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

PROPOSED APPROACH FOR THE DESIGN OF TIMBER CONNECTIONS SUBJECTED TO BLAST LOADING

Christian Viau[1], Antoine Bérubé[2], Ghasan Doudak[3]

ABSTRACT: A near-decade-long comprehensive test program on timber connections subjected to shock tube simulated blast loads has been undertaken. A generalized capacity-based blast design methodology is presented, based on the experimental test results, aimed at promoting ductility in connections and a sequence of failure that seeks to minimize occupant harm during extreme load events. Key results on connection behaviour, typical failure modes observed, as well as overstrength factors for capacity-protected structural elements will be discussed, and a generalized design approach for the design of timber connections will be presented. The proposed generalized design methodology is also evaluated using pressure-impulse diagrams, in which the potential enhancement in the performance and energy dissipation of timber assemblies, designed with proper failure hierarchy in the connections and load-bearing timber elements, is demonstrated. The main outcome of this research program will guide the development and paradigm shift in blast design guidelines for timber connections.

KEYWORDS: blast, connections, timber, capacity-based design, ductility, pressure-impulse diagram

1 – INTRODUCTION

Blast loads generated by far-field explosions involve high magnitude and short-duration loads that have the potential to cause devastating effects to infrastructure and occupants. To mitigate these effects, engineers aim to design structural components (e.g., connections) to dissipate energy through inelastic deformations to avoid catastrophic failures occurring at both the component and system levels. This design approach is similar, in principle, to that used in seismic design, where the connections are generally the main source of energy dissipation and where timber elements are designed to remain elastic. However, current blast design provisions for timber promote overdesigning the connections while ensuring that structural elements fail first. The logic behind that is to eliminate failure in the boundary connections such that premature failure of the system does not occur before attaining the full capacity of the main structural element. While detailing for ductility can be attained in reinforced concrete and steel elements, timber elements have been known to lack the inherent ductility required to dissipate energy through inelastic deformation due to the brittle nature of timber when the failure occurs in flexure, shear or tension perpendicular-to-grain. Since timber is ductile in compression parallel- and perpendicular-to-grain, controlled design and detailing of the connections to yield and dissipate energy, while allowing the main structural element to develop full capacity, can help optimize the structural system and enhance the overall performance. While contemporary blast design standards (e.g., CSA S850 [1]) do not provide designers with guidelines to properly detail structural elements for such energy optimization, the concept has, in principle, already been proposed [2]. This paper summarizes the available knowledge on timber connections under blast loading and explores practical implementation of connection design to promote more energy dissipation while maintaining the system's structural integrity. It also proposes design philosophies for various timber connection types to resist blast loads.

2 – BACKGROUND

Established research on timber elements subjected to blast loads has mainly focused on establishing the material behaviour under high-strain-rate effects. A correlation between increasing strain rate and strength has been observed through small-scale testing of wood

[1] Christian Viau, Assistant Professor, Dept. of Civil & Environmental Engineering, Carleton University, Canada, christian.viau@carleton.ca

[2] Antoine Bérubé, PhD Student, Dept. of Civil Engineering, University of Ottawa, Canada, aberu025@uottawa.ca

[3] Ghasan Doudak, Professor, Dept. of Civil Engineering, University of Ottawa, Canada, gdoudak@uottawa.ca

 https://doi.org/10.52202/080513-0129

specimens [3–5]. Dynamic increase factors (DIF) have been developed on full-scale elements, primarily through extensive investigations using a Shock Tube apparatus. Examples include testing on individual wood studs [6], light-frame wood wall assemblies [7–10], glulam beams and columns [11–17], and CLT panels [14,15,18–20]. In order to eliminate variability in the results, these studies have mainly been conducted using idealized boundary conditions (e.g., simply-supported), which is consistent with overdesigned connections that undergo little to no deformation during loading. Certain timber assemblies, such as light-frame wood stud walls, were observed to possess some ductility (mainly by engaging the panel-to-framing connection), but the energy dissipation is still limited. These observations have necessitated the investigation of the behaviour of connections in isolation and as part of timber assemblies in order to establish a hierarchy of failure required to enhance their performance.

Studies on typical nailed connection details for light-frame wood walls in high-seismic regions meant to resist in-plane loads performed poorly when subjected to out-of-plane simulated blast loads [10]. A similar study was conducted for CLT panels detailed with self-tapping screws (STS) as end connections, where significant damage in tension perpendicular-to-grain in the CLT panel as well as shear and withdrawal failure in the STS were reported [19]. Intentionally overdesigned connections were shown to result in the wood elements reaching their flexural capacities but with no additional energy dissipation provided by the connections [20–22]. CLT panels with flexible bracket end connections [19,20] and glulam elements with bolted connections ,detailed to have a yield strength less than the connection load associated with flexural failure of the glulam elements [21,22], showed some improvements in system ductility and energy dissipation when compared to overdesigned connections. Using STS to reinforce bolted glulam joints against splitting failure was found to provide enhanced performance, thus ensuring that bolt yielding and wood crushing could occur [21,22].

Energy Absorbing Connections (EACs) can be designed to undergo substantial plastic deformation without significant change in strength following yield. The behaviour can be controlled through the development of plastic hinges in steel shapes of various geometries, such as angular (e.g., Figure 1a) or circular (e.g., Figure 1b) [23]. Research involving quasi-static and Shock Tube testing of timber-EAC assemblies [2,24] and individual EAC components [23] demonstrated that significant energy dissipation could be achieved, thereby allowing timber structures to withstand larger blast loads (i.e., greater pressures and impulses) prior to failure of the load-bearing elements [2,24].

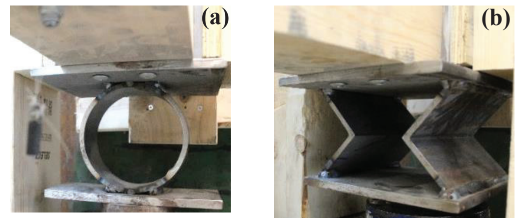


Figure 1: (a) Angular-shaped EAC, and (b) circular-shaped EAC

2.1 CURRENT DESIGN STANDARDS AND GUIDELINES

Blast design guidelines have been developed to enhance protection against accidental and intentional explosions (e.g., [1,25–27]). The design approach is based on obtaining the specified strengths from standards (e.g., CSA O86 [28]) and modifying them with a strength increase factor (SIF) and DIF. The SIF converts the design-level capacity from the near-5th percentile to the 50th percentile level, while the DIF increases the material property to account for high-strain rate effects. In timber elements, DIFs have been reported to vary between 1.1 and 1.4 [1]. A major shortcoming of the current Canadian blast design standard [1] is the lack of guidance and design provisions for timber connections. Currently, the default design method requires the connections to be capacity-protected at a level corresponding to 1.2 times the strength of the load-bearing timber elements. As mentioned earlier, this design philosophy does not promote optimization in the energy dissipation of the assembly and may be inadequate for timber structures. This paper investigates the implications of designing connections to be energy-dissipative or capacity-protected relative to the wood element.

3 – EXPERIMENTAL PROGRAMS

A series of experimental programs have been undertaken at the University of Ottawa Shock Tube Test Facility (Figure 2), which is capable of simulating the effect of far-field blast loads. The specimens ranged from individual connections to full-scale specimens such as columns or walls, with and without boundary connections. Table 1 summarizes the tested wood assemblies, connections, and observed failure sequence in each test campaign. Overall performance of the studied wood assemblies depended heavily on the connection performance and the design approach used. In all instances, when the connections were not properly detailed or detailed such that the ultimate capacity of the connection was lower than the load level associated with failure of the wood elements, premature failure of the assembly was observed, often in a brittle manner.

Table 1: Summary of experimental test campaigns

Wood element	Connections	Sequence of failure	Ref.
Light-frame stud walls	Nails	Nail withdrawal	[7,9,10]
	Joist hangers	Flexural Failure	[10]
Glulam	Bolt (overdesigned)	Flexural Failure	[21,22]
	Bolt (energy dissipative)	Connection yielding, followed by flexural failure	[21,22]
	EACs	Connection yielding, followed by flexural failure	[2,23,24]
CLT	STS	Connection rupturing/wood splitting	[19]
	Angle bracket (energy dissipative)	Connection yielding, followed by flexural failure	[19,20]
	Angle bracket (overdesigned)	Flexural failure	[19,20]
	EACs	Connection yielding, followed by flexural failure	[2]

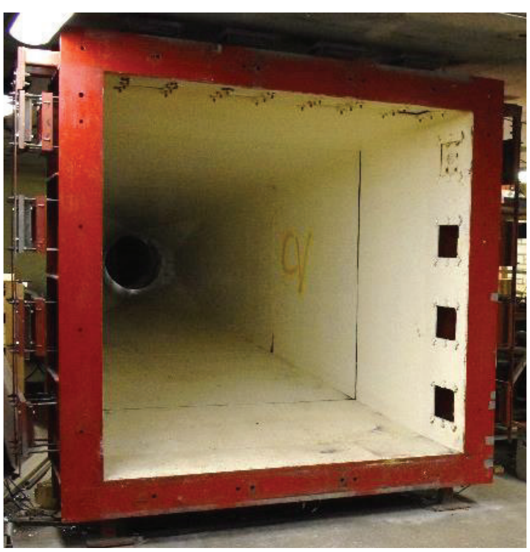


Figure 2: University of Ottawa Shock Tube Test Apparatus

4 – CONNECTION DESIGN APPROACH

The peak resistance (R_{peak}) of the wood element is reflective of average strength properties and accounts for high strain-rate effects, and for the purpose of connection design is assumed to be known. The resistance curve used to model the load-displacement response can typically be assumed linear-elastic (e.g., sawn lumber, glulam), bi-linear (e.g., light-frame wood stud walls), or staircase-shaped (e.g., CLT) [29]. Although the post-peak characteristics are needed when conducting dynamic analysis, they typically have little impact on the design of the connections.

The load-displacement curve of connections in timber assemblies can be idealized by a linear-elastic region followed by: 1) increasing resistance but with reduced stiffness; 2) a descending branch, or 3) a plastic plateau, as shown in Figure 3, where F_e corresponds to the 50[th] percentile (i.e., average) elastic limit (i.e., yield strength) of the connection. In the case of heavily overdesigned connections, the behaviour can be assumed to remain in the elastic regime.

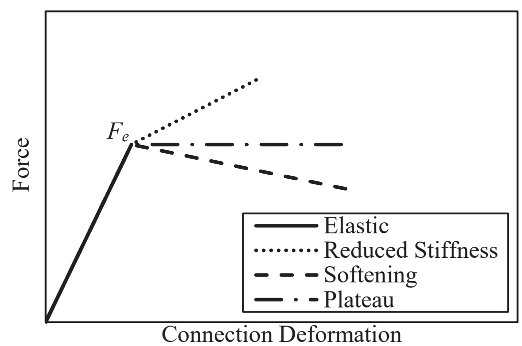


Figure 3: Idealized Connection Load-Displacement Curves

When connections are overdesigned, the value of F_e is required to be sufficiently high such that it is not surpassed before the timber element reaches its ultimate strength, R_{peak}, coinciding with the reaction load F_w (i.e., $0.5R_{peak}$):

$$F_e \geqslant \Omega_w F_w \tag{1}$$

Where, Ω_w is the overstrength factor applied to the average wood strength. For the connections to be overdesigned, the probability of connection yielding before wood failure, as described in (2), is required to be minimized to an acceptable level:

$$P(C - W \leq 0) \tag{2}$$

https://doi.org/10.52202/080513-0129

Where C and W are normally distributed random variables representing the density function of the connection elastic limit strength and the ultimate wood strength, respectively. The difference between C and W can be described by a normal distribution with the following mean and standard deviation:

$$\mu_D = \mu_e - \mu_w = \Omega_w \mu_w - \mu_w = \mu_w(\Omega_w - 1) \quad (3)$$

$$\sigma_D = \sqrt{(CoV_e \Omega_w \mu_w)^2 + (CoV_w \mu_w)^2} \quad (4)$$

Where CoV_w and CoV_e are the coefficients of variation for the peak resistance of the wood element and the elastic limit of the connection, respectively. Applying *Z*-score normalization with (3) and (4), and simplifying, one obtains:

$$Z = \frac{-(\Omega_w - 1)}{\sqrt{CoV_e^2 \Omega_w^2 + CoV_w^2}} \quad (5)$$

Through simplification of (5), one obtains:

$$\Omega_w = \frac{-2 - \sqrt{4 - 4(CoV_e^2 Z^2 - 1)(CoV_w^2 Z^2 - 1)}}{2(CoV_e^2 Z^2 - 1)} \quad (6)$$

Equation (6) provides a closed-form solution for an appropriate overstrength factor as a function of the chosen *Z* score (i.e., probability of failure), as well as variability of the connection and wood element. It must be noted that (6) is limited by $CoV_e < \sqrt{1/Z^2}$, representing a limit whereby a required Ω_w cannot be attained for a chosen *Z*. In such circumstances, a lower level of reliability should be adopted, or a connection with smaller variability should be selected. This design approach is illustrated in Figure 4a.

Alternatively, designing the connections to be energy-dissipative can be achieved by requiring F_e to be lesser than F_w, while taking into consideration the variability in yield strength and wood material failure through a reduction factor to allow for energy dissipation, Ω_e:

$$F_e \leqslant \Omega_e F_w \quad (7)$$

The probability of wood failure prior to yielding in the connections can be written as:

$$P(W - C \leq 0) \quad (8)$$

Using a similar methodology as that for Ω_w, and solving for Ω_e, one obtains:

$$\Omega_e = \frac{-2 + \sqrt{4 - 4(CoV_e^2 Z^2 - 1)(CoV_w^2 Z^2 - 1)}}{2(CoV_e^2 Z^2 - 1)} \quad (9)$$

Similar to (6), (9) is limited by $CoV_w < \sqrt{1/Z^2}$. In addition to this limit, (9) has another asymptote as CoV_e approaches $\sqrt{1/Z^2}$. However, a value for Ω_e can be obtained by evaluating the limit as CoV_e approaches $\sqrt{1/Z^2}$. Doing so, one gets:

$$\Omega_e = \frac{1 - Z^2\,(CoV_w^2)}{2} \quad (10)$$

The design approach for energy dissipative connection is outlined in Figure 4b. The proposed methodologies described herein require the designer to know the variability of the wood element and connections of interest. For the former, one may obtain near-5th percentile design values from the CSA O86 [28], which can then be brought to average values through the CSA S850 [1], which provides appropriate factors. In the case of the connection, the Canadian blast design standard currently does not provide the factors necessary to obtain average values. Alternatively, representative *CoV* values may be obtained from published literature or through experimental testing.

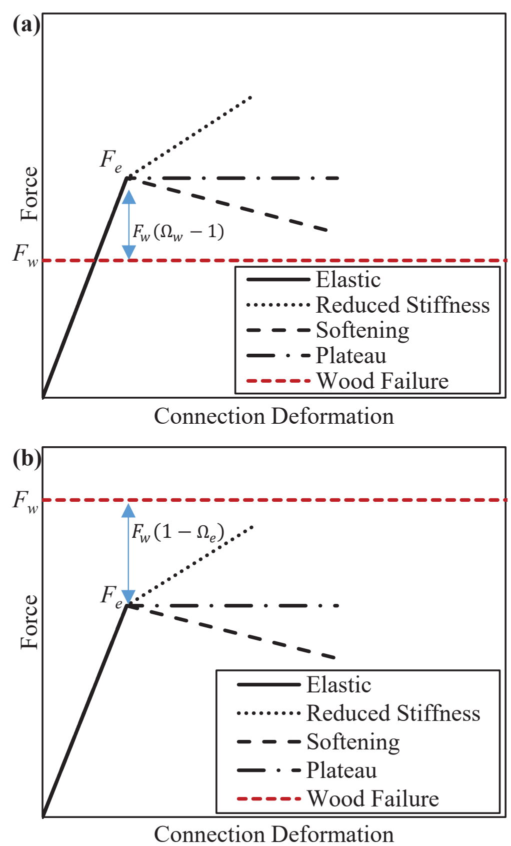


Figure 4: Design Approaches for: (a) Overdesigned Connections, (b) Energy-Dissipative Connections

https://doi.org/10.52202/080513-0129

It is important to emphasize that allowing the connection to yield and dissipate energy can only lead to the desired optimized design if it is ensured that the ultimate strength of the connection is greater than that of the load-bearing element. Failure to do so may lead to premature failure in the connection and an increased probability of disproportionate collapses, effectively jeopardizing the integrity of the structural system. For example, all the idealized connection load-displacement curves shown in Figure 4b would result in failure ultimately occurring in the connection. This can be remedied by selecting a connection with sufficient post yield strength and displacement capacity. For example, in the case of bolted connections, such as those investigated by Viau and Doudak [21], the behaviour followed that of increasing resistance but with reduced stiffness. As shown in Figure 5, this secondary stiffness, although degraded relative to the initial stiffness, allowed the joint to reach sufficiently high resistance and dissipate energy, while still resulting in ultimate flexural failure in the glulam element.

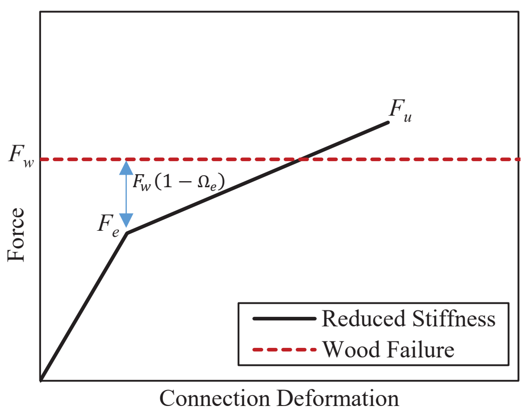


Figure 5: Idealized Load-Displacement Curve for Energy-Dissipative Bolted Connections

EACs, when compared to conventional bolted connections, offer designers more control over the response by more precisely defining the connection yield point, initial stiffness, and secondary stiffness [2,23,24]. One of the most important aspects of EACs is their ability to undergo densification at the end of their energy dissipation stage. Figure 6 shows the idealized load-displacement curves, where Ω_e is applied to the yield strength. When EACs exhibit a post-yield stiffness that is greater than zero, designers should be aware that attaining densification may no longer take place. However, the desired sequence of failure is maintained. In order to optimize such designs, the designer can select a densification level that is below the point at which wood failure occurs.

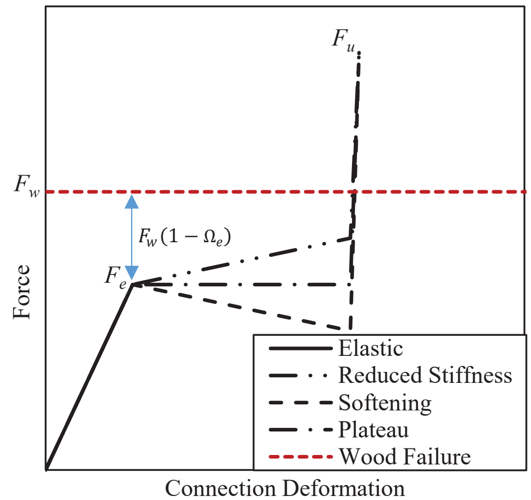


Figure 6: Idealized Load-Displacement Curve for EACs

5 – EVALUATION OF DESIGN CONCEPT USING PRESSURE-IMPULSE DIAGRAMS

The effects of the end connections on the overall performance of various wood assemblies can be quantified and visualized with pressure-impulse (PI) diagrams. This method describes the performance characteristics for a range of blast loading (i.e., pressure and impulse combinations) for specific types of elements and failure modes. Each curve indicates different damage levels and are referred to as iso-damage curves. The PI curves delineate the iso-damage regions, which allow for a qualitative assessment of the anticipated damage for a given pressure-impulse combination. For example, a pressure-impulse combination positioned to the left of a given iso-damage curve corresponding to "superficial damage" would signify that the element is still within the bounds of such a response limit. However, if the pressure-impulse combinations fall to the right of the curve (i.e., beyond the curve), then the component would have surpassed "superficial damage" and is now in the next iso-damage region. Single-degree-of-freedom (SDOF) analysis can produce defined element responses, which are often correlated with blast response limits.

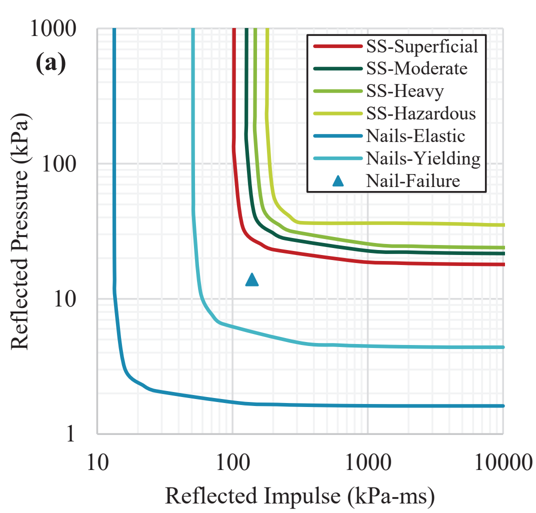
The impact and effects of the end boundary connections investigated in the aforementioned studies were assessed using PI-diagrams, including 2×6 light-frame wood stud walls, 137 mm × 267 mm glulam beams, and 5-ply CLT panels, since significant datasets are available for those elements in the literature (see Table 1). The assemblies were first idealized using two-degree-of-freedom

https://doi.org/10.52202/080513-0129

(TDOF) modelling based on published resistance curves for timber and connection elements. From the output of the TDOF analysis, a composite resistance curve representing the total response was generated and used for the purpose of developing iso-damage curves.

5.1 – LIGHT-FRAME WOOD STUD WALLS

The connections considered for the assessment of light-frame wood stud walls include nail connections, deemed adequate for in-plane loading in accordance with the NBCC [30], but inadequate for the purpose of out-of-plane blast loading. As shown in Figure 7a, the iso-damage curve related to yielding of the nails clearly demonstrates the low yield resistance, and as such, even when the wall was subjected to a relatively low magnitude of blast loading, it underwent inelastic deformation that was unstable and led to ultimate failure occurring in the boundary connections (see Figure 7b). This sequence of yielding and failure is clearly undesirable and is not in the spirit of the design philosophy proposed in this paper. In such a design scenario, requiring overdesigned connections would be preferred as it allows the wall to reach its full flexural capacity. This is demonstrated by the size of the region between the iso-damage curve related to nail yielding and that of expected superficial damage of the assembly when the boundary conditions are simply-supported (SS). Adequate performance has been observed when bracket connections are used on the individual studs (e.g., joist hangers, see Figure 7c) or entire wall in bearing (angle brackets) [10]. Such connections can provide supplemental energy dissipation but are primarily meant to promote failure in the load-bearing wall elements, which inherently have some ductility due to the panel-to-framing nailed connections.



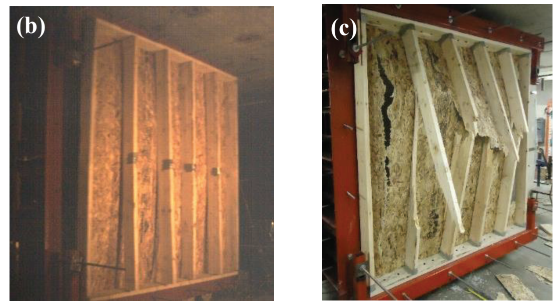
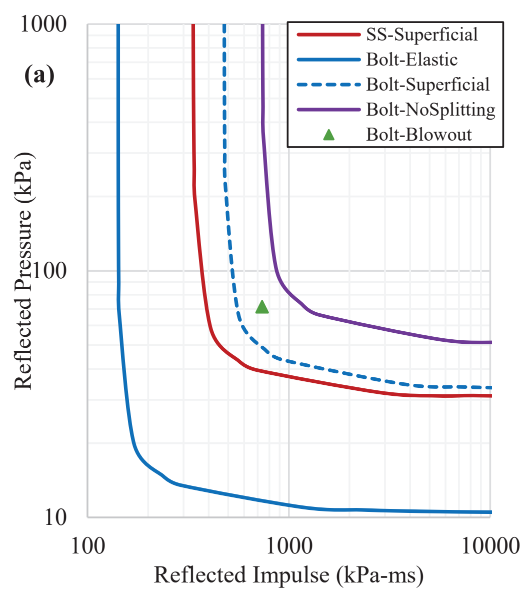


Figure 7: a) PI Diagram for Light-Frame Wood Stud Walls, b) Failure of Nailed Connections, and c) Overdesigned Connections

5.2 – GLULAM ELEMENTS

The approach for glulam flexural elements using typical dowel-type connections (e.g., bolts) has demonstrated that adequate design can be achieved when the connections are protected against brittle failure mechanisms, and allowed to absorb energy via yielding of the steel bolts and crushing of the wood fibres [21]. As shown in Figure 8a, the region between the iso-damage curve relating to flexural failure of the glulam beam with simply-supported boundary conditions and the iso-damage curve for the glulam beam with bolted connections illustrates the potential energy dissipation capacity that properly designed bolts can provide in this system (Figure 8b).



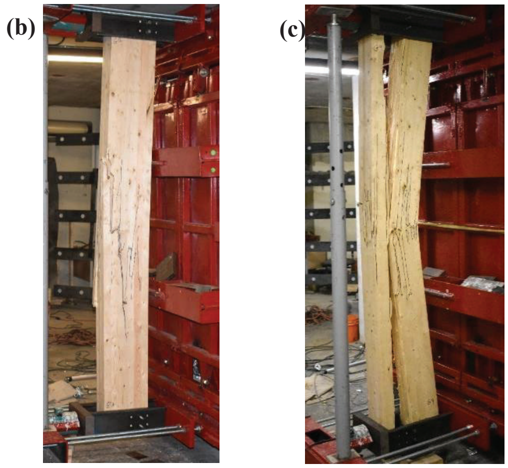


Figure 8: a) PI Diagram for Bolted Glulam Assemblies, b) Yielding of Bolted Connections, and c) Splitting Failure

Through proper detailing of the bolted connection, such as providing enough unloaded edge distance or using self-tapping screws to prevent or delay splitting, the hypothetical iso-damage curves relating to splitting (see Figure 8c) are located beyond that which pertains to flexural failure, signifying that such unwanted failure mode is guarded against. As shown in Figure 9 for the 2×1 bolted connections tested by Viau and Doudak [21], premature failure in the entire assembly can occur as a result of splitting failure in the connection if brittle failure modes are not properly guarded against (Figure 8b). This corresponds to a shift in the iso-damage curves related to splitting failure towards lower pressure-impulse combinations, proportional to the unloaded edge-distance, e_P.

The bolted connections may be overdesigned in lieu of designing the bolted connections to yield prior to the flexural failure of the glulam element. In doing so, the assembly would tend to behave similarly to the case of simply-supported boundary conditions, along with a diminution of the benefit and contribution of the bolted connections towards the overall energy dissipation. This is shown in Figure 10, where the iso-damage curves for 2×1, 2×2, and 2×3 bolted connections are compared. While stronger connections can withstand larger blast loads prior to inelastic deformations occurring within the connections, as shown by the iso-damage curves pertaining to the elastic limits of the bolted connections, the available energy dissipation is limited in the event that a significant blast load is applied.

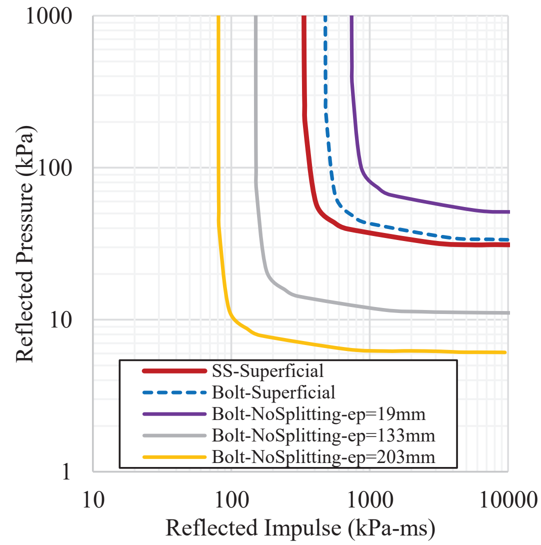


Figure 9: Effect of Bolted Connection Detailing

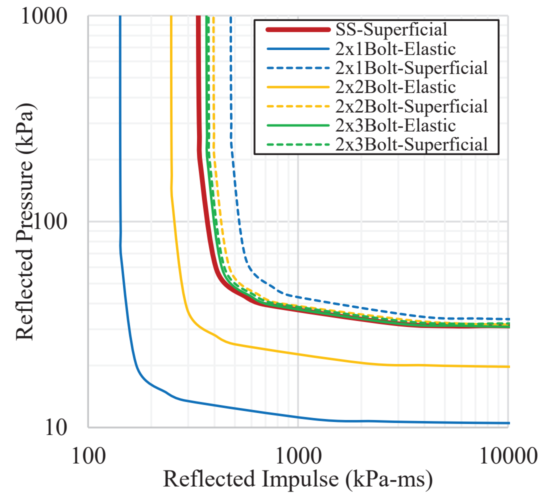


Figure 10: Effect of Bolted Connection Yield Strength

Figure 11a presents the iso-damage curve for assemblies with EACs and compares them to those produced using bolted connections. EACs allow for further optimization of the system by shifting the required P-I combination to engage the connection in yielding (Figure 11b), while ensuring that the ultimate failure still occurs in the glulam (Figure 11c). A significant enhancement of the energy dissipation is observed for assemblies with EACs as illustrated in the observed test results from Viau and Doudak [2], where the yielding of the EACs within the timber assembly, with no damage in the timber element (Figure 11b), is at higher P-I combinations than that involving blow-out damage for assemblies with bolts.

https://doi.org/10.52202/080513-0129

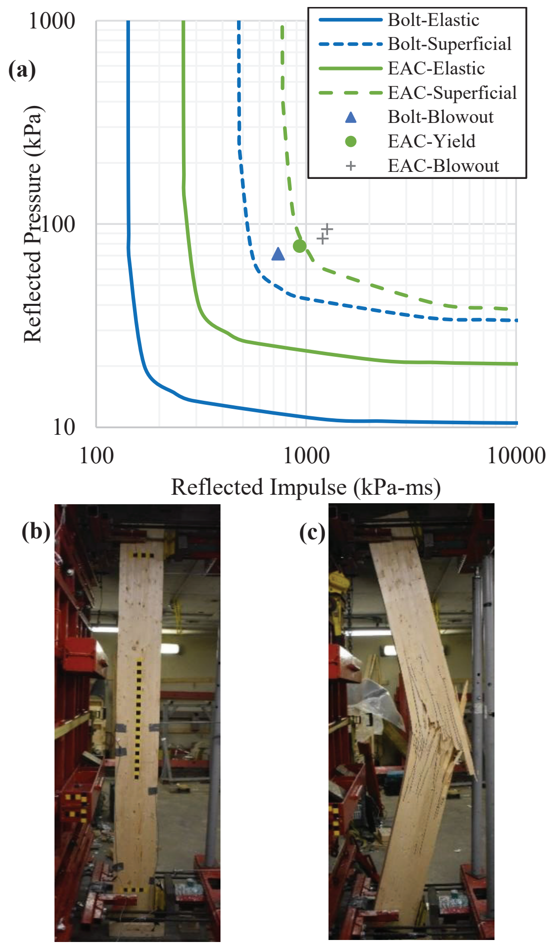


Figure 11: a) PI Diagram for Glulam with EACs, b) Glulam Beam Surviving Blast due to EAC c) Higher PI Causing Densification

5.3 – CLT PANELS

The two design philosophies involving overdesigned and optimized design of boundary connections were also investigated for CLT panels used as wall elements [2]. Figure 12 emphasizes the notion that when the angle brackets were overdesigned (i.e., thick angle bracket), the behaviour mimicked closely that of simply-supported boundary conditions. The green solid line represents the point at which yielding in the self-tapping screws used to fasten the bracket to the floor diaphragm started yielding. However, this did not allow for much energy dissipation, as the joint was very stiff. As such, the upwards shift of the iso-damage curve from using the overdesigned bracket is minimal, as shown in Figure 12. It was noted in the study conducted by Viau and Doudak [20] that even when using fewer screws, the connection did not allow for energy dissipation to be explored at magnitudes that would be meaningful, due to its large stiffness and due to the shape of the resistance curve.

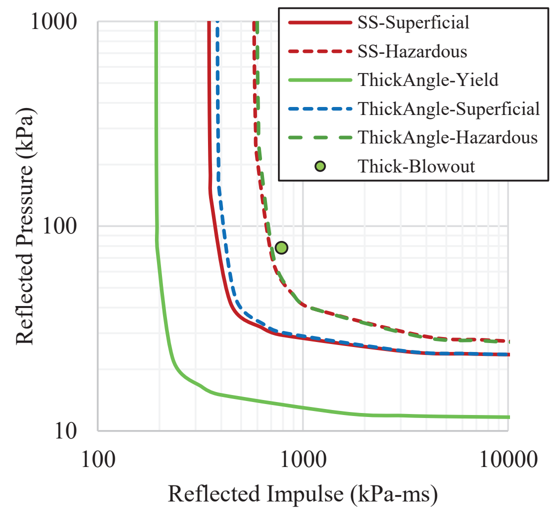


Figure 12: PI Diagram for CLT with Overdesigned Connections

As shown in Figure 13, similar observations to those made for glulam can be noted when EACs were implemented in the CLT assemblies. The early onset of yielding in the EACs and the relatively large region between EAC yielding and superficial damage demonstrate the potential for energy dissipation that can be attained from implementing EACs. Significantly higher P-I combinations are required to cause superficial damage in the assembly with EAC compared to those causing hazardous and blow-out damage in assemblies with the overdesigned connections using thick plate. As the P-I combination increases, the dependency on the energy dissipation in the CLT panel alone decreases, as observed from the decreasing iso-damage region between "superficial" and "hazardous" damage levels for the EACs, when compared to thick angle brackets.

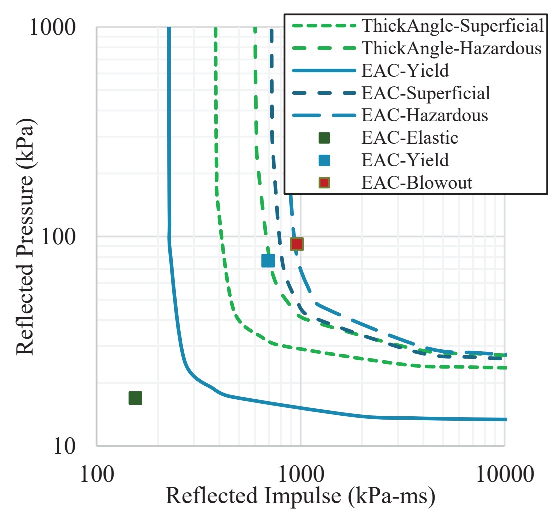


Figure 13: PI Diagrams for CLT with EACs

https://doi.org/10.52202/080513-0129

P-I diagrams, such as those generated in this paper, or more generalized and normalized iso-damage curves (e.g., scaled PI diagrams, see [31]) can potentially be used to determine the quantity of energy dissipation required in the connections to "shift" the line corresponding to the performance of the timber element under simply-supported boundary conditions beyond the targeted P-I combination. Once determined, designers can conduct TDOF analysis using idealized load-displacement curves of connections in order to determine appropriate connection type and detailing that would satisfy the required design objective. The performance of end connections must consider both the pressure and impulse demands on the assembly, and as such, P-I diagrams may be a suitable tool for the initial design of such connections. Once designed, further TDOF evaluations may be conducted to verify whether the targeted level of protection/damage was reached.

6 – CONCLUSIONS

This paper proposed a generalized capacity-based blast design methodology for timber connections, based on the experimental test results obtained from extensive experimental shock tube testing. The overarching aim of this methodology is to promote ductility in connections and a sequence of failure that seeks to minimize occupant harm during extreme load events. The implications of designing connections to be energy-dissipative or capacity-protected relative to the wood element when they are subjected to blast loading were discussed. The expected "shape" of the connection resistance curve, as well as the variability in the wood and connection, were shown to be critical design parameters to ensure proper connection performance. Pressure-impulse diagrams were used to evaluate and demonstrate the efficacy of the proposed design methods. Overall, connections designed to yield before the ultimate failure of the main timber elements (i.e., energy dissipative connections) were shown to allow the wood assemblies to withstand larger pressure-impulse combinations than identical assemblies with simply-supported boundary conditions or overdesigned connections. An important requirement is to ensure that the ultimate capacity of the connection exceeds that of the timber element, ensuring adequate energy dissipation while maintaining the overall integrity of the structural assembly.

7 – REFERENCES

[1] CSA, 2023, *CSA S850:23 Design and Assessment of Buildings Subjected to Blast Loads*, Canadian Standards Association (CSA) Group, Mississauga, ON.

[2] Viau, C., and Doudak, G., 2021, "Energy-Absorbing Connection for Heavy-Timber Assemblies Subjected to Blast Loads–Concept Development and Application," J. Struct. Eng., **147**(4), p. 04021027. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002975.

[3] Sukontasukkul, P., Lam, F., and Mindess, S., 2000, "Fracture of Parallel Strand Lumber (PSL) under Impact Loading," Mater. Struct., **33**(7), pp. 445–449. https://doi.org/10.1007/BF02480664.

[4] Jansson, B., 1992, "Impact Loading of Timber Beams." https://doi.org/10.14288/1.0050473.

[5] Spencer, R., 1978, "Rate of Loading Effect in Bending for Douglas-Fir Lumber," Banff, Alberta, Canada.

[6] Jacques, E., Lloyd, A., Braimah, A., Saatcioglu, M., Doudak, G., and Abdelalim, O., 2014, "Influence of High Strain-Rates on the Dynamic Flexural Material Properties of Spruce-Pine-Fir Wood Studs," Can. J. Civ. Eng., **41**(1), p. 56+.

[7] Viau, C., and Doudak, G., 2016, "Investigating the Behavior of Light-Frame Wood Stud Walls Subjected to Severe Blast Loading," J. Struct. Eng., **142**(12), p. 04016138. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001622.

[8] Lacroix, D. N., and Doudak, G., 2015, "Investigation of Dynamic Increase Factors in Light-Frame Wood Stud Walls Subjected to Out-of-Plane Blast Loading," J. Struct. Eng., **141**(6), p. 04014159. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001139.

[9] Lacroix, D. N., Doudak, G., and El-Domiaty, K., 2014, "Retrofit Options for Light-Frame Wood Stud Walls Subjected to Blast Loading," J. Struct. Eng., **140**(4), p. 04013104. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000885.

[10] Viau, C., and Doudak, G., 2016, "Investigating the Behaviour of Typical and Designed Wall-to-Floor Connections in Light-Frame Wood Stud Wall Structures Subjected to Blast Loading," Can. J. Civ. Eng., **43**(6), pp. 562–572. https://doi.org/10.1139/cjce-2015-0452.

[11] Lacroix, D. N., and Doudak, G., 2018, "Determining the Dynamic Increase Factor for Glued-Laminated Timber Beams," J. Struct. Eng.,

https://doi.org/10.52202/080513-0129

144(9), p. 04018160. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002146.

[12] Lacroix, D. N., and Doudak, G., 2018, "Effects of High Strain Rates on the Response of Glulam Beams and Columns," J. Struct. Eng., **144**(5), p. 04018029. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002020.

[13] Cao, A. S., Houen, M., and Frangi, A., 2024, "Impact Loading of Glued Laminated Timber Beams without Finger-Joints," Comput. Struct., **296**, p. 107278. https://doi.org/10.1016/j.compstruc.2024.107278.

[14] Oliveira, D., Viau, C., and Doudak, G., 2023, "Behaviour of Mass Timber Members Subjected to Consecutive Blast Loads," Int. J. Impact Eng., **173**, pp. 104454-. https://doi.org/10.1016/j.ijimpeng.2022.104454.

[15] Oliveira, D., Viau, C., and Doudak, G., 2023, "Modelling the Behaviour of Heavy and Mass Timber Members Subjected to Blast Loads," Eng. Struct., **291**, pp. 116397-. https://doi.org/10.1016/j.engstruct.2023.116397.

[16] Wight, N., Viau, C., and Heffernan, P., 2024, "Behaviour of Glued-Laminated Timber Beams under Impact Loading," Can. J. Civ. Eng., **51**(7), pp. 753–768. https://doi.org/10.1139/cjce-2023-0298.

[17] Wight, N., Viau, C., and Heffernan, P., 2025, "Cold Temperature Effects on the Impact Behaviour of Glued-Laminated Timber Beams," Int. J. Impact Eng., **195**, p. 105135. https://doi.org/10.1016/j.ijimpeng.2024.105135.

[18] Poulin, M., Viau, C., Lacroix, D. N., and Doudak, G., 2018, "Experimental and Analytical Investigation of Cross-Laminated Timber Panels Subjected to Out-of-Plane Blast Loads," J. Struct. Eng., **144**(2), p. 04017197. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001915.

[19] Côté, D., and Doudak, G., 2019, "Experimental Investigation of Cross-Laminated Timber Panels with Realistic Boundary Conditions Subjected to Simulated Blast Loads," Eng. Struct., **187**, pp. 444–456. https://doi.org/10.1016/j.engstruct.2019.02.009.

[20] Viau, C., and Doudak, G., 2019, "Behaviour and Modelling of Cross-Laminated Timber Panels with Boundary Connections Subjected to Blast Loads," Eng. Struct., **197**, p. 109404. https://doi.org/10.1016/j.engstruct.2019.109404.

[21] Viau, C., and Doudak, G., 2021, "Behavior and Modeling of Glulam Beams with Bolted Connections Subjected to Shock Tube–Simulated Blast Loads," J. Struct. Eng., **147**(1), p. 04020305. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002876.

[22] McGrath, A., and Doudak, G., 2021, "Investigating the Response of Bolted Timber Connections Subjected to Blast Loads," Eng. Struct., **236**, p. 112112. https://doi.org/10.1016/j.engstruct.2021.112112.

[23] Bérubé, A., and Doudak, G., 2023, "Investigation of Energy-Absorbing Connections for Application in Mass Timber Structures," Eng. Struct. https://doi.org/10.1016/j.engstruct.2023.117018.

[24] Bérubé, A., and Doudak, G., 2024, "Enhancing Energy Dissipation in Glued-Laminated Timber Assemblies Using Boundary Connections," Structures, **65**, p. 106744. https://doi.org/10.1016/j.istruc.2024.106744.

[25] Office of the Deputy Prime Minister, 2004, "Disproportionate Collapse, The Building Regulations 2000."

[26] UFC, 2008, *Structures to Resist the Effects of Accidental Explosions*, UFC 03-340-02, Unified Facilities Criteria Program, US of America Dept. of Defence, Washington, DC.

[27] ASCE/SEI, 2023, *Blast Protection of Buildings*, ASCE/SEI 59-22, American Society of Civil Engineers, Reston, Virginia.

[28] CSA, 2024, *CSA O86:24 Engineering Design in Wood*, Canadian Standards Association (CSA) Group, Mississauga, ON.

[29] Doudak, G., Viau, C., and Lacroix, D. N., 2022, "Proposed Design Methods for Timber Members Subjected to Blast Loads," J. Perform. Constr. Facil., **36**(3). https://doi.org/10.1061/(ASCE)CF.1943-5509.0001723.

[30] Canadian Commission on Building and Fire Codes, 2022, *National Building Code of Canada: 2020*, 978-0-660-37913–5, National Research Council of Canada. https://doi.org/10.4224/w324-hv93.

[31] Viau, C., Lacroix, D. N., and Doudak, G., 2017, "Damage Level Assessment of Response Limits in Light-Frame Wood Stud Walls Subjected to Blast Loading," Can. J. Civ. Eng., **44**(2), pp. 106–116. https://doi.org/10.1139/cjce-2015-0418.