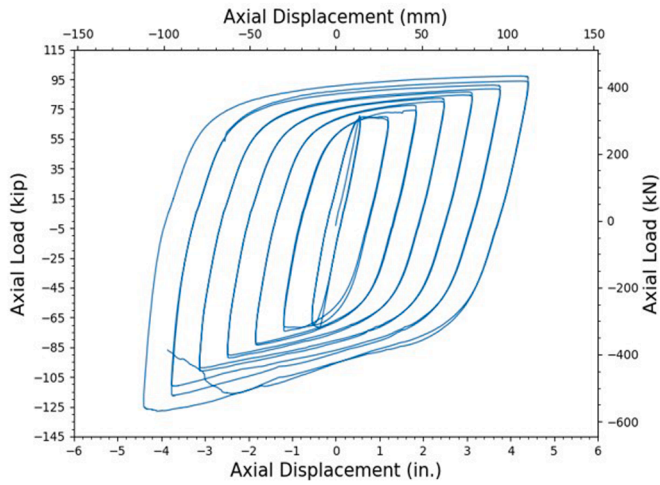


Fig. 9. Hysteresis curves for fatigue loading protocol with associated failure mode: (a) Specimen 1; (b) Specimen 2; (c) Specimen 3.

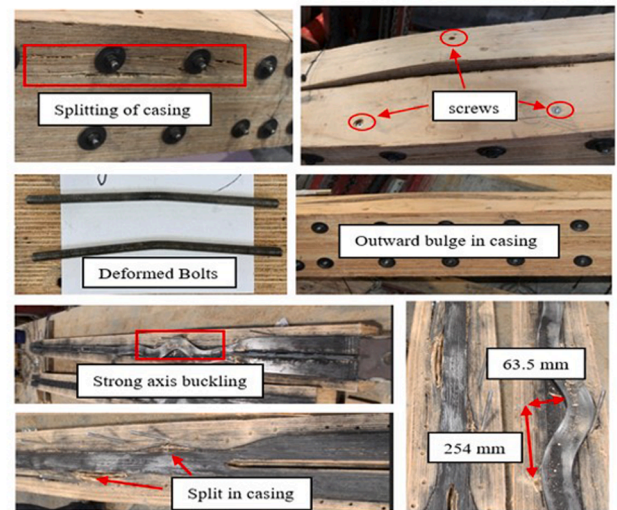
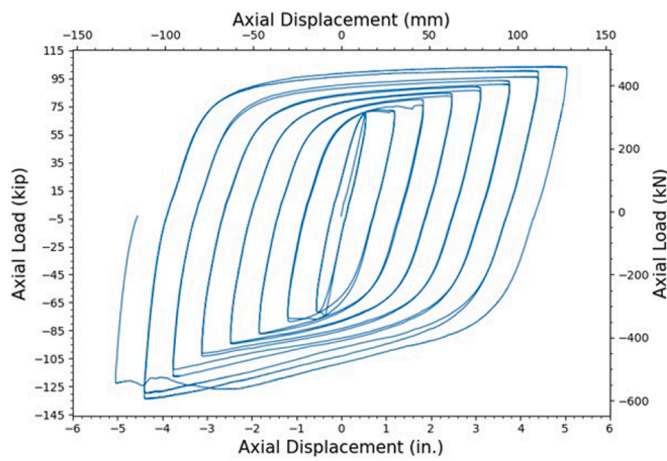
the yield strain, as required by ASCE/SEI 41–23 [23]; Specimen 5 recorded the highest value at 20.37 times the yield strain. Specimen 7, which used screws to assemble the timber casing, achieved a failure strain of only 9.82 times the yield strain. For

each experiment, the compression adjustment factor (β) and strain hardening adjustment factor (ω) were determined using Eq. (2) and Eq. (3), as shown in Table 4. For Specimens 1–3 with bolts tested under the fatigue loading protocol, the maximum value of β ranged from 1.27 to

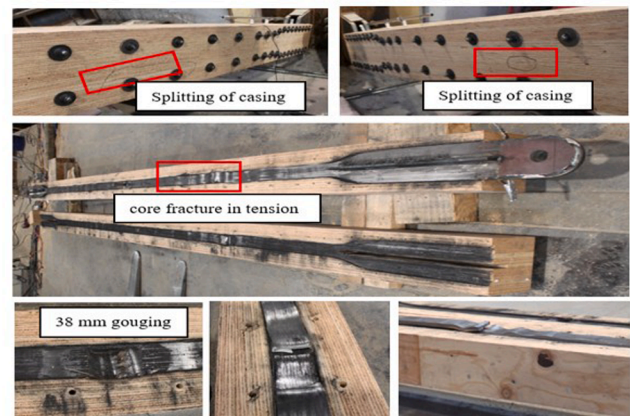
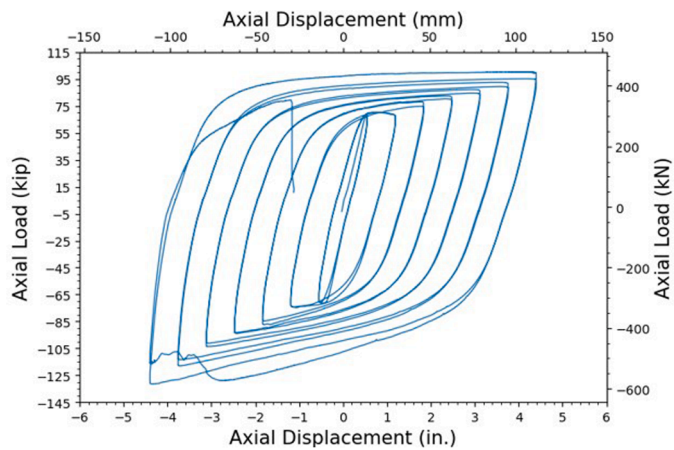
1.30 and ω from 1.42 to 1.48. For Specimens 4–6 with bolts tested under the high strain loading protocol, the maximum value of β ranged from 1.31 to 1.33 and for ω from 1.47 to 1.52. For Specimen 7 with screws tested under the fatigue loading protocol, the maximum value of β was 1.44 and ω was 1.24. For all successful tests (Specimen 1–6), the β factor was well within the limits of $1.0 < \beta < 1.5$ allowed by AISC 341 [11]. The assumed design values of β and ω were encompassed by the experiments, confirming the adequacy of the MT-BRB design. The six



(a)



(b)



(c)

Fig. 10. Hysteresis curves for high strain loading protocol with associated failure mode: (a) Specimen 4; (b) Specimen 5; (c) Specimen 6.

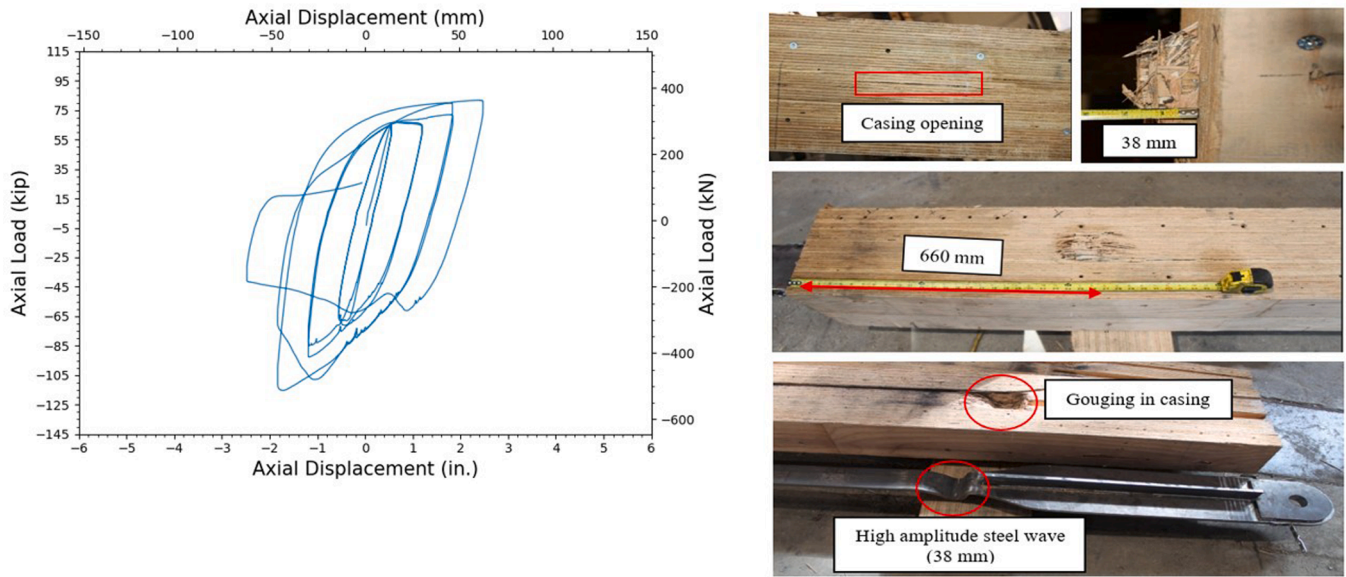


Fig. 11. Specimen 7 hysteresis curves for fatigue loading protocol with associated failure mode.

Table 5
Average high-mode buckling wavelength.

Specimen	Average wavelength		Failure mechanism
	Weak axis [mm (in.)]	Strong axis [mm (in.)]	
1	172.7 (6.8)	13.6	215.9 (8.5)
2	122.7 (4.83)	9.6 t_c	-
3	134.62 (5.3)	10.6	205.74 (8.1)
4	148.34 (5.84)	11.7	314.45 (12.38)
5	133.35 (5.25)	10.5	279.4 (11)
6	173.23 (6.82)	13.6	205.74 (8.1)
7	114.30 (4.5)	9.0 t_c	-

specimens with bolts exhibited stable hysteretic behavior with the MT-BRBs achieving a maximum strain of 3.1 %, which corresponds to an inter-story drift of 4.9 % for the prototype braced frame shown in Fig. 7 (b).

The cumulative inelastic deformation (CID) was found for each specimen using Eq. (23). The achieved CID exceeded the code required value of 200 Δ_{by} for Specimens 1–6, whose casing was assembled with bolts, as shown in Table 4. However, the CID requirement was not satisfied for Specimen 7 whose timber casing was assembled with screws. For Specimens 1–3 tested under the fatigue loading protocol, the CID value achieved ranged from 2.9 t to 6.9 times the required value of the AISC 341 standard [11]. For Specimens 4–6 tested under the high strain protocol, the CID ranged from 2.0 to 2.3 times the AISC 341 requirement.

Screws perpendicular to lamination are crucial for preventing premature splitting, as evidenced from these tests. While Specimen 2 met all the initial qualifying criteria, its life (18.5 cycles) was still shorter than Specimen 1 (41 cycles). To investigate this further, screw spacing was varied for certain specimens. Reducing the screw spacing from 356 mm (14 in.) to 178 mm (7 in.) for Specimens 3, 5, and 6 produced mixed results. Specimen 5 showed a gain of one cycle during high strain loading (14.5 cycles) compared to Specimen 4 (13.5 cycles), but

Specimen 6 did not change significantly (14.0 cycles). Ultimately, Specimen 6 with 178 mm (7 in.) spacing performed similarly to Specimen 4 with 356 mm (14 in.) spacing, and Specimen 1 (41 cycles) with 356 mm (14 in.) spacing performed similarly to Specimen 3 (39 cycles) with 178 mm (7 in.) spacing. This suggests that 356 mm (14 in.) is a reasonable screw spacing, and any further reduction in screw spacing does not significantly improve performance.

5.3. Bolt strain history

Bolt strain was recorded for selected tests to estimate the outward force exerted on the timber casing due to buckling of the steel core. Fig. 13 shows the bolt strain history for two bolts of Specimen 6; one of the bolts was located in the transition zone near the actuator end (Bolt 1) and the other was located at mid-length of the specimen (Bolt 2). There was a sharp increase in bolt strain while tightening the bolts prior to testing. Bolt strain remained relatively constant during the test with some decrease due to slack from permanent deformation of the timber casing caused by embedment of the washer. The tensile yield strain of the bolt was 3200 microstrain; the highest strain recorded was 2550 microstrain. During the test, bolt strain remained relatively constant at 2000 microstrain. This demonstrates that the primary difference between bolts and screws is the pre-stressing effect created by the bolts, washers and nuts, which hold the two timber parts of the casing together and restrain the

steel core from buckling; by contrast, screws are not effective until much later when the steel core begins to exert pressure on the timber casing and begins to buckle, which results in early failure of the MT-BRB.

5.4. Weak axis and strong axis wavelength

Weak axis (l_{pw}) and strong axis (l_{ps}) wavelengths were measured after each test. Table 5 presents the

average wavelengths for each specimen. For the majority of cases, specimens failed due to strong-axis

buckling. The average weak-axis wavelength obtained was 10–11 times the thickness of the steel core, t_f ; this value is consistent with the buckling wavelength of conventional BRBs, equal to $11t_f$ [24]. In the case of strong-axis buckling, the wavelengths were difficult to measure since the strong-axis buckling of the steel core was a localized phenomenon. The observed wavelength, l_{pw} , varied between $2.7w_c$ to

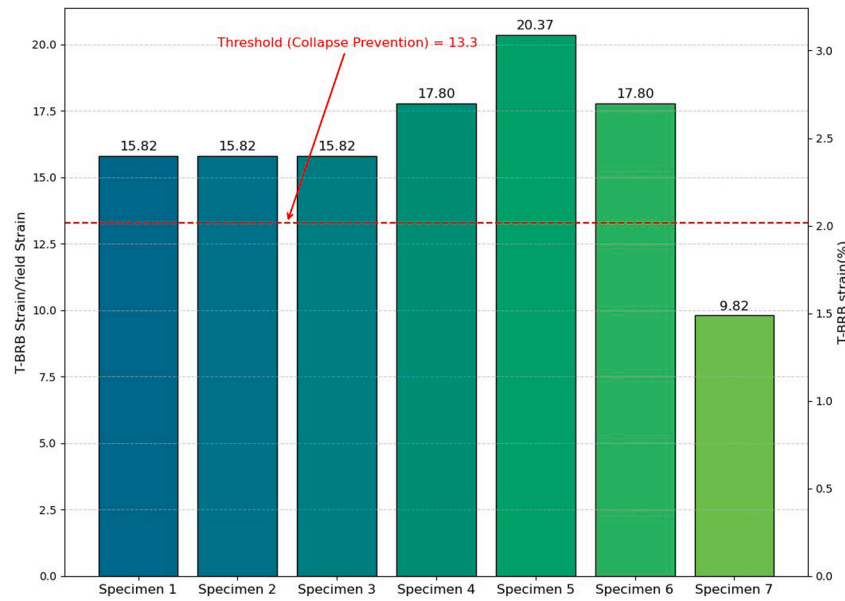


Fig. 12. Peak strain achieved in the MT-BRB tests.

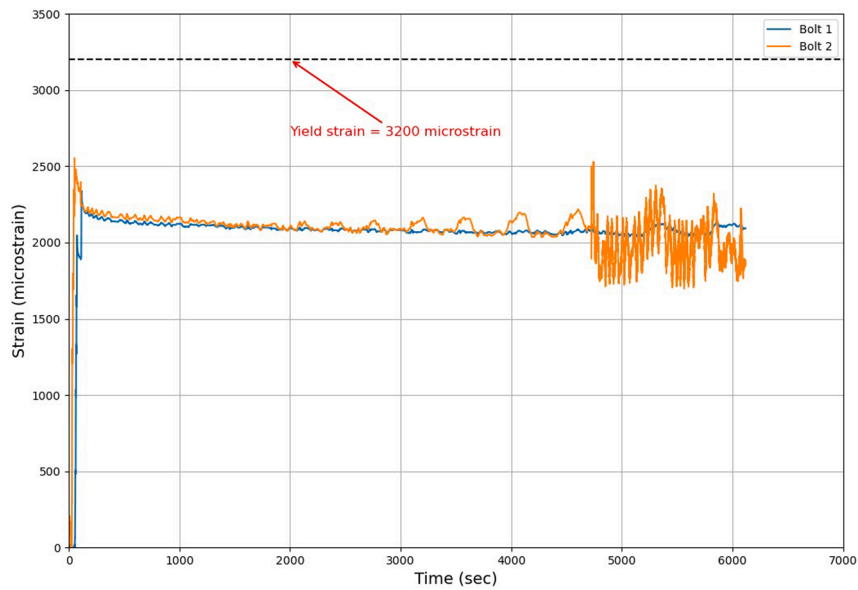


Fig. 13. Specimen 6 strain history for two bolts.

$4.5w_c$, where w_c is the width of the steel core.

6. Conclusions

A series of quasi-static cyclic tests were carried out to evaluate the performance of full-scale MT-BRBs in accordance with AISC 341–16 and ASCE/SEI 41–23 standards. Seven full-scale tests were conducted on 5.5 m (18-ft) long MT-BRB pin connected specimens; six specimens utilized bolts to connect the two parts of the mass timber casing and screws to prevent splitting of the mass plywood panels (MPP); the seventh specimen incorporated screws for both purposes. Small-scale beam tests were performed on MPP blocks with and without screws to assess timber casing resistance against premature splitting. Screw pullout tests were also conducted to measure withdrawal strength of screws oriented parallel or perpendicular to lamination. Based on the findings the following conclusions are drawn:

- Small-scale beam tests on timber blocks demonstrated that fully threaded self-tapping screws inserted perpendicular to lamination significantly enhanced resistance to splitting of the timber laminations. The compressive force capacity for the blocks with screws was 3.2–3.4 times that of the blocks without screws; the compressive displacement for the blocks with screws was 2.8 times that of the blocks without screws.
- Withdrawal test results from MPP blocks show that the withdrawal force parallel to lamination was 0.68 times the withdrawal force perpendicular to lamination. The peak withdrawal force was adequate in determining the spacing of the screws placed perpendicular to lamination for the design of the MT-BRB.
- The current research has developed a full-scale 5.5 m (18 ft) long MT-BRB suitable for industry application. This research has eliminated the use of separate hardwood spacers in between the components of a casing used in earlier studies. Screws perpendicular to

lamination effectively postponed premature splitting of the timber casing by enhancing its strength perpendicular to lamination; the screws served as a cost-effective alternative and eliminated the need for carbon fiber-reinforced polymer (CFRP) wraps or any internal and external steel plates.

- The full-scale MT-BRBs assembled with bolts exhibited stable hysteresis curves with excellent hysteretic energy dissipation. At failure, the CID achieved for the bolted MT-BRBs under the fatigue loading protocol ranged from 2.9 to 6.9 times the requirement of 200 specified in the AISC 341–16 standard. The compression adjustment factor ranged from 1.04 to 1.33 which is less than 1.5, the maximum allowed by AISC 341–16.
- The steel cores of the MT-BRBs with bolts tested under the high strain protocol achieved a maximum strain 20.4 times the yield strain. All bolted MT-BRBs satisfied ASCE/SEI 41–23 strain criteria for the collapse prevention performance level. The average weak-axis buckling wavelength observed in the tests was eleven times the core thickness, which closely matched the weak-axis buckling wavelength observed in conventional composite steel-concrete BRBs. In the case of strong-axis buckling, the observed wavelength varied between 2.7 and 4.5 times the steel core width. The results show that the bolted MT-BRBs designed in this paper performed very well and could be used in actual buildings for improving the seismic performance of mass timber structures.
- One MT-BRB whose casing was assembled with screws experienced premature failure; this indicates that screws are less effective than bolts in connecting the two parts of the timber casing. Bolts provide a better restraining effect compared to screws due to the pre-stressing force generated during tightening, aided by the use of nuts and washers.

Limitations and further studies

For further studies, a full-scale sub-assembly test should be performed using 5.5 m (18 ft) long MT-BRBs. The steel core cross-section used in this research was limited to 968 mm² (1.5 in.²); however, larger cross-sections would be necessary for MT-BRBs to be used in multi-story buildings. Therefore, further testing of 5.5 m (18 ft) long MT-BRBs with larger cross-sections is recommended. Additional tests should be carried out for MT-BRBs built with MPP and screws parallel to lamination adopting different screw types such as partially threaded screws.

CRediT authorship contribution statement

Hans-Erick Blomgren: Writing – review & editing, Investigation, Formal analysis, Conceptualization. **Douglas Rammer:** Writing – review & editing, Methodology, Formal analysis, Conceptualization. **Pantelides Chris:** Writing – review & editing, Supervision, Resources, Project administration, Methodology, Investigation, Funding acquisition, Conceptualization. **Rupesh Upreti:** Writing – original draft, Visualization, Validation, Methodology, Investigation, Formal analysis, Data curation.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Chris P. Pantelides reports financial support was provided by US Department of Agriculture. If there are other authors, they declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data Availability

Data will be made available on request.

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