Blast resistance of timber structural elements: A state of the art review

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Abstract
The response of structures subject to impulsive loads remains a field of intense research. While traditional construction materials, such as steel and concrete/masonry, have been the focus of most studies, further research on the performance of alternative materials for blast resistant applications has been driven by their growing use in sustainable construction. Over the last years, engineers have been re-evaluating the use of timber as a prime construction material for a range of building types, from small office to high-rise residential buildings. As a result, there is now a growing need to study the blast resistance of timber structures, as they may become potential targets of terrorist attacks or being placed in the blast-radius of other critical buildings. A review of existing research on the blast resistance of timber structures is presented and key factors on the blast analysis and design of such structures are discussed. Most of the research has been conducted on light-frame wood stud walls, glued- and cross-laminated timber, and addresses material properties under high strain rates, typical failure modes, behaviour of structural connections and retrofitting solutions. Failure modes are reported to be highly dependent on the element layout and manufacturing aspects, and dynamic increase factors for the modulus of elasticity and maximum strength in the ranges of $[1.05, 1.43]$ and $[1.14, 1.60]$, respectively, have been proposed for different timber elements. Mechanical connectors play a significant role in dissipating energy through plastic deformation, as the brittle nature of timber elements compromises the development of their full capacity. Regardless the element type, SDOF models can accurately predict the dynamic response as long as idealised boundary conditions can be considered. Overall, although a good amount of research is available, more extensive research is needed to guide the design and engineering practice and contribute to the development of design codes and testing standards for timber structures under blast loading.

Keywords
Blast loading, Light-frame wood, CLT, Glulam, Retrofitting, Connections, Modelling
Introduction

Timber has several unique characteristics that make it a versatile yet challenging construction material. It has a remarkable strength-to-weight ratio and is economically and environmentally sustainable when sourced from well-managed forests (1). Due to its highly machinable nature, it can be fabricated into a wide variety of shapes and sizes to fit practically any construction need, which makes it an excellent candidate for use in a diverse range of construction projects (2). However, because it is naturally grown, timber is a multi-scale material with low homogeneity. Its mechanical properties are affected by a number of parameters, including the tree species, the various growth conditions, the soil type, and availability of water and nutrients (3). It is a strongly anisotropic material, with its dimensional and mechanical properties being highly variable with time, temperature, and stress state (4). This translates into different tensile, compressive and shear properties (5). Knot-like defects and other imperfections in the grain further reduce apparent mechanical performance of timber products (6).

Solid-sawn timber sections cut from a single log are the most common form of wood construction used in light-frame construction of residential and low-rise buildings. Recent manufacturing advancements have led to the development of engineered wood products (EWPs) by adhesively laminating smaller pieces of wood, with application of heat and pressure, to form a structurally efficient composite member (7). The use of smaller individual pieces of wood increases the economic and environmental sustainability of EWPs while providing greater reliability, strength, stiffness and dimensional stability that can be achieved with solid-sawn timber (8). Examples of EWPs include parallel-strand timber (PSL), laminated veneer timber (LVL) and oriented strand board (OSB), as well as relatively new products such as glued-laminated timber (glulam) and cross-laminated timber (CLT). Timber construction is now a viable alternative to

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conventional steel and concrete construction for mid- and high-rise structures ranging from 5 to 20 stories (2; 9). As a result, architects and engineers around the world have been re-evaluating the use of timber as the prime construction material for an expanding range of building types, including government, public, and private buildings, airport and border facilities, as well as transportation facilities and entertainment venues (10; 11; 12). Figure 1 depicts both conventional light-frame and mass timber construction.

Figure 1. Timber building construction methods: (a) light-frame construction and (b) mass timber construction. Reprinted from Holden et al. (11) under the Creative Commons license 4.0.

The increasing number of high-profile buildings constructed using timber increases the likelihood that this type of construction may in the future be the target of hostile terrorist attacks or be vulnerable to accidental explosions. This risk is further compounded by increasing urbanisation, wherein timber structures and their occupants may be placed at an unacceptable level of risk by simply being located within the blast-radius of other buildings (13). Blast events occurring near timber buildings may damage structural and non-structural wood members due to significant dynamic loads, ultimately causing the structure to experience hazardously high deformations, possibly leading to collapse (14).

Research on the effect of blast on structures has mostly focused on reinforced concrete (15; 16; 17; 18; 19) and structural steel (20; 21; 22; 23; 24; 25) due to their widespread use in construction. To date, little research has examined the behaviour of light-frame and mass timber structures under blast loading. The lack of consensus and critical knowledge gaps on specific design recommendations for blast-resistant timber structures is problematic and creates uncertainty for engineers and manufacturers about what should be best design practice.
This work aims to provide a state-of-the-art review of the behaviour of timber construction subjected to blast loading and to identify knowledge gaps requiring further experimental and computational research. A survey of experimental techniques used to generate blast loads on timber components is performed to investigate differences in test approaches on performance characterisation.

The blast response of dimensional timber and EWP s to blast loads, including CLT, glulam, and structural insulated panel (SIP) components, is discussed to understand how their unique material, structural, and system characteristics affect damage levels and levels of protection. The role of connections on the global behaviour of wood elements and their influence on the load path is assessed. Additionally, various retrofitting techniques are analysed to understand how their implementation can enhance overall protection levels. Finally, new knowledge gained from these studies is synthesised to better understand how blast effects should be considered in the design process of buildings fabricated from wood materials.

Summary of available literature

A summary of the available literature is presented in Table 1, with reference to the structural element tested, method of blast loading generation and research focus. The peak blast pressure and impulse combination reported in each paper was converted into an equivalent scaled distance $Z$ using the Kinery-Bulmash (26) blast parameter curves assuming a fully reflected shock. Relevant trends can be observed that help shape this state-of-the-art review: (1) most tests were performed at the University of Ottawa using a pneumatically-driven shock tube to generate blast loads; (2) most specimens were subjected to uniform, far-field blast loads with $Z > 5$ m/kg$^{1/3}$ generating global flexural response; (3) most studies include a single-degree-of-freedom (SDOF) analysis component and high fidelity finite element analyses are rarely used; (4) most studies focused on the response of timber structures at the element level, thereby not considering the potential beneficial effects of system behaviour, load sharing and redistribution present in structural assemblies; (5) most research focus on different types of limit states, dynamic material properties, component failure modes, connections, and strengthening of existing structures.
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ST: Shock tube; FT: Field test

**Experimental techniques used to study response of wood structures to blast loads**

Explosive field tests and shock tube testing are the two most common methods used to study the blast performance of timber structures (see Table 1). Both techniques aim to simulate real-world blast scenarios as realistically as possible,
although both have advantages and limitations in terms of the size and scale of test specimens, and magnitude of the blast loads that can be achieved.

One of the typical methods for conducting blast testing is to generate the loading in a test field through the detonation of an explosive charge (51; 57; 58). The advantage of field tests is that the performance of timber structures can be characterised at large or full-scale under actual explosive loads that generate damage patterns and failure modes at realistic scales. Field tests are typically conducted at military test sites or large open spaces located in remote areas where free blast pressure clearing can be achieved under safe conditions, as seen in Figure 2(a). Live explosive test preparation is costly and time consuming as specialised personnel is required for explosive handling, the structures to be tested are comparatively large requiring specialised manufacture, and a wide variety of instrumentation must be placed and calibrated prior to each test. Only a limited number of tests on timber structures have been performed using live explosive tests, as listed in Table 1.

Shock tubes are the most widely reported experimental technique for generating blast loads on timber structures. Pneumatically driven (13) and gas-detonation (59; 60) shock tubes have been used for this purpose, as shown in Figures 2(b) and 2(c), respectively. The advantage of shock tube testing is that relatively large-scale experiments can be performed in a controlled laboratory environment for the exploration of the fundamental behaviour of structures and validation of analytical models. Shock tube tests are rapid and less costly but are limited in size to study individual component-level behaviour. Structures mounted to the shock tubes for testing can be planar and cover the entire blast flow area, such as stud walls (55) and structural insulated panels (59). A load transfer device (LTD) must be used when non-planar targets, such as beams, columns, and connections, need to be tested in a shock tube. LTD, as shown in Figure 2(d), are designed to increase the tributary area over which shock wave pressures act and apply the pressure as a series of dynamic point loads acting on the test specimen. The primary disadvantage of the LTD is that it increases the total mass of the dynamic system, although it allows for load cells to be placed at the supports to measure the dynamic reactions. Shock tube testing with a LTD to generate high strain rate response of timber elements has been successfully used by multiple authors (27; 29; 35; 46; 54; 61).

Regardless of the advantages and disadvantages of both experimental techniques, structural response of components tested subjected to far-field conditions are likely to be reproduced in a shock tube environment, as long as
the scaled distance is similar. However, response will be very different for close-in threats where blast pressure is non-uniform and likely generate shear/local failure, as shock tubes are unable to reach small scaled distances, typical from contact/close-in detonations.

Figure 2. Experimental techniques to study the response of wood structures to blast loading: (a) live explosion, (b) pneumatically-driven (University of Ottawa) and (c) gas-detonation shock tubes (Virginia Tech) and (d) four-point bending load transfer device. Reprinted from Jacques et al. (61; 27) with permission from the © Canadian Science Publishing or its licensors and the authors.

Dimensional Timber

Dimensional timber is most often used in the light-frame wood stud walls of balloon- and platform-framed light timber buildings. Both consists of ribbed panels acting together in partial composite action through metallic fasteners joining OSB or plywood sheathing panels to the dimensional timber framing members (55). While balloon wall studs extend from the sill of the first story
to the top plate or end rafter of the second story, platform-framed walls are independent for each floor (62).

The blast characterisation of dimensional timber construction has so far been limited to component-scale tests investigating the behaviour of individual studs (27) and full-scale walls with idealised boundary conditions (28; 30; 35; 53; 54; 55). Generally, bending failure of the dimensional timber is reported to occur near locations of maximum moment causing a brittle rupture in the tension zone leading to the complete loss of load carrying capacity. According to Jacques et al. (27), dimension timber Spruce-Pine-Fir (SPF) studs subjected to high strain rate four-point bending caused by a hemispherical explosion with a scaled distance $Z = 9.6 \text{ m/kg}^{1/3}$ tended to a flexural failure mode. The typical failure modes observed by the authors are shown in Figure 3. Interestingly, the authors reported that failure did not necessarily occur at locations of defects and knots as expected under static loads (63). In many specimens, rupture of the tension-side fibres was first observed in clear sections of timber in a sudden and brittle manner, as illustrated in Figure 3.

Figure 3. Typical failure modes of dimensional timber elements under blast loading. Reprinted from Jacques et al. (27) with permission from the © Canadian Science Publishing or its licensors.

The failure mode of light-frame stud walls is reported to be highly dependent on the sheathing and connections (35). When the sheathing has sufficient capacity to transfer applied blast loads to the dimensional timber studs, a predominantly flexural failure mode is observed consisting of a brash failure of studs with transverse fracture of longitudinal wood fibres, as illustrated in
Figure 4(a). Depending on the grade of the dimensional timber, however, the failure mode of the studs can shift to a more fibrous type of failure, when fibres tend to be longer and thicker, as suggested by Viau et al. (55). When the sheathing has inadequate capacity to resist the applied blast pressure, it will fail prematurely before transferring any significant load to the main framing members, as shown in Figure 4(b). Failure and fragmentation of the sheathing generates significant debris with high velocities that cause high hazard failures. Blowout failure of the sheathing is a commonly reported failure mode for light-frame construction under blast loading, having been reported by several authors (28; 30; 35). Moreover, field blast tests on full-scale light-frame wood structures have demonstrated that the overall damage to the structures is highly dependent on the damage of the rafters, studs and joists, as these are the main load bearing elements (64; 65; 66; 67).

![Figure 4](image.png)

Figure 4. Typical dynamic flexural failure mode of light-frame wood stud walls: (a) brash tension of studs and (b) sheathing fragmentation. Reprinted with permission of ASCE, from Viau and Doudak (40); permission conveyed through Copyright Clearance Center, Inc.

High strain rates induced by blast loads, typically ranging from 0.1 to 1 $\text{s}^{-1}$, result in an apparent increase of some material properties of the element during dynamic response, such as the modulus of elasticity, modulus of rupture, resistance and stiffness. This increase can be accounted for with a Dynamic Increase Factor (DIF), defined as the ratio of the dynamic property to its corresponding static value (68). While research studies on DIF as a function
of the strain rate are widely available for concrete (69; 70; 71; 72; 73) and steel (74; 75; 76), little research has been undertaken to establish the high strain rate behaviour of timber elements. Early research on the effects of high strain rate on wood has mainly focused on impact loading (77; 78; 79; 80).

The apparent properties of dimensional timber show a strong sensitivity to strain rate. Jacques et al. (27) conducted a series of shock tube tests to investigate high strain rate flexural behaviour of individual dimensional timber studs to determine dynamic increase factors applied to both the modulus of elasticity (MOE) and modulus of rupture (MOR). The authors tested thirty $38 \times 140 \times 2440$ [mm$^3$] SPF specimens in four-point bending by using a shock tube with and an LTD, subjecting the specimens to a simulated blast load characterised by a scaled distance $Z = 9.6$ m/kg$^{1/3}$. As the peak load transferred to the specimens was not recorded, the authors implemented an iterative SDOF model to estimate the actual dynamic stiffness of the specimens $K$, which was selected such that the predicted displacement time-history was in agreement with the experimental observation up to the time of failure. The determination of MOR was then computed based on the manipulation of the load-deformation equation incorporating the elastic member stiffness and relating the bending stress to the MOE and the strain at rupture $\varepsilon_d$, yielding

$$f_d = \frac{K\varepsilon_d a}{48I} (3L^2 - 4a^2),$$

while the corresponding dynamic MOE was determined by relating the measured strain at failure to the dynamic MOR using Hooke’s Law. By relating the dynamic and static properties, obtained through a four-point quasi-static and dynamic bending test, the authors reported a clear increase of both properties under blast loading. For strain rates in the range $[0.1, 1]$ s$^{-1}$, the authors reported a DIF applied to the MOR of 1.41, and a DIF applied to the MOE of 1.14. Additionally, the authors observed that the strain at rupture was enhanced by a factor of 1.18.

To assess the influence of high strain rates on the flexural response of full-scale light-frame wood stud walls, Lacroix and Doudak (28) tested twenty specimens, composed by six machine-stressed-rated timber studs sheathed with a mix of oriented strand board (OSB) and plywood sheets, under static and dynamic loading. Under static loading, the specimens were tested to failure under four-point bending, with spreader beams spanning edge to edge. The dynamic testing was conducted by subjecting the specimens to a simulated
blast load, with scaled distances ranging from 7 to 33 m/kg$^{1/3}$. Similarly to
the approach used by Jacques et al. (27), these authors implemented an SDOF
model to iteratively determine the dynamic stiffness and ultimate capacity, as
the direct measuring of the dynamic resistance is experimentally challenging.
Although two sheathing types were used in the wall systems, the strain rate
sensitivity of the wall systems was found to be similar. For strain rates in the
range of 0.17 to 0.54 s$^{-1}$, Lacroix and Doudak (28) reported a DIF = 1.40 for
the wall resistance and a DIF = 1.18 for the wall stiffness. Figure 5 shows the
relative increase of resistance and stiffness for all tested walls, which is in good
agreement with further testing of light-frame wood stud walls, under strain rates
ranging from 0.19 to 0.91 s$^{-1}$, conducted by Viau and Doudak (35). The reported
relative increase in dynamic properties was also consistent with the findings
of Jacques et al. (27) for individual dimensional timber studs. Additionally,
Lacroix and Doudak (28) compiled a database of high strain rates from tests
on dimension timber to establish the influence of strain rate on their dynamic
properties, reporting that there is a strong correlation between the increase in
strain rate and the relative increase in timber strength at the component level.
For strain rates ranging from $10^{-5}$ to $10^3$ s$^{-1}$ this can be expressed as

$$
DIF = 1.46 + 0.1 \log \ddot{\varepsilon}.
$$

Figure 5. Experimental measurement of relative increase in (a) resistance and (b)
stiffness of light-frame wood stud walls comparing to their quasi-static behaviour.
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Cross-Laminated Timber
Cross-Laminated Timber (CLT) is an EWP constructed from multiple layers
of timber boards stacked at right angles and glued on their wide faces. This
structural configuration provides good in- and out-of-plane strength and
stiffness (29). Cross-laminated timber is perhaps one of the most studied type of EWPs under blast effects (see Table 1). This can be attributed to the fact that CLT panels have less variable mechanical properties than dimensional timber, leading to much more accurate design processes, which makes it an excellent candidate for blast protection purposes (46; 81).

The failure mode of CLT members is reported to exhibit strain rate sensitivity. Poulin et al. (29) conducted an experimental study where the out-of-plane static and dynamic behaviour of 3- and 5-ply CLT panels using a shock tube and four-point bending LTD. These authors reported that the predominant static failure mode was flexure, while some specimens under dynamic loading experienced significant rolling shear in addition to flexure failure, as shown in Figure 6. Rolling shear was characterised by an inter-fibre cracking due to shear strains in the plane perpendicular to the longitudinal axis of the wood fibres. Interestingly, Poulin et al. noted that due to the early onset of rolling shear failure, the longitudinal laminates deflected almost as individual elements rather than a composite section. Furthermore, tension-side failure initiated in locations such as finger joints or defects, followed by damage propagation throughout the section. Such complex failure modes were also reported by Viau et al. (52), who observed a degradation of the stiffness when significant rolling shear with no flexure failure was observed.

Weaver et al. (58) conducted the first of its kind full-scale explosive tests on three 4.6 m² footprint two-storey CLT structures with a single bay (see Figure 7(a)) attempting to demonstrate the ability of CLT construction to resist blast loading. Tests were conducted using 0.45 kg solid TNT charge at a stand-off distance of 22.9 m. Two of the structures had 3.66 m high storeys and one had 3.05 m high storeys. Aside from story height and CLT panel grades, the dimensions and detailing of the three structures were identical. The structures included proper boundary connections between panels. Cases with and without superimposed dead loads were investigated by placing concrete blocks on the first floor and roof, with the former aiming to simulate a five-story office/residential building. The authors reported that the structures with dead load exhibited less damage than those without dead load for the same loading conditions, for all the CLT grades. Failure of the front panels was characterised by a flexure failure of the same laminates on the tension side of the front face, as shown in Figures 7(b-c), which is in accordance with the findings of Poulin et al. for single CLT panels (29).
Poulin et al. (29) addressed the out-of-plane behaviour of CLT panels under blast loading in the only publicly available study so far. In total, eighteen cross-laminated timber panels composed of three and five-plies of spruce-pine-fir timber were tested under shock tube blast loads and quasi-static four-point bending. These authors included a load cell between the specimens and the supports to directly record the dynamic support reactions. Dynamic resistance functions were then calculated as a function of the recorded dynamic reactions and force-time histories, assuming the specimen to be a simply supported beam with distributed mass to resolve inertial loads. The dynamic resistance curves exhibited an increase in strength when comparing to the static data, but an evident increase in member stiffness was observed, as shown in Figure 8. Poulin et al. reported an average DIF of 1.28 on the maximum resistance of the specimens. It should be noted that these are applicable to CLT specimens exposed to blast waves representative of far-field blast explosions with uniform loading and minimal localised damage.

Two studies on CLT panels were conducted by Lowak et al. (82; 83) testing a series of simply supported panels under out-of-plane simulated blast loads. These authors also proposed an SDOF model to predict the behaviour of the specimens. While the reports are unpublished, it has been described in the
literature that properties of CLT under blast loading can be calculated from manufacturer data using a 0.70 factor applied to the stiffness and a strength increase factor of 1.60 applied to the recommended design stress (84).

Glued-Laminated Timber

Glued-laminated timber (glulam) consists of a stacking sequence of two or more layers of dimensional timber bonded together with durable structural adhesives, where the grain of all layers runs parallel to the length of the member. While glulam can be manufactured with one single timber board per layer, multi-piece laminations are also frequently used, where two or more pieces of timber are placed together across the width. The applicability of glulam typically ranges from simple, straight beams or columns to complex, curved members. Due to long-span requirements of specific projects, individual timber layers are connected at their ends by cutting a set of complementary, interlocking profiles in the two pieces of wood, increasing the surface area for
gluing and enabling a larger force to be transmitted across the discontinuity at the end of one lamination to the next. This is commonly referred to as finger jointing, which allows for continuous members of any required length. Due to different manufacturing approaches, multi-piece lamination members can be found to have closely-aligned finger joints (FJs) over the width of the member or randomly distributed, commonly referred to as staggered FJs.

Blast-loaded glulam beams and columns have been typically tested in shock tube testing facilities (31; 36; 37; 56; 85). A common finding among these studies is that FJs affect the behaviour of glulam members subjected to blast loading. Lacroix and Doudak (31) reported that the presence of FJs in single-piece lamination members are most likely to induce failure initiation at their location, typically leading to brash tension failure with some longitudinal cracking. On the other hand, multi-piece lamination members generally experienced significantly more crack propagation and splintering, as shown in Figures 9(a-b). Similar failure modes were also reported by Lacroix et al. (85), who presented the results of a group of 80 × 228 [mm$^2$] cross-section glulam beams under simulated blast loading in four point bending. Specifically, their findings indicate that the initiation of failure at FJs is more likely to happen when these are located in the constant moment region. However, while some beams developed a brash tension failure, others developed a simple tension failure, without reaching the outer compression fibre.
Several authors have studied the effect of axial loading on the performance of glulam elements used as columns. Lacroix and Doudak (50) compared the dynamic failure mode of 137 × 222 [mm²] cross-section glulam members with and without axial loading subjected to simulated blast loads. These authors found out that axial load was not constant throughout the response of the column and that it dropped at a consistent rate independent of axial-load levels. It was also reported that the application of an axial load induces damage around the member’s mid-span, with failure being characterised by significant compression damage and wrinkling of fibres, as shown in Figure 9(c). Such findings are also supported by previous research (31; 49).

Considerations on strain rate sensitivity of glulam have been limited to the studies conducted by Lacroix and Doudak (38; 49; 56) at the University of Ottawa, who observed an enhancement of its ultimate strength under high strain rates. Thirty-eight full-scale 2.4 m long glulam timber beams with different dimensions were statically and dynamically tested to failure under four-point bending (56). Blast testing was conducted using a shock tube and load transfer...
device, generating high strain rate responses in the range of $0.14 - 0.51 \text{ s}^{-1}$.

The performance of glulam beams was closely correlated with how the finger joints were distributed along the member. Findings indicate that single-piece lamination members, where only one continuous FJ running across the width of the element and near the maximum moment region (see Figure 10(a)), had a significantly lower resistance than multi-piece lamination members with closely-aligned (Figure 10(b)) and staggered FJs (Figure 10(c)). The results have shown that a DIF = 1.14 in flexural strength is appropriate for glulam members that have staggered FJs in the outer tension lamination. By contrast, no DIF is observed when continuous of closely-aligned FJs are present. Such conclusions were also highlighted by Lacroix and Doudak (38), who tested seventy unretrofitted and FRP retrofitted glulam beams and columns. An additional observation indicate that no dynamic increase was found for the stiffness and failure strain.

![Glue-laminated specimens tested by Lacroix and Doudak: (a) single-piece lamination and multiple-piece lamination members with (b) closely-aligned and (c) staggered FJs. Reprinted with permission of ASCE, from Lacroix and Doudak (56); permission conveyed through Copyright Clearance Center, Inc.](image-url)
Structural Insulated Panels

Structural insulated panels (SIPs) are panelised construction systems composed of a lightweight foam core between two oriented strand board (OSB) facers. This structural system is particularly relevant due to its insulation properties and high stiffness, which are generally primarily related to the thickness of the foam (59). At a material level, OSB facers are known to be strain-rate dependent, as identified by Meng et al. (86) after a series of dynamic tensile tests. The authors concluded that, for a strain rate up to $28.7 \text{ s}^{-1}$, the material tensile strength is approximately twice the quasi-static tensile strength. Generally, the authors found that, for strain rates ranging from $4 \times 10^{-5}$ to $28.7 \text{ s}^{-1}$, there is a steady increase of the dynamic tensile strength with the strain rate as

$$\sigma_{\text{dyn}} = \sigma_{\text{stat}} (1.776 \log(\dot{\varepsilon}) + 14.27),$$  \hspace{1cm} (3)

where $\sigma_{\text{dyn}}$ and $\sigma_{\text{stat}}$ are the dynamic and quasi-static tensile strength, respectively.

When subjected to axial or out-of-plane transverse loads, SIPs exhibit a sudden, brittle failure, after a linear load-deflection response, with highly variable failure modes (87; 88). Phillips et al. (59) tested 95 mm thick polyurethane closed-cell foam core with 11 mm OSB facers SIP panels subjected to simulated blast loads, as shown in Figure 11(a). Simulated roof and floor diaphragms were installed to include the effect of realistic boundary conditions. The authors reported that as-built SIP panels exhibited poor performance when subjected to blast loading, with a controlling failure mode of longitudinal splitting of the inlet nailer at the top of the wall for the roof connection and shear failure of the lag screws connecting the roof SIP to the inlet nailer. Such failure mode is consistent with previous research on light-frame wood stud walls subjected to blast loads (35). In addition to connection failure, OSB panels failed after delaminating from the foam core, as shown in Figure 11(b).

Performance of connections

As timber structures tend to be statically determinate, the ability of connections and fasteners to maintain a continuous load path and dissipate energy is of extreme importance under blast loading. Often, these connections are either proprietary or developed on a project-by-project basis. Ideally, timber connections should be designed to yield and dissipate energy as timber structural
Figure 11. SIP panels under simulated blast loads: (a) test setup and (b) failure due to connection failure and consequent OSB fracture. Reprinted with permission of Elsevier, from Phillips et al. (59); permission conveyed through Copyright Clearance Center, Inc.

elements tend to be brittle (89). Research on timber connections has mainly focused on seismic behaviour (90; 91; 92) and there has been little research into the behaviour of structural timber connections subject to blast loading. To date, research has been limited to the study of connections in light-frame wood stud walls, wall-to-floor connections in CLT assemblies and exploratory testing of bolted connections on plain wood.

Some authors have discussed the behaviour of proprietary and commercially available wall-to-floor and wall-to-roof connections. Viau and Doudak (41) conducted a series of blast tests on 2070 × 2108 [mm²] light-frame wood stud walls with varying connection details. Examples of the tested connections and typical failure modes are shown in Figure 12. The use of prescriptive connection detailing, such as nailed/screwed connections, is not adequate to maintain connectivity to the floors prior to the wall studs reaching their full capacity, leading to premature failure and production of flying debris during blast. The use of joist hangers, however, allowed the studs to reach their ultimate capacity and fail in flexure, contrarily to the use of nails and screws. Twists straps also performed poorly, leading to either rupture of the straps or splitting of the stud. The use of steel angles, however, performed reasonably well and allowed for all the studs to achieve their ultimate capacity with little damage to the connector. It caused a slight increase in rotational stiffness reducing the maximum and
residual deflections of the stud. This indicates that while an over-design factor is appropriate to ensure development of full capacity of the structural elements, the type of connector and construction detailing are of great importance to the performance of the wall system (41).

In CLT construction, connections may enhance the performance of the elements by providing energy dissipation mechanisms, preventing the wall elements from reaching their intended flexural capacity. Côté and Doudak (42; 46) investigated the effects of realistic boundary conditions on the behaviour of CLT walls when subjected to simulated out-of-plane blast loads (see Figure 13). These authors performed 13 full-scale dynamic tests on 5-ply 2082 × 450 × 175 [mm³] CLT panels. To simulate the in-plane stiffness of the floor diaphragms above and below the CLT wall, specimens were connected to two horizontal end blocks cut from undamaged CLT panels. Although screw-type connections were designed to resist 20% more load than the anticipated dynamic reactions, they performed poorly when subjected to out-of-plane loading and failed prematurely in brittle tension. On the other hand, bearing type connections, where compression perpendicular to grain failure is promoted, performed well even when under-designed for the expected dynamic reactions. Steel yielding
of thin steel angles provided significant energy dissipation and wood crushing, which resulted in reduced overall deflections. Although thicker angle connectors experienced little deformation in the angle itself, the connection translated appreciably due to the deformation of the screws, which provided a certain degree of rotational restraint. Furthermore, using balloon type construction detailing provided fixity at the wall-to-floor locations due to the continuous wall span. As the possibility for any brittle tension perpendicular to grain failure was eliminated using this solution, the failure mode for this connection type was dominated by bending.

Figure 13. Typical CLT wall-to-floor connections and failure modes: (a) end-grain screws, (b) thin steel angles, (c) thick steel angles and (d) balloon type construction. Reprinted with permission of Elsevier, from Côté and Doudak (46); permission conveyed through Copyright Clearance Center, Inc.

McGrath et al. (45) measured the dynamic dowel embedment strength and stiffness of bolted connections on plain wood specimens. The authors used steel side plates with a single bolt connecting the wood specimens, in both parallel and perpendicular directions to the grain. When loaded in blast parallel to the grain, the wood specimens exhibited a small load plateau (due to wood cell crushing) before failing by wood splitting. To avoid the splitting, tests were repeated with self-tapping screws applied into the wood, perpendicularly to the bolted connection. In that case, specimens were kept together, suppressing the brittle tension and significantly improving the ductility of the connection. Specimens loaded in the perpendicular direction to the grain, however, exhibited
splitting only at large displacements, which was not detrimental to the load
carrying capacity of the connections. McGrath et al. reported that the yield
load and stiffness experienced DIFs of 1.09 and 1.20, respectively, in the direction
parallel to the grain, and DIFs of 1.24 and 1.77, respectively, in the direction
perpendicular to the grain.

Similarly, Viau and Doudak (47) conducted a series of tests on glulam beams
aiming to better understand the performance of slender bolts within full-scale
glulam assemblies, when subjected to blast loads perpendicular to the wood
grain. Shock pressure waves were converted into two concentrated loads applied
directly onto the specimens with an LTD. Eleven specimens with representative
beam-to-floor connections were tested with various steel–wood–steel bolted
connections, consisting of 12.7 mm diameter, 203.2 mm long bolts with 6.4 mm
thick steel side plates, as shown in Figure 14(a). These authors observed that
failure in the connections was dominated by bolt bending (yielding) and wood
crushing caused by deformation of the bolts, as visible in Figures 14(b-c). This
was most prevalent in specimens where wood splitting did not occur. On the
one hand, all glulam beams failed prior to attaining an ultimate shear failure in
the bolts, which explains the limited bolt deformation observed in Figure 14(b).
On the other hand, specimens designed to fail in brittle perpendicular-to-grain
splitting, achieved by an increased unloaded edge distance, induced lengthwise
cracks passing through the centre line of the bolt holes and propagating
throughout the length of the beam, as shown in Figure 14(d). Viau and Doudak
reported that, overall, connections designed to yield in bolt bending performed
better than those that were over-designed.

Viau et al. (93) developed and tested specially designed energy-absorbing
connections (EACs) for glulam members, such as those shown in Figure 15.
These authors reported that EACs fully deformed and reached densification,
which significantly increased the energy-absorbing capabilities of glulam
assemblies. As the connection yielded, it limited the load on the wood
member, delaying failure. Assemblies equipped with these EACs were capable
of withstanding greater blast pressure-impulse combinations when compared
to typical glulam and CLT connections. It has been shown that the type of
connections and detailing play a significant role in the performance of timber
structural systems. Typical connection details used to resist gravity and in-
plane shear loads cause premature failure of the systems and are inadequate for
out-of-plane blast loads.
Figure 14. (a) Representative wall-to-floor bolted connection; typical (b) bolt yielding and (c) wood crushing damage, and splitting failure in over-designed beams. Reprinted with permission of ASCE, from Viau and Doudak (47); permission conveyed through Copyright Clearance Center, Inc.

Retrofitting techniques for blast-deficient timber structures

Blast retrofitting may be necessary when existing timber structures have design deficiencies leading to inadequate strength, stiffness and ductility. Most studies have focused on retrofitting techniques to reduce overall damage levels at component level by allowing the members to develop flexural capacity beyond the peak resistance.

Blasts tests on light-frame walls have shown that some types of sheathing are prone to blow out failures, causing hazardous flying debris. Viau and Doudak (33; 35) proposed special sheathing details to increase sheathing capacity to transfer loads to the studs, as well as catcher systems to limit debris if sheathing failure can not be avoided. Viau and Doudak reported that stud walls with typical 11 mm OSB sheathing panels fastened to the studs with nails
tend to fail prematurely at pressures higher than 35 kPa. This type of failure, shown in Figure 16(a), created significant debris due to failure of connections between the sheathing and the studs. These authors also tested an improved sheathing design using 18.5 mm plywood fastened to the studs with screws. The new design, shown in Figure 16(b), shifted the failure to the studs after bending failure of the sheathing at the wall mid-height, decreasing the amount of sheathing debris. Lacroix et al. (32) reported similar levels of blast enhancement when plywood panels were used in light-frame walls, rather than OSB sheets, which exhibited poor performance.

When the use of OSB sheathing in light-framed walls is unavoidable, welded wire mesh (WWM) can be used as a reinforcement, as investigated by Viau et al. (33; 35). When WMM is applied in the compression side of the studs, placed at the OSB/studs interface, it acts as a reinforcement of the sheathing, allowing for larger displacements to occur in the OSB. Contrarily, when applied on the outer face of the studs, it mainly works as a catcher system, allowing for the axial capacity of the studs to remain unchanged. Both attempts showed that the WWM allowed the pressure to be completely transferred to the studs when used as a reinforcement and successfully capture all the sheathing debris when used as a catcher system, as shown in Figures 16(c) and (d).

Although the incorporation of non-timber materials into a composite timber-based element has not been extensively evaluated, exploratory testing was conducted by Viau et al. on the replacement of wood sheathing panels by corrugated steel panels commonly used in metal deck floor construction.
These authors also reported that such a solution induces a significantly stiffer behaviour of the global system, which resulted in significantly different cracking patterns and higher post-blast axial capacity.

The use of fibre reinforced polymers (FRP) in other construction methods, such as reinforced concrete, has been a first choice for strengthening elements under blast loading (94; 95). Its potential to strengthening timber elements was investigated by Lacroix and Doudak (34; 36; 37; 38). The authors investigated the effect of glass-fibre reinforced polymers (GFRP) on the flexural behaviour of glulam beams by using different retrofitting solutions, as shown in Figure 17. Depending on the solution, peak resistance and displacement at peak resistance increased by 1.35 – 1.57 and 1.3 – 1.62, respectively. Nevertheless, the use of tension-only reinforcement induced premature debonding between the GFRP and the wood as the outer tension laminates pushed the FRP outwards, causing separation between the two materials, as shown in Figure 17(a). Contrarily, the use of additional FRP confinement over the full length of the member was found to prevent the loss of composite action, altering the failure mode from simple tension failure to a combination of brash tension and compression failure, limiting the damage to a small region, as shown in Figure 17(b). While neither of the two retrofitting options discussed above provided significant post-peak resistance, the addition of a U-shaped tension reinforcement with partial-length confinement provided a post-peak resistance of approximately 75% of peak resistance, as the partial-length confinement was able to delay the debonding between the FRP and wood beyond peak resistance. Results also
indicated that the addition of FRP on glulam beams contributed to arresting crack development and bridge defects.

![Image of glulam beams retrofitted with FRPs](image)

**Figure 17.** Description of the glulam retrofitted beams, and corresponding failure modes, tested by Lacroix and Doudak: (a) UD longitudinal FRP, (b) UD longitudinal FRP with full-length UD confinement and (c) UD U-shaped FRP with partial-length UD confinement. Reprinted with permission of ASCE, from Lacroix and Doudak (37); permission conveyed through Copyright Clearance Center, Inc.

FRPs have also been employed in retrofitting insulated structural panels. Aiming to increase the flexural strength of SIPs, Phillips *et al.* (59) attached FRP wallboards to the front and back faces using structural adhesive. In this case, failure of the panels initiated with a shear crack located in the top third of the panel, as visible in Figure 18(a). These authors observed that the FRP reinforced SIP withstood maximum pressure and impulse that were 1.4 and 3.0 times larger than the as-built panel, respectively.

In addition to FRPs, Phillips *et al.* (59) have also evaluated the use of three 38 mm wide, 0.9 mm thick galvanised steel straps attached to both faces of the panels using long screws. The additional strength provided by the steel straps was found to dramatically increase flexural resistance and prevent flexural failure of the OSB, forcing the panels to fail in shear, as shown in Figure 18(b).

Similarly, Lopez-Molina and Doudak (39) conducted an experimental testing campaign on 3 and 5-ply CLT panels using a fabric retrofit solution. The authors reported that, regardless the retrofit configuration (U-shaped fabric and full confinement), the use of GFRP layers enhanced the overall behaviour of the panels resulting in a significant increase in peak resistance, ductility, and post-peak resistance. Particularly, peak resistance and ductility of the specimens were

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increased by factors 1.3 – 1.6 and 1.6 – 3.0, respectively. Between the retrofit solutions, it was noted that the U-shaped configuration delivers the best results, which is in accordance with the findings of Lacroix and Doudak (37) regarding glulam beams.

Tensioned steel strapping, commonly used to package industrial equipment, was studied as a retrofit technique to reduce or eliminate the effect of rolling shear in CLT panels subject to blast loads. Steel straps have mostly been used as a way to provide active and passive confinement pressures in bridge columns, which in turn improves the ductility, flexural response, and shear capacity of the concrete columns. A recent study, however, investigated their efficiency in improving the flexural capacity of CLT members under blast loading (39). Lopez-Molina and Doudak tested 3 and 5-ply CLT panels retrofitted with 9 width and 0.74 mm thickness. These authors observed that using transverse steel straps only at the bottom and top thirds of the slab had minimal effect on the flexural behaviour. Contrarily, their use at every laminate significantly increases the post-peak resistance of the member. Additionally, when longitudinal and
transverse wraps are simultaneously used, a reduced rolling shear failure and
the complete elimination of flying debris was achieved, as shown in Figure 19.
Although this technique allows higher peak resistance and initial stiffness, it
also induces delamination between the longitudinal and transverse wood layers.

![Figure 19](image1.png)

**Figure 19.** Example of the damage observed in CLT panels retrofitted with steel straps
under blast loading: (a) failure mode and (b) detail of the compression and tension zone.
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**Modelling of timber structures**

Most of the research conducted on the behaviour of light-frame and heavy timber
under blast loading is based on experimental testing. It is extremely useful,
however, that engineers and designers can access simplified analytical models
for predicting the behaviour of a given element, expediting the design process
of timber protective structures.

Single degree-of-freedom (SDOF) models, which consist of transforming a
structural element with distributed properties into an equivalent system, by
means of a lumped mass and a spring representing the member stiffness, have
been implemented by several authors for predicting the dynamic response of
timber structures under blast loading.

Lacroix *et al.* (28; 31; 49; 56) have modelled the behaviour of glulam beams
and columns and reported that SDOF models can be effective in predicting
the maximum displacement and time to maximum displacement, providing
resistance curves that can accurately capture the behaviour of the element are
used. Linear-elastic relationships were found to adequately capture the dynamic
response of glulam beams. As for CLT, step-wise relationships with drops in
resistance, representing the loss of the bottom laminates, have been proposed
by several authors, delivering accurate predictions on the behaviour of CLT
members under blast loading (42; 44; 46; 52; 81). SDOF modelling has also
been employed to predict the behaviour of light-frame wood stud walls. Viau et
al. (35; 40; 53) proposed simplifying wood stud walls as T-sections composed
by one stud with the flange pertaining to its tributary width. According to
the authors, the model was found to accurately predict the behaviour of the
wall studs when sheathing tearing was observed, as long as full transfer of the
pressure to the studs is ensured. However, SDOF modelling is found to be an
unreliable method when heavy damage and failure occurs in the connectors due
to its inability to account for the effects of rotational restraint from the boundary
connections, as reported by Viau and Doudak (41).

While the majority of studies have primarily dealt with idealised boundary
conditions, a few studies have focused on the modelling of realistic conditions.
Côté et al. (42; 46) concluded that simplified SDOF models developed
for idealised simply-supported conditions are not adequate to describe the
behaviour of CLT panels with representative wall-to-floor connections, either
screws or angle brackets. The model over-predicted the deflection of the panel
and was not capable of incorporating the deflections in the connections. In
an attempt to overcome this, Viau et al. (44; 81) proposed a two degree-of-
freedom system (TDOF) where the CLT assembly was represented by a beam
element connected to springs, accounting for the lateral stiffness of the various
connections. This approach was found to adequately capture the system’s
response with reasonable accuracy in terms of maximum mid-span displacement
and time-to-maximum mid-span displacement, as well as the correct failure
mode (i.e. connection or panel failure).

Although it is common practice amongst designers and engineering
practitioners to choose simplified models over more complex analyses, resorting
to more resource-intensive finite element models allows for more detailed
and thorough investigations. While multiple studies have focused on the
development of FE models to evaluate the fire resistance (96; 97; 98) and
seismic performance (99; 100; 101) of mass timber products, the use of such
models for blast analysis is still very scarce. In one of the studies, Sliseris et
al. (57) presented a non-linear numerical simulation of a CLT wall subjected to a blast wave with a peak reflected pressure of 10 kPa. The authors reported that the deformation of shear wood layer has a major impact on the predicted behaviour of the wall. The model was developed considering full bond action between timber layers, which might be over simplistic to accurately predict the complex inter-layer interactions. More recently, Van Le et al. (60) developed and validated a FE model of CLT panels against experimental testing, taking into account the anisotropy of each CLT layer. These authors conducted a sensitivity analysis on the the stiffness of the panel and reported that it has a marginal effect on the peak displacement. Their model was also used to investigate the strain rate dependency of the modulus of elasticity, which was found to increase with increasing blast pressures, leading to a DIF in the range 1.05 – 1.43.

In a different study, Chen et al. (102) developed and calibrated a FE model of a structural insulated panel aiming to study its blast resistance and energy absorption capacities when subjected to far field explosions. Although the authors used metal skins instead of typical OSB boards, the modelling approaches and simplifications adopted could be applicable for studying timber-based SIPs.

Design Codes, Standards, and Guidelines

There are very few published design codes, standards, or guidelines related to blast-resistant design of timber structures. The Canadian Standard CSA S850 “Design and assessment of buildings subjected to blast loads” is the first and only structural code that provide explicit requirements for the blast design of timber buildings (103). Its provisions for timber design, however, are still underdeveloped due to the limit experimental testing available at the time they were developed. Generally, the CSA S850 approach is to compute the dynamic design resistance of wood elements using a DIF of 1.40 applied universally to the material strength of all types of wood products under blast loading. Recent research has, however, indicated that this overly generic approach does not reflect the behaviour of all wood products, as the difference in behaviour between defect-free and in-grade wood raises the question of how applicable these results are to the practical design of blast resistant wood structures (27; 31; 48). Other studies also raise concerns about code response limits, which tend to overestimate the ductility of wood elements, especially those corresponding to hazardous damage levels (55).
Typically, blast design standards also advise on the design of connections and anchorage systems. CSA S850 (103) recommends connections in wood elements to be over-designed to allow the member to fully develop their resistance and/or ductility capacity, with a governing limit state of bearing failure in the wood member or connector. Such requirements may not be appropriate for the design of timber structures due to their brittle nature, as requiring the connections to be over-designed may shift the failure mode of the structural element (44; 53). Additional guidance on how to retrofit timber elements to enhance their performance under blast loading is not available in the Canadian blast design standard.

Wood components are also referenced in ASCE/SEI 59-11 “Blast Protection of Building” (104). ASCE/SEI 59-11 considers that structural systems designed using timber shall meet or exceed the provisions of “The American Forest Paper Association’s National Design Specification for Wood Construction” (105) and meet ductility limits of $\mu = 1$ (superficial damage), $\mu = 2$ (moderate damage), $\mu = 3$ (heavy damage) and $\mu = 4$ (hazardous damage) for blast considerations. The code refers to the material as dimensional timber, and recommends, however, that specific test data, if available, should be used, as the response limits documented are based on very limited test data from temporary light-frame military structures. It does not provide any response limits for mass timber components under blast loading or design guidelines, which is a major shortcoming of this document.

More recently, the US Army Corps of Engineers published a technical report (PDC-TR 18-02) on the analysis of CLT construction subjected to airblast loads (106). The report defines that the resistance of a CLT panel subjected to out-of-plane airblast loading shall be computed assuming one-way action, suggesting the use of bilinear or trilinear relationships. The document also reports on response limits of CLT structures, based on qualitative damage observed from the experimental tests performed by Weaver et al. (58). PDC-TR 18-02 defines how to determine expected material properties for use in the analysis, how to construct a resistance function appropriate for CLT panels and how to assess the results of SDOF analyses using the documented response limits. It does, however, lack extensive experimental support for the general recommendations made throughout the document regarding the characterisation of the structure’s resistance and boundary conditions.

Overall, given the infancy of existing design codes, there is no practical information on which EWP systems are preferred for different blast...
conditions/structural use. Nevertheless, based on the current literature, it can be estimated that different EWPs provide better response than others for particular conditions. Exterior cladding is the first line of defence of a structure and, for that reason, extra precaution should be taken when selecting the EWP type. For high scaled distances the use of light-frame stud walls or structural insulated panels might be a good option, while heavy cross-laminated timber panels are a better choice in case of near-field or contact detonations (low scaled distances), due to the low probability of generating flying debris. Lateral force resisting systems, as moment-frames or shear walls, greatly benefit from the use of dimensional timber studs in the event of blast loading with high scaled distances, while cross-laminated timber panels and glue-laminated timber beams and columns are preferred when dealing with explosions characterised by a low scaled distance, as their ability to withstand high intensity and localised shock waves play a significant role. Although such preliminary conclusions can be drawn based on studies addressed in this review, further research and support is still needed. Nevertheless, they can help guide practitioners and engineers on the use of mass timber products in protective design.

**Future perspectives and conclusions**

This paper presents a comprehensive review of experimental and analytical/numerical research on the blast performance of light-frame and heavy-timber structures. It provides insights of experimental testing of timber structures under high strain rates, performance of mechanical connectors, external retrofitting and modelling of timber structures. Furthermore, prescriptions included in design codes and technical reports considering timber to resists blast loading are discussed, highlighting key aspects to which designers and engineers may need to pay special attention. Existing research has indicated that timber structures can effectively resist blast loads and be considered excellent candidates for blast protection. While much of the available research is qualitative, there is still a general lack of understanding on the fundamental behaviour of timber under blast loading.

Despite the significant findings on the blast performance of light and heavy timber structures, further research efforts are still required to thoroughly understand its complex behaviour under blast loading. Some specific aspects that have been identified are listed below:
1. Increasing axial load levels on timber elements subjected to out-of-plane blast loading have been found to contribute for a loss in moment capacity. However, the correlation with member height and cross section dimensions is still unclear and not accounted for in the generation of resistance-deflection relationships.

2. Given the brittle nature of timber elements, mechanical connectors play a significant role dissipating energy through plastic deformation. When not properly designed, however, premature failure of the structural systems may occur. While some studies have been conducted on commercially available connectors, a comprehensive study for blast resistant applications is still lacking, as the understanding of parameters such as strength, ductility and energy dissipation capacity is still in its early stages.

3. Traditionally, building components support additional non-structural elements, such as insulation and gypsum wallboard, that may become flying hazard debris. While the added mass and stiffness may improve the structural performance from a theoretical point of view, their presence can potentially shift failure modes and the overall response of timber components. Although this is a critical aspect of protective design, available research is still limited when it comes to timber components.

4. Retrofitting of timber elements aim to increase the flexural capacity of bending elements and/or to reduce the hazard of flying debris due to wood failure. While studies have indicated that blast resistance of timber structures can be significantly increased through the use of bonded FRPs or steel straps, the development of full bearing capacity is highly dependent on suitable retrofit anchoring systems. Further studies on the design of these anchorage systems based on specific timber components should be carried out.

5. Previous studies have solely concentrated on far-field detonation effects, while conclusions drawn on timber structures subjected to close-range detonations are nonexistent. Additionally, the study of shear transfer mechanisms due to blast loading is still very scarce or nonexistent.

6. Simplified analytical methods can accurately predict the response of timber structures, provided resistance curves that accurately capture the global load-deformation response of the timber component are used. However, SDOF techniques break down when retrofitting techniques affect the resistance characteristics of timber components or heavy damage/failure occurs in the connections;
7. Further efforts shall be devoted to developing finite element models capable of predicting the dynamic failure modes of timber structures and the complex wave-structure interaction;

8. Prescriptions included in design codes for the analysis and design of timber structures under blast loading are still underdeveloped and not species-specific, as they generally consider timber as a universal material. Additionally, codes typically recommend connections to be over-designed to allow the member to fully develop its resistance capacity. This may shift the global failure mode due to timber brittle nature. Further development of design codes is highly recommended;

9. Blast testing has been typically conducted on a case-by-case basis given the lack of testing standards for blast loaded timber components. Researchers and manufacturers will greatly benefit with the development of dedicated testing protocols, allowing for a comprehensive timber characterisation under impulsive loading environments.

Declaration of conflicting interests

The Authors declare that there is no conflict of interest.

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