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Structural Repair of Fire-Damaged Glulam Timber

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(Manuscript Author order: Chorlton, Gales)

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Abstract

Engineered timber is being used to create increasingly taller structures, with building codes adapting to reflect this drive. While these structures continue to become more common, there has been little investigation into the possibility of repairing fire-damaged timber structural members, leaving practitioners with little guidance in the event of a fire and insurance companies with little information to assess risk. The research herein attempted to repair fire-damaged timber members by removing damaged portions of the timber and replacing them with new timber panels secured with structural screws. Members were loaded in four-point bending, and the findings concluded that the members were able to regain a significant amount of stiffness compared to the control members and on average deflected 19% less than they did prior to repair. The repaired members were not able to reach full strength, however, failing at a load ranging from 49% to 66% of the failure load of the control members. This indicates the need to further examine possible alterations that may improve this repair procedure to the point where composite action is fully enabled and a larger portion of the original strength is recovered. A hypothetical cost analysis based on this repair procedure was provided herein, aiming to help direct several research gaps to be addressed in order to enable the repair of fire-damaged timber.

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Introduction and Background

Recently, there has been a worldwide trend to build increasingly larger and taller timber structures. Timber can be an appealing material choice for its environmental performance, with studies showing that using timber can significantly reduce carbon emissions (by 14-31%) compared to other materials (Oliver et al. 2014). In Canada, one of the first of these structures was known as the hybrid Brock Commons building, which opened in 2017 and stands at 18 stories (53 m) tall. The Brock Commons building was constructed with Glued Laminated Timber (Glulam) and Cross Laminated Timber (CLT), which was nearly entirely encapsulated in fire rated gypsum board.

Since then, changes to the National Building Code of Canada (2020) have been made to reflect the demand for taller timber structures, allowing for 12-storey timber buildings (provided that several requirements are met in these structures, such as requirements over minimum levels of encapsulation) (National Research Council of Canada 2020). Given that these tall timber structures will continue to become more common, there remains uncertainty over how these structures may be able to be repaired should they experience a fire. This is important to consider for both building owners, as well as building insurers. For owners, the ability of a structure to recover (repair) from a fire is beneficial for operational resilience. If a structure can be quickly repaired, business can resume quickly, versus lengthy recovery times which may cause the building and businesses to close for some time. As for building insurers, it is critical to have an understanding of the recovery potential of a timber structure after a fire, so that insurance premiums can be balanced with risk.

When timber is exposed to a fire, it begins to decay. Timber is generally considered to char at 300 °C in structural applications (Bartlett et al. 2019), forming a char layer. Below the char layer, there is another layer of timber that has been heated to the point where the water may have evaporated and the timber may have degraded, but has not yet fully charred. Simplified methods have been created to assess the capacity of timber members post-fire. Charred areas of the timber are assumed to retain no structural capacity. In addition, a depth of 7 mm (for fire durations of 20 mins and greater, and proportionally less for shorter exposures) is often considered to have lost its capacity as well, accounting for strength loss in timber beyond the char layer (CSA Group 2014). This is known as the zero-strength layer. The remaining portion of the timber member (beyond the char and zero-strength layers) is considered to have retained it’s full capacity, effectively creating a ‘reduced cross section’ member (CSA Group 2014). If the capacity provided by the reduced cross section member is insufficient, replacement or repair of the member may be needed. Currently, there is little guidance on how structural timber elements may be repaired post-fire and what the associated repair is and what this financial cost may be – particularly in Canada where the authors jurisdiction is and focus. To the authors’ knowledge, there is no formal guidance on how to repair a timber structural member specifically post-fire. The Canadian standard for engineering design in wood (CSA O86-14) does not directly address how to repair timber sections (regardless of the cause and severity of the damage). It does offer guidelines regarding creating built-up sections, in which the fastening guidelines using nails, bolts, or split-rings are described including minimum spacing requirements and minimum penetration depths (CSA Group 2014). The Institution of Structural Engineers in the UK also offers some suggestions on the repair of fire damaged timber, though repair strategies are broadly described as potentially removing the char and repairing the member using nails, bolts, screws, steel plates or glue, with no formal guidance on the detailing of these repairs (The Institution of Structural Engineers 2010). From the limited guidance available, practitioners would likely be required to use their engineering judgement in the design of a repaired timber member. As an aside, it may prove challenging to determine and utilise an appropriate procedure in Canada, as timber education in undergraduate civil engineering programs in Canada is often limited to one course, if any at most universities (Chorlton et al. 2019). While experienced practitioners may also be able to take on timber repair tasks, the number of available practitioners willing and able to take on such a design may be limited.

Other researchers have proposed different methods of repairing timber structural elements. In all of these cases, the timber elements are not specifically fire damaged. Alam et al. (2009) proposed using steel or Fibre Reinforced Polymers (FRPs) to repair timber elements. In those experiments, 1.8 m long timber beams were loaded to failure using four-point loading and repaired along the tension or compressive face of the beam (as needed) using steel or pultruded FRPs and epoxy adhesives (Alam et al. 2009). Alam et al. (2009) found that the repairs provided variable recovery of stiffness, though several repair configurations were effective in restoring the flexural strength of the beams to their original capacity. Morales-Conde et al. (2015) aimed to present a procedure for repairing or reinforcing timber beams using fibreglass and cork plates (Morales-Conde et al. 2015). Two types of damaged beams were examined, the first being beams whose ends had rotted, and the second being beams with faults in the centre of the beam. The beams had grooves (towards either the side of the beams or the centre depending on the damage) into which the GFRP plates were bonded. The GFRP plates consisted of a cork backing material with two layers of fibreglass fabric. The beams were tested in four-point bending and shear. Morales-Conde et al. (2015) concluded that the repairs of both the shear and moment beams were effective at regaining the beams' original strength (Morales-Conde et al. 2015). Procedures using GFRPs need to also consider the vulnerabilities to fire, often requiring additional fire protection measures which can increase cost should they be applied in a structure (Correia et al. 2015).

Ferreira et al. (2017) also proposed a method for repairing timber beams, this time using timber only as the repair material. Four sets of beams were examined; the first with no existing delamination, and the other three with pre-existing delamination in various regions along their length. All beams were tested in four-point loading and then repaired. The repair consisted of plywood sheets applied to the top and the bottom of the beam, attached using screws. Ferreira et al. (2017) also examined the stiffness before and after the repair. The beams with no pre-existing delamination recovered 79% of their initial strength and 81% of their initial stiffness (Gomes Ferreira et al. 2017). The beams with pre-existing delamination recovered from anywhere between 86%-249% of their initial strength, and 68%-121% of their initial stiffness (as the repairs compensated for the damage from the previous failure as well as the initial delamination) (Gomes Ferreira et al. 2017).

While the authors of these previous publications found their methods of repair to be effective to some extent at least, none considered fire-damaged members. Fire-damaged timber is different from timber with mechanical damage or minor defects, in that, for a fire-damaged member, a considerable area of the cross section is lost due to char and adhesive degradation effects (Frangi et al. 2009; Klippel et al. 2011; Lineham et al. 2016; Quiquero et al. 2018). Thus, even following the methods reviewed above may not be sufficient for the repair of a fire damaged timber. Franke et al. (2015) reviewed state-of-the-art methods for repairing and reinforcing timber beams (Franke et al. 2015). Franke et al. (2015) recommended replacing the fire damaged member, potentially by propping the beams and removing the damaged section of the beam, which is then replaced by a timber prosthesis connected by rods or plates (Franke et al. 2015). This method may be beneficial in many cases; however, it may be costly to perform this repair for timber sections with only minor fire damage. Further, no strict guidance is provided on the requirements of connecting the remaining portion of the timber to the new timber. To the authors' knowledge, there are little to no publicly available documents providing insight into the potential cost of the repair methods proposed above, or other repairs.

Flexure members constructed of multiple materials (or multiple laminates) have the potential to achieve composite action. For instance, solid beams will be able to achieve a certain stiffness and strength. If a beam of the same dimensions and material is sliced horizontally and not adequately fastened together, their strength will be the sum of the individual pieces, which is less than what the capacity would have been of a solid beam with the same cumulative dimensions. This is because design procedures for moment resistance consider the square of the beam height (CSA Group 2014). If the connection is designed adequately to achieve composite action, the resultant capacity can be equal (or greater than) what would be expected from a solid beam of the same dimensions. Repair of timber beams in which a portion of the cross section is removed and replaced with an additional laminate of timber is essentially attempting to take advantage of composite action. Thus, if the repair and the connection are sufficient, it may be possible to restore the member to the serviceability and strength state of the original design.

The research herein examines the potential to recover fire damaged engineered timber members (Glulam) to their original serviceability and strength state. While we address the Canadian context, results may be more universally applied. The results of these experiments are subsequently considered in the analysis of a hypothetical repair of a fire damaged timber building case study to introduce the discussion of repair cost and more importantly future research needs for the reader. The following sections of this manuscript examines the performance of four repaired Glulam members that had been previously fire damaged. The members were repaired by carving out the fire damaged region (affected by char) and replacing the lost cross section with additional timber laminates of the same species, secured with structural screws. All members were loaded and unloaded three times in four-point bending (to a load below the ultimate capacity) to examine stiffness recovery of the repaired members. The Glulam members were then loaded to failure to examine recovery of strength of the repaired members relative to the undamaged control members. This manuscript concludes by recommending additional research needs in the development of repair strategies for fire damaged engineered timber members. This study is, to the authors' knowledge, the first of its kind to explicitly address the repair of timber damaged by fire. This paper therefore serves as a preliminary examination into the repair of fire-damaged timber members, where the experimental program and cost analysis allow for the identification of knowledge gaps that should be addressed through future research to enable the repair of fire-damaged timber structures.

Experimental Methodology

The experimental research herein considers six Douglas Fir Glulam members of 16c-E stress grade (fabricated in accordance with CSA O122) (CSA Group 2016). The flexural strength of the beams was calculated from the four-point bending test to be 59 MPa, and the Modulus of Elasticity was calculated as 13 900 MPa (unfactored and determined from beam deflection, and section dimensions). This is above the Modulus of Elasticity required by CSA O122 for the 16c-E stress grade of 11 000 MPa (CSA Group 2016). The density of the Glulam was measured as 605 kg/m^3 ± 8 kg/m^3 and the moisture content was measured as 10% ± 0.1%. These members are summarized in Table 1. The members are of two different sizes, with Members denoted 1, 3 and 5 being 175 x 190 mm, and Members denoted 2, 4 and 6 being 175 x 228 mm. All members were 2532 mm in length (as delivered). The adhesive used in the Glulam members is a melamine-formaldehyde resin.

Table 1. Summary of Glulam members and previous fire damage

Member Number	Size (mm)	Damage State	Maximum char depth (mm)
1	175 x 190	None (control)	0
2	175 x 228	None (control)	0
3	175 x 190	Fire damaged with encapsulation	15
4	175 x 228	Fire damaged with encapsulation	27
5	175 x 190	Fire damaged without encapsulation	18
6	175 x 228	Fire damaged without encapsulation	23

Fire Damage

The non-standard fire exposure used to damage the Glulam members is described elsewhere in literature by the authors, in a study that examined the performance of fire rated Type X gypsum board applied to Glulam elements in lab and field scales (Chorlton et al. 2020). The test procedure described in the aforementioned publication created a damaged state on the lab scale timber elements, and these are the timber elements that are used in this paper's study. To summarize the damage created for the purposes of this paper: four Glulam members were subjected to an approximately 30-minute methanol pool fire, utilizing 14.3 L of fuel in a 0.48 m (width) by 0.6 m (length) pan. Two of the Glulam members were protected using one-layer of fire rated Type X gypsum board, and the other two were exposed to the fire without fire protection. The members cooled for 30 minutes after the pool fire. The members self-extinguished (no external flaming) at the end of the heating period but continued to smoulder for the duration of the cooling cycle. At the end of the 30-minute cooling period, light amounts of water were applied to stop any remaining smouldering. Two additional control (undamaged) members will be considered herein, with the members summarized in Table 1. While Members denoted 3 and 4 were protected using one layer of encapsulation during the fire, from Table 1 it is seen that they still experienced considerable charring (especially Member 4, which had the greatest char depth even relative to the unencapsulated members). The high char depth of Member 4 is attributed to a smouldering fire, during the cooling phase of testing, under the gypsum board on the side away from the fire. The mechanisms allowing for this, as well as the full details of the fire damage created by the fire testing phase of the study are described elsewhere (Chorlton et al. 2020).

Repair (Carving and Rebuilding)

As discussed in the introduction, there is little guidance available regarding procedures for repairing damaged timber members particularly in the Canadian jurisdiction. The procedure discussed herein is an adaptation of the available guidance (the procedure set out for built out compression members as per CSA O86-14 (CSA Group 2014)), modified where necessary to meet the specific requirements of the test setup. The general procedure is as follows. First, charred and pyrolyzed wood was removed mechanically. Then, panels of the same wood species were attached to the main member such that the panel secured to the bottom of the member spanned the width of the cross-section prior to damage (175 mm). Panels were also fastened to the sides of the member such that the original (undamaged) cross-sectional dimensions were restored by the fastened panels. This procedure was determined in hopes of achieving sufficient composite action to regain at least a portion of the pre-damaged strength and stiffness.

The members were loaded in bending. To the authors' awareness, CSA O86-14 offers no strict guidance on fastening requirements for built-up bending members. Structural screws were selected as fasteners, as previous studies by other authors have found them to be effective in repairing delaminated Glulam (Gomes Ferreira et al. 2017). Other fasteners were considered as well. CSA O86-14 lists requirements for using nails, bolts and split rings to create built-up compression members, however, due to the sizing of the main member and the repair panels, the use of these fasteners becomes more difficult. Due to the final width of the section, driving nails to meet the minimum penetration depth becomes difficult, as does creating bolt holes. Further, adhesives could be used, though the use of adhesive requires stricter quality control (Gomes Ferreira et al. 2017). Using adhesives also raised concerns regarding toxicity for on site application procedures. The repair procedure was meant to be a procedure that could easily be done on site, and the use of adhesives may require extra safety precautions and additional expense. Therefore, structural screws were selected as the fastener for this introductory research.

CSA O86-14 guidance for built-up compression members using mechanical fasteners (nails and bolts) was followed. In particular, requirements stating that the spacing of the fasteners parallel to the grain would be less than six times the thickness of the thinnest piece, spacing of fasteners perpendicular to the grain would be less than 10 times the fastener diameter, the fasteners would penetrate at least 3/4 of the thickness of the last piece, and there will be at least two rows of fasteners across the member width (due to the thickness of the member and panels) (CSA Group 2014). Therefore, to meet the required penetration length, 5/16" (8 mm) diameter, 6" (152 mm) length screws were used to secure the majority of the panels. The only exception to this was the bottom panels on Members 4 and 6. Due to the height of the member, longer screws were needed to meet the 3/4 penetration requirement, and therefore 3/4" (19 mm) diameter, 8" (203 mm) length screws were used. Due to the limited guidance available, many of the decisions regarding the fastening procedure relied on the authors' judgement for meeting as many of the requirements outlined in CSA O86-14 as possible, while at the same time maintaining a process that could efficiently be implemented in real, fire damaged structures.

The char depth of each of the fire damaged members was measured, with the maximum char depths recorded in Table 1. Any material that was damaged by the fire was removed (the charred layer as well as a pyrolysis layer), assessed visually by colour with the authors' interpretation of the char and pyrolysis layers seen in Fig. 1. Multiple researchers were used to concur on colour interpretation but in all cases definition of char depth was subjective but consistent between specimens.

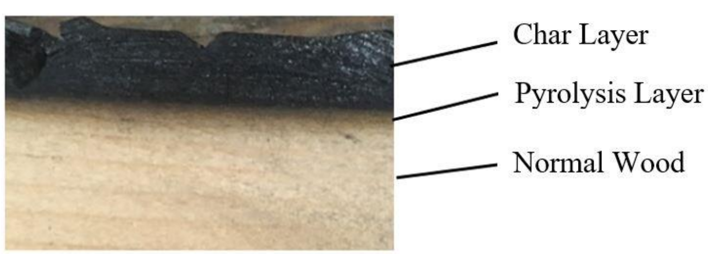

Fig. 1. Char formation and identification of damaged wood on a piece of timber.

The char depth was removed along the bottom of the members, and the two adjacent sides. This char depth was not removed along the top of the members, as char was not observed along the top of these elements. The damaged length of the member was determined by measuring the maximum length of char at any location along the member and considering that entire region to be damaged along all three sides. This, in theory, removed a greater area of the cross section than was necessary, however removing less than this would create an uneven length removed across the three sides, making the repair more complicated and less visually appealing. The carved (and eventually repaired) regions for each side of the damaged members are seen in Table 2.

Table 2. Carved and repaired area for each member

Member Number	Depth x length of area carved and repaired along bottom (mm)	Depth x length of area carved and repaired along side (mm)
1	0	0
2	0	0
3	15 x 504	15 x 504
4	27 x 582	27 x 582
5	12 x 1128	18 x 1128
6	23 x 1110	23 x 1110

The carving procedure consisted of using a saw to create guides of a specified depth along the damaged region of the members. The majority of the damaged material was then removed with a chisel. A belt sander was then used to create a smooth and uniform surface, precisely to the desired depth. A carved member before repair is seen in Fig. 2 (where the sides of the member are parallel to the floor, and the bottom is towards the camera).

Fig. 2. Carved member before repair.

The members were repaired using new laminates of timber of the same species and equivalent grade (per NLGA Standard Grading Rules for Canadian Lumber (National Lumber Grades Authority 2017)). The panels were cut to the appropriate size and planed to the appropriate depth, such that they would fit in the areas that were previously carved out. The panel along the bottom of the member spanned the entire width of the member (175 mm), while the panels along the sides of the members were in line with the top of the member, and the top of the bottom panel. This configuration was chosen to maximize the potential to recover strength from the contributions of the bottom panel, where the tensile stresses would be highest. A side panel is seen in Fig. 3.

Fig. 3. Side of a repaired member.

Mechanical Loading

All six members were loaded in four-point bending at a rate of 2.5 mm/min (following a modified ASTM D143 procedure for small clear specimens of timber) (ASTM International 2014). The test setup is seen in Fig. 4. Each member was loaded six times, three after carving but before repair, and three after repair. Once each member was carved, prior to being repaired, it was loaded and unloaded three times to a load much lower than it’s ultimate capacity. After repair, the member was again loaded and unloaded three times to the same load, lower than its ultimate capacity. The six loading and unloading cycles were intended to help look at the recovery of stiffness at a serviceability limit state. Initially, the applied load for these loading cycles was meant to correspond with the theoretical load that would bring the members to their deflection limit, however, these loads were found to be extremely small to the point where differences in deflection between the control members and the carved members (prior to repair) were minimal. Thus, it was decided to increase the load to 45 kN to see a greater difference in displacement, and therefore more potential to recover member stiffness through repair. The 45 kN load was determined experimentally, by loading the members until the load displacement curves began to show a very slight plateau indicating the introduction of plastic deformations(the plateau was defined as the point where the load was being applied at 80% of the rate prior to plateauing). As the intent was not to damage the members, 45 kN was taken as a safe value much below the ultimate capacity of the members (this was eventually found to be at least less than half of the strength capacity of every member). After the loading and unloading cycles were complete, each member was then loaded until failure, and therefore each member was loaded seven times in total.

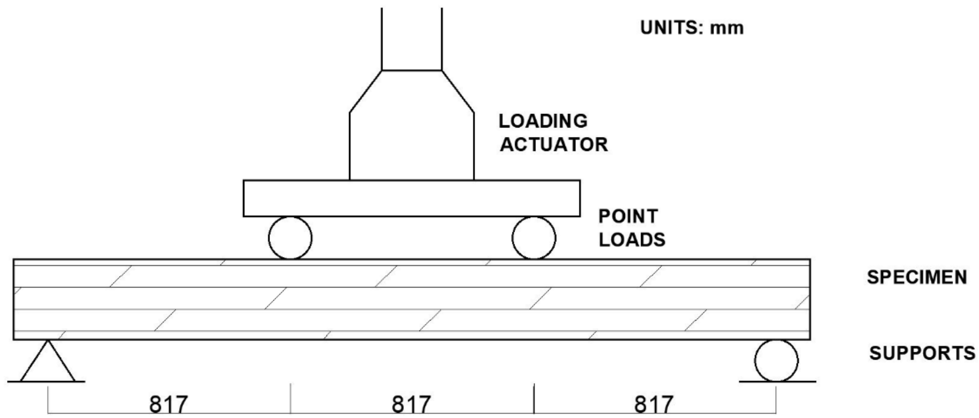

Fig. 4. Mechanical loading test setup.

Digital image correlation was used to analyze the displacements of the timber members, a technology that has proven to be accurate for measuring displacement in wood specimens (Quiquero and Gales 2016). A Canon EOS 5Ds camera was used to take images at 5 second intervals (Fig. 4). Members were speckled using white and black paint to create high contrast patterns for the GeoPIV RG software to track and analyze the test images (Stanier et al. 2016).

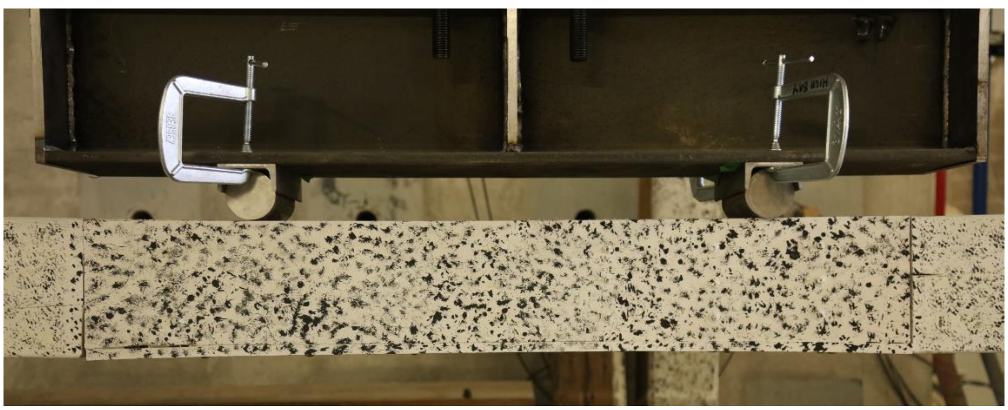

Fig. 5. Photograph that could be used in digital image correlation analysis, showing the test setup and speckled pattern paint.

Experimental Results

The results of loading the members to 45 kN are seen in Fig. 6 and 7. Note that in Fig. 6 and 7, each data set represents the average of the three trials, where shaded areas represent standard deviation across trials. It should also be noted that in Fig. 6 and 7, the displacement determined is from digital image correlation, and the force is the total force from both point loads. Fig. 6 shows that the members which were originally smaller in cross sectional area (Members 1, 3 and 5) saw a significant recovery of stiffness. Member 5, which was previously directly exposed to the fire and had more extensive damage (and therefore a greater repaired area), deflected 5.1 mm less once repaired (versus when carved), bringing the total repaired deflection to only 1.4 mm more than the control member. Member 3 which had a smaller repaired area had 1.2 mm less deflection than it did prior to repair, Because the initial degree of damage of the carved member had a deflection not significantly greater than the control member, the repaired member had less deflection than the initial control member.

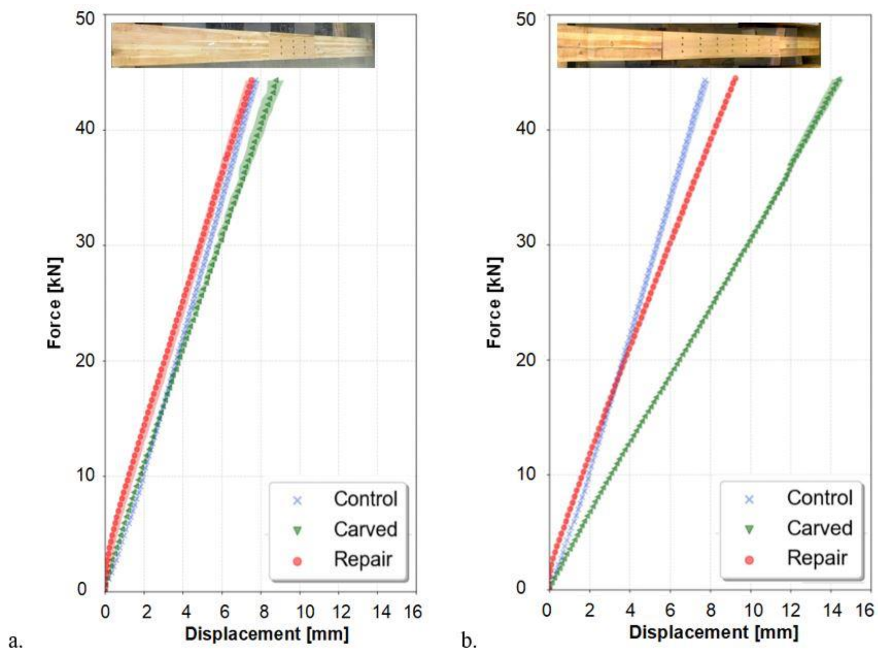

Fig. 6. Loading and unloading of the members of175 × 190 mm cross section: (a) Members 1 (control) and 3 (previously encapsulated); and (b) Members 1 and 5 (previously exposed).

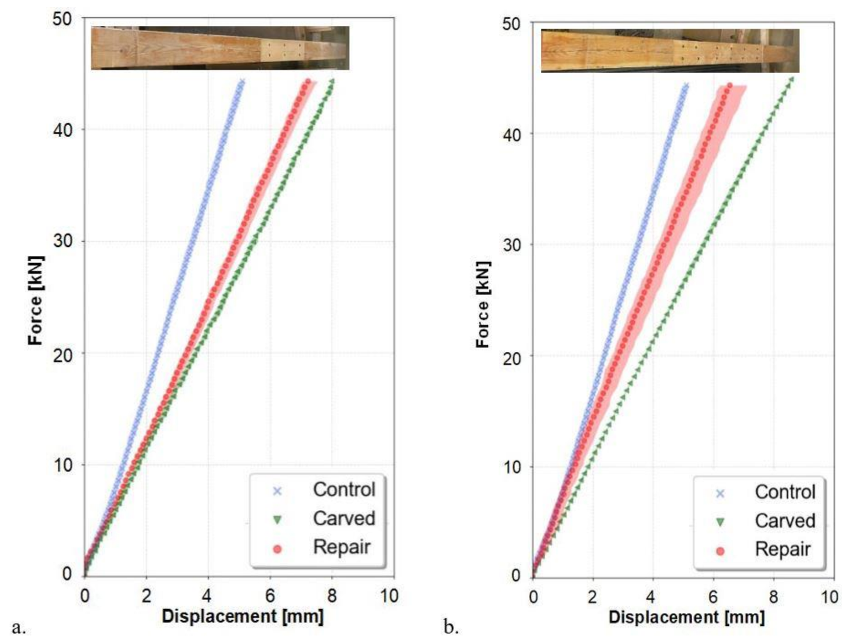

Fig. 7. Loading and unloading of the 175 × 228 mm cross section members: (a) Members 2 (control) and 4 (previously encapsulated); and (b) Members 2 and 6 (previously exposed).

For both Members 4 and 6, the maximum deflections of the repaired members have been reduced compared to those of the carved members. Member 4 saw a maximum deflection of the repaired member 0.7 mm less than that of the carved member, and Member 6 saw a maximum deflection of the repaired member 2.7 mm less than the maximum deflection of the carved member (about 30% of the total deflection of the carved member). In the cases of both these members, however, the repaired members were not able to return completely to the deflection observed by the control members.

Fig. 8 depicts the force displacement curve of each member loaded to failure (where again, displacement is from digital image correlation and force is the total force from both point loads). This figure shows that the control members were able to achieve a higher force than the repaired members, with all repaired members holding only 49 - 66% of the force that the control members were able to withstand. Members were not loaded prior to carving, however existing work examines the flexural performance of laminated timber members at high temperatures (Klippel and Frangi 2017; Quiquero et al. 2018; Schmid et al. 2014; Wiesner et al. 2020).

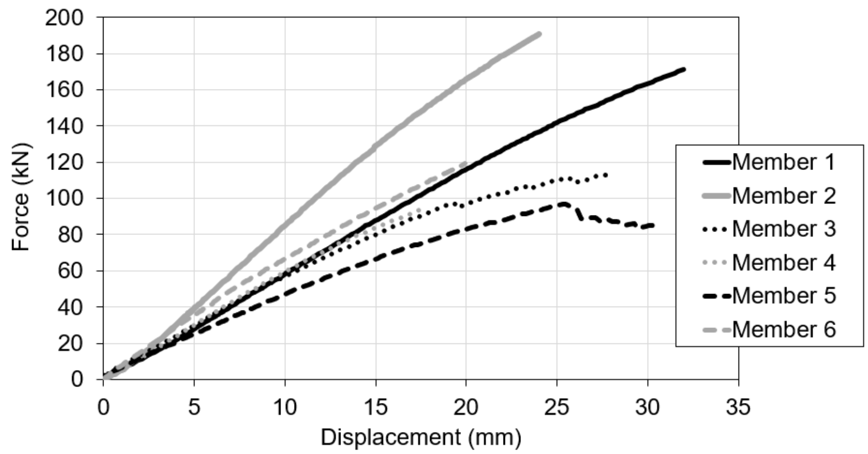

Fig. 8. Each member loaded to failure.

Discussion

While none of the repaired members were able to achieve the strength capacity of the control members, all members were able to achieve well over the strength calculated by the CSA O86-14 procedure for Glulam bending members. Calculated and actual failure loads for all members (after repair) are seen in Table 3. All members failed at a load of at least twice greater than predicted by CSA O86-14. Thus, even the repaired members that did not achieve the strength of the control members still held a considerable amount of strength relative to the predicted strength of CSA O86-14.

One aspect that must be considered when analyzing results is the grade of the members, which was 16c-E. Typically this grade is used for compression members, however, in this instance the members were tested in bending. The grade of the members will therefore affect the overall performance, and members whose grades are meant to be loaded in bending may perform better in this test setup. Members were loaded in bending as opposed to in compression, however, as through the bending test setup information about elasticity could be deduced as opposed to information solely about axial performance. Moreover, lateral loads can be present in real structures which will induce bending in the columns.

Table 3. Calculated and experimental failure loads

Member	Calculated Failure Load (kN)	Experimental Failure Load (kN)
1	31	171
2	45	191
3	31	113
4	45	93
5	31	96
6	45	119

Members 3 and 5

From Fig. 6 and 7, it is seen that in all cases, the repaired members were able to recover some stiffness compared to the carved members, although to varying degrees. The repairs of the members which were smaller in cross section (Members 3 and 5) both resulted in significantly less deflection compared to the carved members. Member 3 was able to regain sufficient stiffness such that the deflection of the repaired member was less than the deflection of the carved, and even control member. Maximum deflections of all members are seen in Table 4. The relatively strong performance of Member 3 may be due in part to the notion that the carved member did not deflect significantly more than the control member. When the repair was performed, the increased stiffness was sufficient in enabling the member to deflect even less than the control member. Moreover, Member 3 was the repaired member that held the greatest percentage of the failure load of the control member of the corresponding size (66%). Thus, while the member regained a promising amount of stiffness when loaded to 45 kN, it still failed at a lower force than the control member.

Member 5 also experienced significantly less deflection (35%) when repaired versus at its carved state. This large recovery in stiffness could be due to the relatively lengthy area of repair, as 1128 mm was carved away and repaired along three sides of the member. The repair therefore comprised of a significant amount of material, allowing for a significant regain of stiffness.

Table 4. Maximum deflections of members after carved (before repair), and after repair when loaded to 45 kN.

Member	Maximum Deflection Carved, before Repair (mm)	Maximum Deflection after Repair (mm)	Difference between Carved and Repaired (mm)	Difference between Carved and Control (mm)[a]	Difference between Repaired and Control (mm)[a]
3	8.9	7.7	1.2	0.8	-0.4[b]
4	8.0	7.3	0.7	2.9	2.2
5	14.5	9.4	5.1	6.4	1.3
6	8.7	7.0	1.7	3.6	1.9

a- Members 3 and 5 with respect to control Member 1 (deflection 8.1 mm), and Members 4 and 6 with respect to control Member 2 (deflection 5.1).
b- Repaired Member had less deflection than control.

The failure load of Member 5 was 57% of the control failure load. This is less than the capacity of the other repaired member of the same size (Member 3 that failed at 66% of the control load). One factor that could have played into the low failure load of Member 5 (relative to Member 3) was cracks that were present along the side of the member (primarily along the adhesive lines). These are seen in Fig. 9 and were present after the fire exposure of the member (before any mechanical loading). While this crack or delamination may have a different effect on the structural performance of the member if loaded in compression (as is typically done for Glulam members of 16c-E stress grade, where cracking can impact the moment of inertia reducing buckling capacity of some columns), it will also impact the member tested in bending. The presence of these cracks may have reduced the capacity of the member and made it more difficult to achieve composite action. The effect of cracks and delamination in Glulam members is also something that needs to be considered with regards to repair. Cracks occur in many timber structures, both historic structures as well as structures that were constructed relatively recently. Cracking and delamination in the column of a contemporary timber structure in Toronto, Canada is seen in Fig. 10, where the structure was constructed in 2015, with the photo taken four years later. The column seen in Fig. 10 shows both delamination along the adhesive lines as well as within the timber itself. The presence of cracks and delamination in timber members complicates both the structural and fire performances of the member, as well as the repair procedure. The effect of cracks and delamination in timber should be addressed in future research, with regards to the structural fire performance as well as how they may impact the repair.

Fig. 9. Side of Member 5.

Fig. 10. Timber column in a contemporary structure.

Members 4 and 6

As for the members that were larger in cross section, Members 4 and 6, both were able to regain at least a portion of their stiffness. Both these members had a similar char depth (27 mm along the bottom of Member 4, and 23 mm along the bottom of Member 6), though Member 6 had a more extensive damaged length (1110 mm) than Member 4 (582 mm), where the large repaired area could account for variations in performance (Member 6 deflecting less, and sustaining more load than Member 4).

One aspect which could have impeded on the performance of Member 4 (relative to Member 6) is that while the other members self-extinguished soon after the fuel of the pool fire was consumed, a smouldering fire occurred underneath the gypsum board layer of Member 4. The procedure of the fire exposure included a thirty-minute heating period followed by a thirty-minute cooling period, at which time the fuel had been consumed but the member continued to be observed without intervention. After thirty minutes, the gypsum boards were removed, and light amounts of water were used to extinguish any smouldering. Therefore, although the visible damage was therefore comparable to that of the other members, at least one side of the member was exposed to high temperatures well past the heating period due to the smouldering fire. This additional heating of the member could have caused increased material degradation beyond the char front, such as degradation due to the heating of the adhesives. It is therefore possible that

Member 4 may have had additional degradation relative to the other members, resulting in the lower recovery of stiffness and strength.

As seen in Fig. 7b, the repaired trials of Member 6 had the greatest amount of standard deviation. This occurred because in the first of the three trials, the member deflected almost 1 mm more than the second and third trials (which were near identical) and returned to an unloaded position of 1 mm below the initial position. Because the member returned to a position slightly below it's original position, this suggests that during the first loading cycle, the member adjusted slightly in the loading apparatus causing it to move downward by about 1 mm. If this is the case, then the deviation of Member 6 would be greatly reduced once this slight adjustment is considered.

Improvements to the Repair Procedure

The members tested in this study were loaded in bending after fire exposure. This procedure varies from what would occur in a real structure, where the members would be loaded during fire exposure. When subject to changes in temperature and moisture content, loaded timber members have been found to be affected by creep (Armstrong and Kingston 1960; Wiesner et al. 2020), an effect that would not be observed in members that are heated and then loaded afterwards. This difference is important to note in interpreting the results of this study.

While the repair did somewhat improve the performance of the members, further improvements could be made to the repair procedure in order to fully attain composite action. It is possible that the thermomechanical degradation of the timber contributed to the larger deflections of the repaired members relative to control, however previous studies that have looked at fire-damaged timber beams have not seen as significant of reduction in failure load or increase in deflection in damaged members, relative to control members (Quiquero et al. 2018). The differences in performance between the repaired and control members is therefore largely attributed to the adequacy of the repair procedure. In these tests, the area removed corresponded to the maximum damaged length and depth, however in order to fully achieve composite action, more area may need to be removed. In particular, it may be beneficial to repair a greater length than is charred in order to better achieve composite action. Furthermore, the spacing of the screws followed that of the requirements for built-up compression members from CSA O86-14 (guidance on bending members was not found by the authors, and therefore engineering judgement needed to be used). Additional screws could help to achieve a more developed connection between the main timber members and the added panels. Different types of screws, or different screw embedment lengths could also be investigated. The test series presented in this paper was somewhat limited by the number of members available. If a greater number of members were available, it would have been beneficial to experiment with the effect of different aspects of the repair, such as changing the repaired length, screw type, screw embedment length, and even investigating different fasteners such as adhesives. Each of these aspects needs future research. In addition to improvements to the mechanical repair, there is also a need to fully understand the thermomechanical degradation of engineered timber to accurately determine how much strength has been lost due to the fire exposure. In a real application of the repair of a building post-fire, this may be challenging as the fire exposure may be difficult to quantify in the first place. While the visible damage could be quantified visually by assessing the extent of the charring, degradation beyond the char front may also have occurred including the further breakdown of the adhesives. While zero-strength layers are present in code procedures (such as CSA O86-14) to account for heated wood beyond the char front, previous studies have shown that these allowances may not be conservative (Lange et al. 2015; Quiquero et al. 2018; Wiesner et al. 2017). This uncertainty makes it difficult to assess the area of timber in need of repair. Moreover, engineered timber and solid timber sections (such as heritage timber) will degrade differently in fire (due to the presence of adhesives in engineered timber), and so the damage induced on a timber section will vary based on if engineered timber or heritage timber is considered. To have an effective method of repairing a fire-damaged timber member, a further understanding of the thermomechanical degradation of engineered timber is still needed. Due to the differences between engineered timber and solid heritage timber, improved repair procedures are needed for both these timber types.

The fire performance of the repaired member must also be considered should another fire occur in the presence of a member that had been previously repaired. The configuration of the repaired members examined in this test series consisted of timber panels secured into the main member via structural screws. Future research is needed regarding the fire performance of the timber panels and main member, as it has been seen that fire may be able to penetrate seams of other materials fastened to timber (Chorlton et al. 2020). Evidently, future testing is required to understand the potential for fire to reach the main member before the side panel has completely charred through. Furthermore, metal screws have the potential to conduct heat into the timber. It has been suggested that exposed steel as a part of a connection can transfer heat into the timber and increase the char rate (Barber 2017). Future research is needed to better understand the effect of heat transfer between the screws and the timber.

Cost Analysis of Hypothetical Building Repair

In addition to practitioners needing guidance regarding how the repair of a timber structure might be carried out, insurance companies also need information concerning how to appropriately balance their premiums for these timber structures. A cost analysis has been carried out on the hypothetical repair of an exposed timber structure if it were to experience char damage from a fire – but also conducted to evaluate research needs required to make an accurate cost assessment. It will be found that due to the assumptions that are needed in this hypothetical example, this example serves primarily as a case study pointing to the research needs regarding what should be done next to improve the repair procedure and determine an associated cost to repair.

For the purposes of this hypothetical example, the assumptions made regarding the setup of this case study include that the columns considered in this case study will be similar to the columns tested, Douglas Fir species of 16c-E stress grade. It will be assumed that the columns will be square and slightly larger than the columns tested, at 415 x 415 mm. One level within a building will be evaluated, assuming a floor height of 3 m. A hypothetical floorplan is considered, consisting of a square column grid with 80 columns in total. This hypothetical floorplan is seen in Fig.11 (for illustrative purposes only and not to be construed as what is permissible by building codes).

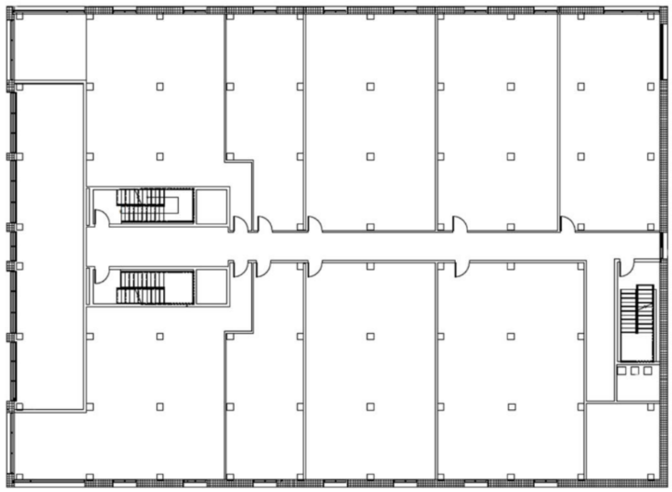

Fig. 11. Hypothetical building floor plan (for illustrative purposes only).

While some columns will be corner and perimeter columns and are therefore likely to have one or more sides not exposed directly to fire, it will be assumed that charring occurs on all four sides of the column. This will be done to be conservative, as previous studies have shown that gypsum board is not always effective at protecting timber from fire (Brandon and Östman 2016; Chorlton et al. 2020; Hadden et al. 2017; Su et al. 2018). Furthermore, all interior columns are assumed to be unencapsulated and exposed to the fire. This setup is not meant to necessarily reflect what is currently permissible by current code allowances, but to reflect the architectural desire to have exposed timber within tall structures.

Nine damage cases will be examined. The first three where the fire has been limited to one single column, for durations of 30 minutes, one hour, and two hours. In all cases, it will be assumed that charring has occurred along the entire surface, but the char depth with time is reflective of the standardized charring rate. The charring rate with time is considered arbitrary as a standardized fire is not indicative to a real fire. A standard fire charring rate was chosen for simplistic illustrative purposes. Cases 4-6 will expand the hypothetical fire to one compartment, containing eight columns (taken as the average number of columns in the compartments of Fig. 11., for illustrative purposes) for durations of 30 minutes, one hour, and two hours, respectively. The Cases 7-9 will consider the entire floor of the building (considering 80 columns), for the same heating durations as the previous cases. In this case study, it is assumed that the fire burns out and water suppression is not used. If water suppression were to be needed, the damage to the timber may be altered potentially requiring different repair strategies and altering the cost of repair.

The charring rate used will be 0.7 mm/min, as per CSA O86-14 notional charring for Glulam, as well as a zero-strength layer of 7 mm (CSA Group 2014). This charring rate is not necessarily reflective of the charring that would occur in a real structure, which is highly dependant on several factors unique to the fire, the compartment, and the timber itself. Moreover, the zero-strength layer of 7 mm (which as previously mentioned) has raised concerns that it may not always be conservative will nevertheless be considered herein in order to follow CSA O86-14. The charred depths are taken as 21 mm for Case 1 (30 minutes), 42 mm for Case 2 (1 hour), and 84 mm for Case 3 (2 hours), as per the CSA O86-14 charring rate of 0.7 mm/min (CSA Group 2014). A zero-strength layer of 7 mm brings the total depths that would be removed and repaired to 28 mm for Case 1, 49 mm for Case 2, and 91 mm for Case 3. These char depths are thicker than the panels used in the test series in the previous sections of this paper. Thus, for a repair to be carried out at these damage states, future research is needed using panels of these thicknesses. If thicker panels are needed, the beam may have been exposed to a higher severity fire which along with increasing char depth may also increase the thermal degradation beyond the char layer. Thicker panels will be needed to replace a larger area of timber that would be removed, and evaluation would be needed to see if the main member and thicker side panels act as a composite and are able to achieve desired capacity, or if the members are acting separately.

Preliminary costs are seen in Table 5 (note that no taxes are considered in this analysis). The factors considered in the following cost analysis include the materials, the cost of engineering, and the cost of labour. Not included in this analysis is the time and cost it may take to temporarily support the column's load while it is being repaired. The floor also may need to be propped up to unload the column for repair. Capital lost due to business discontinuity is also not considered and will vary greatly between specific structures. Engineering and labour hours noted in Table 5 are based on the length of time taken by the researchers to carry out the experimental test programme described in the Experimental Methodology section of this paper. Material costs are also extrapolated from the repair costs of the test programme described above. Engineering costs per hour have been derived from fee guidelines published by the Ontario Society of Professional Engineers (2015), and construction labour costs per hour have been taken from Statistics Canada (2019) data. Cases 4, 5, and 6, as well as Cases 7, 8, and 9 were identical to Cases 1, 2 and 3, respectively, but were multiplied to consider all columns in the hypothetical compartment or on the hypothetical floor. At this larger scale, it is possible that the cost per unit of the materials may be altered from what is listed. Costs are reflective of a repair conducted in Toronto, Ontario, Canada in late 2019 (costs will vary in other regional jurisdictions and with time).

Table 5. Cost of the hypothetical column repair

Case Number	Item	Quantity	Cost per unit ($ CAD)	Total Cost ($ CAD)
1 – 30 minutes, single column	Structural Screws	250 screws	4.49	1122.50
	Timber panels	0.13 m^3	3 285.61	427.13
	Engineering Cost	2 hours	215.00	430.00
	Labour Cost	6 hours	23.80	142.80
	Total Cost	--	--	2121.63
2 – 1-hour, single column	Structural Screws	250 screws	4.49	1122.50
	Timber panels	0.21 m^3	3 285.61	707.09
	Engineering Cost	2 hours	215.00	430.00
	Labour Cost	8 hours	23.80	190.40
	Total Cost	--	--	2449.99
3 – 2 hours, single column	Structural Screws	250 screws	4.49	1122.50
	Timber panels	0.42 m^3	3 285.61	1 388.51
	Engineering Cost	2 hours	215.00	430.00
	Labour Cost	12 hours	23.80	285.60
	Total Cost	--	--	3226.61
4 – 30 minutes, all columns in compartment	Total Cost	--	--	16 973.04
5 – 1 hour, all columns in compartment	Total Cost	--	--	19 599.91
6 – 2 hours, all columns in compartment	Total Cost	--	--	25 812.88
7 – 30 minutes, all columns on floor	Total Cost	--	--	169 730.40
8 – 1 hour, all columns on floor	Total Cost	--	--	195 999.20
9 – 2 hours, all columns on floor	Total Cost	--	--	258 128.80

The costs listed in Table 5 are the lowest possible costs for repairing timber members using replacement timber panels and structural screws. Since these costs correspond to the test programme described in the previous section which indicated that while some stiffness was regained, an enhanced repair procedure was needed to achieve composite action and regain full strength capacity. More extensive repairs are needed, necessitating increased materials, raising engineering and labour costs, as well as potentially increasing the time it takes for the repair to be completed (further disrupting business continuity). Furthermore, the engineer would need to check that the capacity of the connection is sufficient for the specific situation, which may necessitate a more extensive connection than is accounted for in Table 5. Other aspects that may also increase the cost of a building repair but may not be directly related to the structural performance of the timber include the extent of the smoke damage. The structure is likely to need significant cleaning for the smoke damage, which will be dependant on the extent of the fire. A fire investigation may also be necessary which would further increase the repair cost. Finally, as previously mentioned, the repair cost considers the repair of the columns only. Should repair of other members be required, the cost will further increase.

In some cases, it is possible that replacement of timber members may be more economical than repair of timber members. Utilizing the same price per cubic meter of timber as in Table 5, the cost of a replacement column (materials only) would cost approximately $1700CAD. Even accounting for materials only (not including increased labour costs that would be required to repair), the replacement option is still less expensive that the hypothetical 1-hour and 2-hour exposure repairs. Thus, for severely damaged columns, replacement may be more economical than repair. In some cases, a repair may still be preferred however, which may be due to environmental considerations or instances where the fire damage is mild.

Conclusions and Future Research Needs

Given the motivations for tall timber construction, with these structures becoming increasingly taller, matched with the desire to leave timber structural members exposed, now more than ever there is a need to understand the potential for the repair of fire damaged structural members. Currently, there exists little to no guidance in Canada (and limited guidance internationally) directing practitioners on potential repair strategies, and the lack of existing information makes it very challenging for insurance companies to appropriately balance their premiums for resilience consideration. The experimental test programme and corresponding hypothetical cost analysis presented in this paper provided a first-look at a possible repair technique to be built upon by other researchers, and from which several research needs were identified.

From the experimental test programme, some clear trends were observed across all samples regarding the recovery of strength and stiffness by the repaired members. While the objective of this repair relied on achieving composite action between the main members and the panels installed during repair, which was not entirely achieved as seen by the lower strengths and capacities of repaired members compared to control members, the repaired members still showed some improvement in stiffness. When brought to a load meant to represent a serviceability limit state, all of the repaired members deflected less than when they were carved. One of the repaired members (Member 3) even had less deflection than its corresponding control member. Another example of a significant recovery of stiffness is Member 5, deflecting over 5 mm (35%) less than the carved member after it was repaired. These trends showed that overall, the repair was able to regain a portion of the stiffness lost during damage. With regards to overall strength capacity, none of the members reached the ultimate strength that was carried by the control members, with each member failing at only between 49-66% of the load of its corresponding control member. This indicates the need to further examine possible alterations which may improve this repair procedure, to the point where composite action is fully enabled, and a larger portion of the original strength is recovered.

A hypothetical cost analysis provided a baseline as to what the repair of a fire damaged member might cost, however costs found were an absolute minimum of what might be expected as the costs reflected the experimental test programme, which showed that more extensive repairs may be needed. However, both the experimental test programme and cost analysis were successful in identifying many areas where research still needs to be done to inform a more accurate analysis. Several steps should be taken to further develop the effectiveness of timber repairs post fire and determine the corresponding cost. To develop a possible procedure for repairing fire-damaged timber members, the following research needs should be considered:

- Future studies defining the heat damage done to engineered timber beyond the char layer (including degradation of the adhesives), such that residual strength can be accurately determined, and the extent of any required repair will be well understood;
- Methods of achieving composite action, including extending the length of repair or altering the connection;
- Different repair configurations, including using thicker replacement panels that may correspond to more severe fire exposures, as well as using different fasteners including adhesives;
- Testing of repaired members through a variety of mechanical loading test setups (including loading in compression), as well as creating induced fire damage through a variety of heat exposures;
- The possibility of heat transfer between the timber and the screws (or other metal connections) which may alter the charring or damage to the timber;
- The effect of pre-existing cracks as well as delamination on both the structural and fire performances of timber members;

- Once a repair procedure is established, the fire performance of the repair configuration should be studied.

Further research on each of these elements would be beneficial in the development of an effective repair procedure, and consequentially the determination of the associated cost would then follow.

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Data Availability Statement

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

References

Alam, P., Ansell, M. P., and Smedley, D. (2009). "Mechanical repair of timber beams fractured in flexure using bonded-in reinforcements." *Composites Part B: Engineering*, Elsevier Ltd, 40(2), 95–106.

Armstrong, L. D., and Kingston, R. S. T. (1960). "Effect of Moisture Changes on Creep in Wood." *Nature*, 185, 862–863.

ASTM International. (2014). *ASTM D143-14: Standard Test Method for Small Clear Speciments of Timber*. West Conshohocken, PA.

Barber, D. (2017). "Determination of fire resistance ratings for glulam connectors within US high rise timber buildings." *Fire Safety Journal*, Elsevier Ltd, 91, 579–585.

Bartlett, A. I., Hadden, R. M., and Bisby, L. A. (2019). "A Review of Factors Affecting the Burning Behaviour of Wood for Application to Tall Timber Construction." *Fire Technology*, 55, 1–49.

Brandon, D., and Östman, B. (2016). *Fire Safety Challenges of Tall Wood Buildings-Phase 2: Task 1-Literature Review*. Quincy, MA.

Chorlton, B., Forrest, B., Gales, J., and Weckman, B. (2020). "Performance of Type X Gypsum Board on Timber to Non Standard Fire Exposure." *Fire and Materials*, 1–16.

Chorlton, B., Mazur, N., and Gales, J. (2019). "Incorporating Timber Education into Existing Accredited Engineering Programs." *2019 Canadian Engineering Education Association (CEEA-ACEG19) Conference*, Ottawa, ON.

Correia, J. R., Bai, Y., and Keller, T. (2015). “A review of the fire behaviour of pultruded GFRP structural profiles for civil engineering applications.” *Composite Structures*, 127, 267–287.

CSA Group. (2014). *CSA O86-14: Engineering Design in Wood*.

CSA Group. (2016). *O122-16: Structural glued-laminated timber*. Toronto, Ontario.

Frangi, A., Fontana, M., Hugi, E., and Jübstl, R. (2009). “Experimental analysis of cross-laminated timber panels in fire.” *Fire Safety Journal*, 44, 1078–1087.

Franke, S., Franke, B., and Harte, A. M. (2015). “Failure modes and reinforcement techniques for timber beams-State of the art.” *Construction and Building Materials*, Elsevier Ltd, 97, 2–13.

Gomes Ferreira, J., Cruz, H., and Silva, R. (2017). “Failure behaviour and repair of delaminated glulam beams.” *Construction and Building Materials*, 154, 384–398.

Hadden, R. M., Bartlett, A. I., Hidalgo, J. P., Santamaria, S., Wiesner, F., Bisby, L. A., Deeny, S., and Lane, B. (2017). “Effects of exposed cross laminated timber on compartment fire dynamics.” *Fire Safety Journal*, 91, 480–489.

Klippel, M., and Frangi, A. (2017). “Fire safety of glued-laminated timber beams in bending.” *Journal of Structural Engineering*, 143(7).

Klippel, M., Frangi, A., and Fontana, M. (2011). “Influence of the Adhesive on the Load-Carrying Capacity of Glued Laminated Timber Members in Fire.” *Fire Safety Journal*, 10, 1219–1232.

Lange, D., Bostrom, L., Schmid, J., and Albrektsson, J. (2015). “The Reduced Cross Section Method Applied to Glulam Timber Exposed to Non-standard Fire Curves.” *Fire Technology*, 51(6), 1311–1340.

Lineham, S. A., Thomson, D., Bartlett, A. I., Bisby, L. A., and Hadden, R. M. (2016). “Structural response of fire-exposed cross-laminated timber beams under sustained loads.” *Fire Safety Journal*, Elsevier, 85, 23–34.

Morales-Conde, M. J., Rodríguez-Liñán, C., and Rubio-De Hita, P. (2015). “Bending and shear reinforcements for timber beams using GFRP plates.” *Construction and Building Materials*, 96, 461–472.

National Lumber Grades Authority. (2017). *Standard Grading Rules for Canadian Lumber*. Vancouver, BC.

National Research Council of Canada. (2020). “National Building Code of Canada.” Ottawa, Ontario.

Oliver, C. D., Nassar, N. T., Lippke, B. R., and McCarter, J. B. (2014). “Carbon, Fossil Fuel, and Biodiversity Mitigation With Wood and Forests.” *Journal of Sustainable Forestry*, Taylor & Francis, 33(3), 248–275.

Ontario Society of Professional Engineers. (2015). *OSPE Fee Guideline 2015*.

Quiquero, H., Chorlton, B., and Gales, J. (2018). “Performance of Adhesives in Glulam after Short Term Fire Exposure.” *International Journal of High-Rise Buildings*, 7(4), 299–311.

Quiquero, H., and Gales, J. (2016). “Behaviour of Fire Damaged Engineered Timber Beams.” *5th International Structural Specialty Conference of the Canadian Society of Civil Engineers*, London, ON, 993–1001.

Schmid, J., Klippel, M., Just, A., and Frangi, A. (2014). “Review and analysis of fire resistance tests of timber members in bending, tension and compression with respect to the Reduced Cross-Section Method.” *Fire Safety Journal*, Elsevier, 68, 81–99.

Stanier, S. A., Blaber, J., Take, W. A., and White, D. J. (2016). “Improved image-based deformation measurement for geotechnical applications.” *Canadian Geotechnical Journal*, 53(5), 727–739.

Statistics Canada. (2019). *Table 14-10-0340-01 Employee wages by occupation, annual.*

Su, J., Lafrance, P., Hoehler, M., and Bundy, M. (2018). *Fire Safety Challenges of Tall Wood Buildings – Phase 2: Task 2 & 3 – Cross Laminated Timber Compartment Fire Tests*. Quincy, MA.

The Institution of Structural Engineers. (2010). *Appraisal of existing structures*. IStructE Ltd.

Wiesner, F., Deeny, S., and Bisby, L. A. (2020). “Influence of ply configuration and adhesive type on cross-laminated timber in flexure at elevated temperatures.” *Fire Safety Journal*.

Wiesner, F., Randmael, F., Wan, W., Bisby, L., and Hadden, R. M. (2017). “Structural response of cross-laminated timber compression elements exposed to fire.” *Fire Safety Journal*, 91, 56–67.

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