Building Tomorrow's Society
Bâtir la Société de Demain

Fredericton, Canada
June 13 – June 16, 2018/ *Juin 13 – Juin 16, 2018*

An Investigation into the Resilience of Glulam Timber Beams after Fire Exposure

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Abstract: A critical element in the design of timber structures is the building's response in the event of a fire. Currently, engineered timber, which relies on complicated adhesives to join wood elements, is used to build up large section sizes for structural assemblies. When not encapsulated and exposed to fire, however, engineered timber mechanically degrades via charring and adhesive breakdown. Understanding this degradation will provide greater confidence for the practitioner for new tall timber designs since it is necessary to better determine the performance of engineered timber under fire exposure, and also to determine the level of reparability required after a fire. Therefore, the present study examines a hypothetical design example as motivation for research into fire response in tall engineered timber construction. The case study prompts the need for demonstrative testing to build confidence in use of exposed (unencapsulated) timber elements. For this, the after-fire performance of engineered timber Glulam beams was considered in comparison to undamaged control beams in twelve, two point, loading tests. The twelve beams were exposed to localized fires in controlled locations or cut specifically to represent charred timber. After self-extinguishment, the char depth of the beams was measured, and an anticipated strength for the loading conditions was considered. Undamaged control beams failed on average at 21.2 kN (total applied load). Beams charred in the shear region failed on average at 13.9 kN, beams charred in the moment region failed with an average 19.3 kN failure, and the beam whose cross section was mechanically reduced (carved) in the shear region with 18.4 kN failure. This indicates that the charred shear region failed at a lower load than charred in moment region, and that mechanical cross-section reduction failed at a higher load suggesting adhesive degradation is playing a role in these decreased strengths.

1 INTRODUCTION

Engineered timber is becoming an increasingly common building material, as building codes evolve incorporate recent research findings, and as there is a growing appreciation for use of sustainable construction materials. Engineered timber is characterized by the fusion of wood and adhesive, and includes products such as glued laminated lumber (Glulam).

One of the challenges of building with engineered timber products is designing for a high level of fire protection. This can be done through encapsulation with fire-rated gypsum boards, or with other fire protection measures such as introduction of fire retardants (see FPI 2014). When exposed to a fire, timber begins to pyrolyse and char, and in the case of engineered timber, other effects such as adhesive degradation also begin to occur. Current design guidance (Eurocode for example) allow for a portion of the fire-exposed member to be considered "undamaged". This is typically done by assuming a zero strength layer beyond the char zone, which is meant to implicitly account for degradation effects that include the break-down of adhesive. Yet, this quantification is in need of study (see Lange et al. 2015; Quiquero and Gales 2017; Gales et al. 2018). To build further confidence at large scale it is important to better understand the severity and consequences of adhesive degradation. This will also help to determine the after-fire performance of engineered timber, thereby beginning to determine the level of reparability that would be required. This level of reparability influences if and how long the building would be out of service after a fire – and thus how well the building can enable resilience of the operations within.

The purpose of this study is to better understand the performance of Glulam beams during and after exposure to a controlled localized fire source. In particular, it was of interest to determine if they self-extinguish (exposed timber) and if their residual strength is comparable to a specimen whose cross section was simply carved away (to isolate the adhesive degradation effect)). Such results provide new indications towards determination of the resilience of a building. Further, the results help define the orientation of next research steps.

1.1 Motivation

A key consideration in the design of buildings is the cost. Yet for engineered timber buildings there are few comparable building and design case studies to use as benchmark guidance. While engineered timber construction has some advantages, such as relatively short construction timeframes, it has varying costs related to different levels of encapsulation and fire protection (NRCC 2017). An investigation was completed to examine the cost to build a particular building using engineered timber, compared to both steel and reinforced concrete construction (see NRCC 2017). In the engineered timber category, two options were analyzed: exposed (unencapsulated) and full encapsulation with two layers of fire-rated type X gypsum board. The exposed option, however, did not include any costs for fire protection of the structural members, which when not done by encapsulation would normally be accomplished by increasing the member sizes which would also increase the overall cost of the option.

Overall, the NRC analysis was favourable towards timber, with both the exposed and encapsulated timber options scoring well in the construction cost, time to build, and maintenance cost categories (NRCC 2017), but it does primarily suggest using encapsulation as the fire protection strategy. On the other hand, exposed timber is architecturally desirable, and appearance is one of the incentives for its use, advantages that are lost when encapsulation is needed in order to fulfil a strict standardized fire-rating requirement. In other situations, oversized timber structural members are specified when it is desired to leave them exposed, thus allowing for a char depth to be sacrificed in the event of a fire. Although not considered in the NRC analysis, a design-cost comparison that includes the potential use of larger member sizes to satisfy fire protection requirements and consequent increased cost is required before exposed engineered timber construction can fully be realized. Accounting for the extra cost is required for a more fair comparison of design options. This subject requires in depth investigation into the fire response of engineered timber elements to develop better guidance for scaling of members during design of exposed timber in demonstration buildings.

The motivation of this study, therefore, is to begin to understand the member sizing required to meet a targeted fire resistance, so that appropriate analyses can be carried out and the potential for use of exposed timber can be better assessed.

2 DESIGN CONSIDERATIONS

As a first step in the research, existing studies on structural design of engineered timber with respect to fire safety were examined, particularly with respect to use of non-encapsulated timber members. It was found that clear case-study design examples that explicitly address fire protection strategies for exposed engineered timber are rare (in the public domain at least). Several are highlighted in the following section followed by a hypothetical case study developed by the authors intended to identify research avenues and generate discussion around fire safety issues in engineered timber construction, as opposed to providing specific design guidance.

2.1 Literature Design Examples

To instill confidence in engineered timber construction, Skidmore, Owings & Merrill LLP (SOM) has offered two detailed demonstration timber hybrid building designs in public literature (see SOM 2013; SOM 2017).

The first is a 42-storey timber hybrid, featuring engineered timber structural elements connected using steel rebar passing through concrete joints. The engineered timber beams, columns and floor are connected through lap splicing of the rebar. The floors and shear walls are specified as cross-laminated timber (CLT) and the columns as Glulam. The fire protection design of the structural system is not entirely addressed in this investigation; instead it is stated as general provisions with recommendations that indicate further analysis is necessary. It is indicated that it may be possible to leave the bottom face of the CLT exposed (unencapsulated), if it can be assumed that the timber would self-extinguish, which would require further testing (SOM 2013). This is still being identified as research need today (Jeanneret et al. 2017).

SOM and the American Institute of Steel Construction (AISC) have also created a demonstrative case study featuring a 9-storey steel and timber residential building. This building makes use of steel for the structural framing, as well as CLT for the floors. The CLT floors are topped with concrete, and each CLT plank will be notched in the manufacturing process in order to connect with the steel beams and columns. The height and floor area of the building exceed the (current) allowable values for combustible construction, as outlined by American standards. It is suggested to use cladding or fire-resistant coatings to provide the required fire resistance rating of each structural element, in particular using fire rated gypsum board to encapsulate the CLT ceilings as required. The CLT will not be present in the core of the building, in order for the egress path to directly meet the code requirements (SOM 2017). Again detailed guidance for fire safety design is not entirely provided in this example.

These case studies are innovative in their use of engineered timber to demonstrate possible designs for high-rise buildings. They instill confidence with the use of timber in terms of structural design; however, the case studies focus primarily on aspects of the design other than fire safety, even though this is one of the major concerns for the use of timber in high-rise structures. Beyond these documents, there are few published case studies, to the awareness of the authors, on design of tall timber buildings in the public domain which address fire safety. Yet, such case studies are needed from a practitioner point of view. To illustrate this point, a theoretical case study is presented in the next section, after other literature on the fire response of engineered timber is reviewed. In addition there is literature on fire performance of engineered timber elements available to the practitioner (see FPI 2014; Yang 2015; CSA 2014; and Gales et al. 2018) and this was primarily referred to in the hypothetical case study below.

2.2 Hypothetical Conceptual Example

This section summarizes a theoretical case study which explicitly addresses some aspects of fire safety in engineered timber construction. It was developed by the primary authors as part of the final year undergraduate capstone design course with the strict intent to demonstrate how a carefully targeted case study might generate discussion regarding research needs and fire safety issues in engineered timber construction. Through publication of more detailed case studies related to real designs, practitioners might work towards methods by which to instill appropriate confidence in tall timber construction.

For the present hypothetical case study, an entirely theoretical concept building, *Hypothetical A*, is designed to follow the guidance of the Canadian building codes. The site of *Hypothetical A* is within the King Street West district, in Toronto, Ontario, a vibrant neighbourhood where sustainability and heritage are important.

As a result the architectural design of *Hypothetical A* is architecturally inspired by historic buildings that have been lost to new development (old Union and old Toronto Star building).

Figure 1: Structure of *Hypothetical A* engineered timber hybrid building (left), and exterior of *Hypothetical A* building (right).

The theoretical structure of *Hypothetical A* can be seen in Figure 1. It is not the intent to provide anything other than a hypothetical rendering of the structure and therefore specific design is omitted. An engineered timber structure can take on many different structural timber configurations. This could, though not limited to, include: Glulam beams and columns, Nail laminated timber slabs, Cross laminated timber cores and partitions. Typical load path configurations in many timber buildings feature a grid shape.

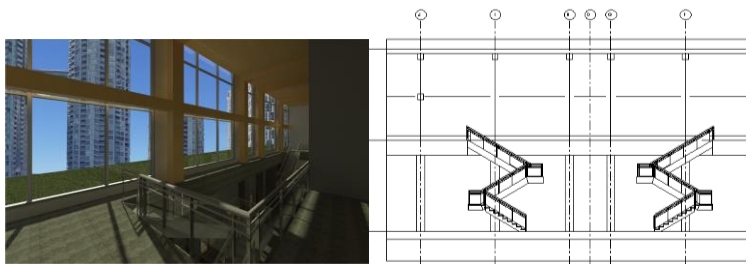

Figure 2: An architectural rendering of the interior of *Hypothetical A*, showcasing the use of exposed Glulam beams and columns.

Fire safety was taken as a priority in the design of *Hypothetical A*. As shown in Figure 2, the potential for showcasing timber as an architectural goal was specifically included in order to highlight and generate discussion on fire safety related issues. In this hypothetical building, exposed structural members are oversized for increased resistance toward fire; this is combined with other fire protection strategies including encapsulation and use of concealed connections and non-combustible materials. In addition, *Hypothetical A* is entirely sprinklered and has multiple egress paths through the multiple cores. The CLT cores would house egress stairs, and would be proposed to be encapsulated with multiple layers of fire-rated gypsum in order to meet the necessary 2 hour FRR (FPI 2014). In an attempt balance exposure of some of the timber structure while managing issues related to structural fire safety design, it might be proposed that the NLT slabs are encapsulated on the top and left exposed on the bottom sides. It could further be proposed, for example, that beams and columns are left exposed to showcase the use of timber. For purposes of discussion, they might be oversized in a fashion similar to guidance provided in CSA O86-14 Annex B by accounting for an assumed char depth plus allowance for a zero strength layer. The theoretical measures outlined would accomplish the key intent of prioritizing fire safety while managing to expose some of the timber structure and would contribute architectural value in addition to design innovation for building *Hypothetical A*. However, a major limitation to the proposed approach lies is the fact that there is presently no definitive benchmark for the use of such strategies for fire safety in design of engineered timber buildings. For example, questions arise such as will the timber self-extinguish? And will the increase in member size ensure that adequate strength will remain in the beams post-fire to enable the building's resilience? Therefore, while the theoretical concepts can be laid out on paper, they clearly need extensive discussion with support by definitive research results and/or additional testing before incorporation into any real building design and final costing determined. On the other hand, the systematic design and analysis of building *Hypothetical A* points directly to the importance and need for development of exemplar case studies by others that lay out specific fire protection strategies for exposed timber.

3 EXPERIMENTAL METHODOLOGY

The second stage of this study was focussed towards improved understanding of some of the effects of fire on engineered timber, specifically glulam, members. As noted in the hypothetical case study above, current allowances for the effective cross-sectional area of fire-damaged timber are determined by assuming a zero-strength layer beyond the char zone (CSA 2014). Use of this design approach is meant to implicitly account for effects of degradation of the element, including the breakdown of binding adhesives between the timber laminates, loss of moisture, and other effects. As an alternative, a practitioner can utilise finite element modelling techniques for design, but because this may be very time consuming, the more simplified approach is often utilised. Yet, the simplified guidance is often questioned in literature and a range of different values for the depth of zero strength layer are currently promoted (Lange et al. 2015; Quiquero and Gales 2017; Gales et al. 2018). In fact, it is sometimes construed that it may even be penalizing (necessitating overly large section sizes), as most studies on fire-exposed timber rely only on standardized fire exposure. The research herein presents a preliminary examination into the response of Glulam beams to localized fire exposure, in order to better understand the underlying mechanisms which govern the potential behaviour of engineered timber during a fire.

3.1 Specimens

The Glulam specimens considered were grade 24f-ES, spruce-pine-fir of dimensions 45x195x4200 mm. The anticipated hand-calculated estimated ambient temperature failure load (for the ultimate limit state) using the test setup described below is 13.6 kN due to flexure for an undamaged beam, as per the procedure described in CSA O86-14. This calculation was performed including modification factors, meant to underestimate the strength of the member. It is noted that the beams at ambient temperature are much stronger than this (21.2KN). It should be noted that the scale of thickness of the beams is much less than would normally be used in design, so caution should be taken for drawing conclusions in the results based on reduced cross sections. Real beams may have more reserved cross sections.

3.2 Test Procedure

A custom experimental test procedure was designed for the fire response tests conducted with some aspects of the procedure drawn from existing ASTM standards. The testing consisted of two phases: the first was fire exposure and the second was mechanical testing of the beams. The first phase of the experiments was completed at the University of Waterloo Fire Research Facility on October 24th, 2017. The overall test setup, consisting of a single beam locally exposed to a 1 meter long line fire is shown in Figure 4. The fire for each test was fuelled by 1 litre of kerosene, contained in a meter long steel trough. The fire burned for approximately 5 minutes, producing a peak surface temperature of 900°C on the exposed beam. The fire was meant to represent a short period of real fire impingement on a structural element during a building fire. The fire exposure was designed as non–standard to explicitly induce a controlled and quantified amount of charring on the samples from test to test. Six identical Glulam beams were charred at different locations on the beam for the full duration of the fire (the remaining six were control samples or samples that would be carved mechanically to simulate charring for a total of 12. Carving was performed using cutting tools). All were charred on two sides in the desired charring region to provide a relatively consistent char depth. Half of the six beams were charred directly in the center of the beam, and the other half were charred on one end, 270 mm from the edge of the beam (Figure 3). This heating configuration had negligible differences in the amount of char observed. The charring locations were chosen to give varied results in the second phase of testing; those charred in the center were expected to experience a large moment force, and those charred on the end a large shear force.

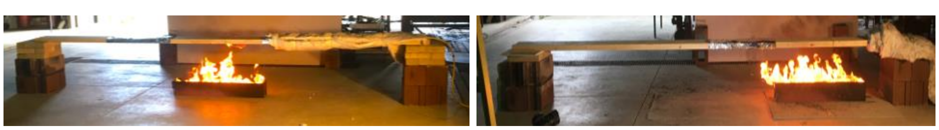

Figure 3: Test setup of the heating portion of the experiment for the moment region (left) and the shear region (right), in which the 4.2m Glulam beams were exposed to a pool fire for approximately 5 minutes on each side

The charred regions of the beam were limited by the length of the fire source. Aluminum foil was wrapped around the beam immediately adjacent to the intended char zone to limit the radiant heat and flame spread to other parts of the beam. When the fire burned out, after approximately 5 minutes, the beams were left to self-extinguish. Since this happened immediately, no water was used to control the fire or for extinguishment. An area of cross section equivalent to the charred area on the six beams that underwent the fire testing was removed from two of the non-charred beams to facilitate comparison of strength of the beams. The area removed was approximately 5 mm deep and 1 m long. It was removed from all sides of the beams, in the same locations as were char during the fire tests (either in the moment region or the shear region). All beams therefore had the same non-charred cross-sectional area for the mechanical testing in Phase 2 below. In this manner, any variation in the strength data will be due to factors other than the effective cross section reduction of the charred beams. The intention of this charring degree (5 mm) is not to be thought of as the authors providing information on timber's fire resistance, but rather a controlled set amount of damage that can allow the underlying breakdown mechanisms of timber exposed to fire to be rationally studied.

The second phase of testing, the two point mechanical loading, occurred the week of November 20th, 2017. The delay in performing after fire tests was to allow for the beams to re-acclimatize to real building temperature and humidity conditions. As displayed in Figure 4, each acclimatized sample was loaded using a two-point loading test. The beams were supported by roller supports with an equal length between each support and the applied point load. Deflection of the beams was measured using digital image correlation techniques and the software GeoPiv RG (Stainer et al. 2015).

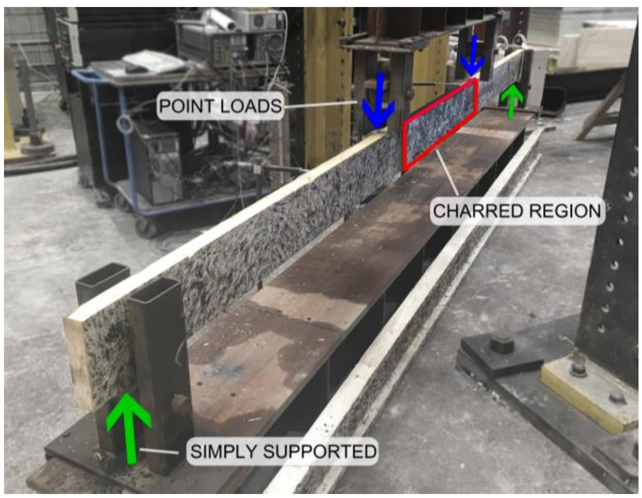

Figure 4: Mechanical loading of simply supported beam with two point loads. The beam in this image was charred in the centre, the region of high moment.

4 EXPERIMENTAL RESULTS

The stiffness and average failure loads obtained during the mechanical loading are summarized in Table 1. On an average basis, the control beams failed at the highest applied load, with all of the carved and charred beams failing at lower loads. All failure loads were higher than the hand calculated estimated strength (13.6KN). The displacement of the beams as they were mechanically loaded was recorded and is presented in the plots of load versus vertical displacement in Figure 5. Amongst beams damaged in the moment region (midspan), the results were quite variable; the beam with mechanically reduced cross section failed at a relatively low load compared to the charred specimens. Amongst beams damaged in the shear region (towards the side), the beam that had its cross section mechanically reduced through carving displayed the highest stiffness, as the slope of its curve is the greatest, and experienced a higher failure load than any of the charred beams.

Table 1: Test Schedule and Results

Beam #	Damage type	Average Failure Load / Stiffness (EA)
1 - 2	Charred - Mid span	19.3 kN / 89400 kN
3	Carved - Mid span	13.7 kN / 89400 kN
4 - 6	Charred - Side	13.9 kN / 89400 kN
7	Carved - Side	18.4 kN / 89400 kN
8 - 9	Control	21.2 kN / 115000 kN

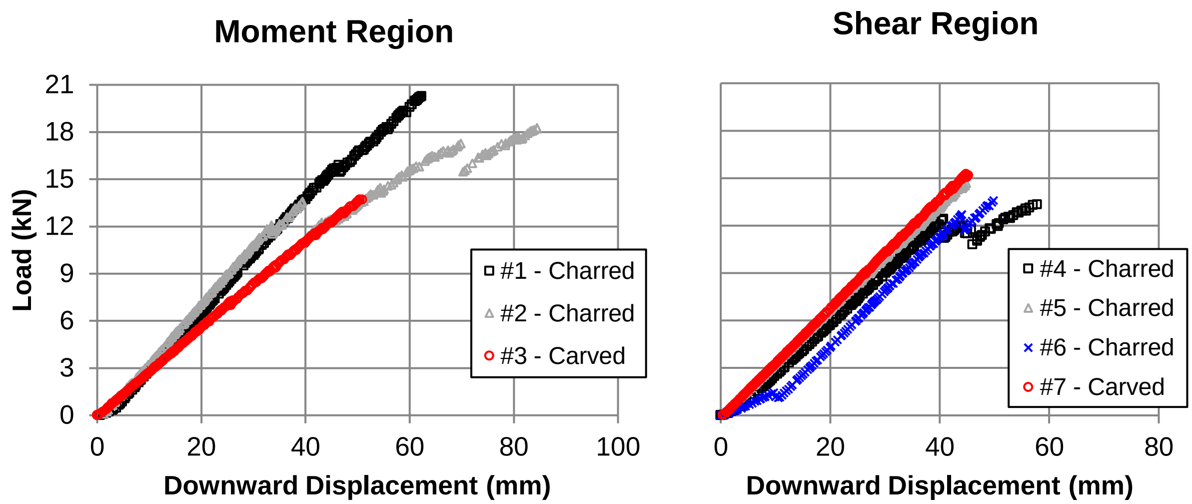

Figure 5: Load versus downward displacement of the carved and charred beams that have been damaged in the moment region (left) and shear region (right)

A comparison of the peak load reached by each beam before failure can be seen in Figure 6. The two control beams failed at relatively high loads with the difference possibly due to inherent materials defects between the two specimens. The beams charred in the moment region withstood relatively high loads, in comparison to the beam whose cross section was mechanically carved away which failed at a discernably lower load. In contrast, the beams charred in the shear region failed at relatively low yet consistent values of load, while the mechanically carved beam performed better, and failed at a higher load.

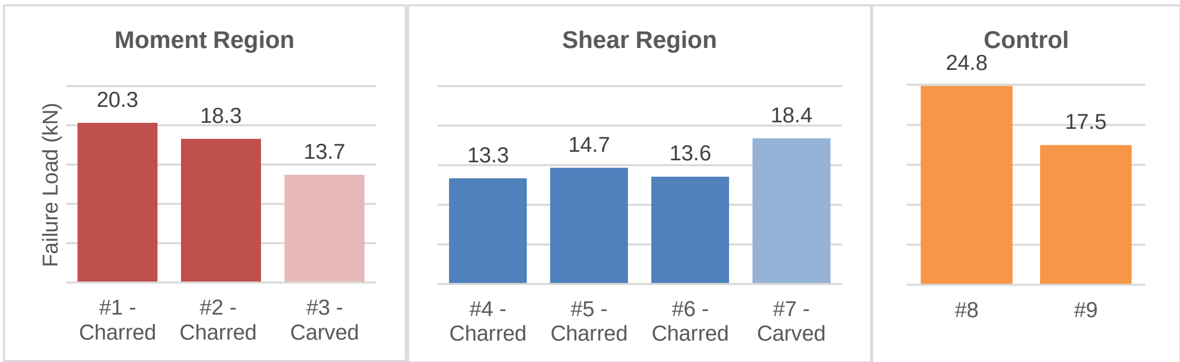

Figure 6. Comparison of the failure load of all beams damaged in the moment region, shear region, as well as the control beams

5 DISCUSSION AND FUTURE RESEARCH

The control beams failed at an average load 36% higher than the calculated failure load of 13.6 kN for an undamaged beam. This is expected considering the reduction factors for safety used in calculating the failure load. The beams that were damaged in the moment region failed on average, at a load 42% higher than their predicted strength of 11.1 kN. In this case, the calculated strength was determined via code procedures that account for reduction in cross section lost to char, as well as additional reduction in cross section due to heating (that may account for adhesive degradation and other effects). The beams damaged in the shear region were also predicted to have a failure load of 11.1 kN, while on average they failed at a load 20% higher than this. This analysis demonstrates that for the beams and localized fire exposures considered in these tests, the procedures for estimating failure load outlined in the code appear sufficient in ensuring that the specimen will fail at a higher load than predicted.

As anticipated, the control beams had a higher strength and stiffness than the damaged beams since they were full cross section, and no possible damage had occurred due to heating. The beams charred in the moment region performed comparatively to the control beams; their strength was not greatly reduced through heat exposure. The mechanically carved moment beam exhibited a relatively low failure load and stiffness, even in comparison to the charred beams. It is possible that this beam had a material defect that was not visibly apparent, causing the low failure load.

For the shear damaged beams, the beam whose cross section was mechanically carved away failed at a higher load and had a higher stiffness than the charred beams. Since the cross sectional areas for the charred and carved beams were very similar, the reduction in strength and stiffness of the charred beams is indicative of adhesive degradation. It is interesting that the better performance of the carved beam over the charred beams is a trend that only occurred in the set of beams damaged in the shear region. This was not observed in the set of beams damaged in the moment region (assuming the low failure load for the carved beam was not entirely due a material defect). This may indicate that the underlying failure mechanism of the beams was shear induced, or caused by a combination of shear and moment that occurred towards the sides of the beams, as they were loaded. Though not definitive from the present results, this could point to differences in response to fire exposure in regions of high moment versus high shear. This directly affects the design of timber connections, where notches in the shear regions are common, the design of connections would have to be carefully considered.

Apparent visible material defects resulted in two of the beams being excluded from the analysis, those beams would never be used in a real building due to their obvious visual defects. One of the beams was initially a control beam. It exhibited a high number of shakes, checks and wanes within the laminates and failed at a load of 14.0 kN, lower than the failure load for many of the damaged beams and was thus excluded here. One of the charred moment beams was also excluded because one of its laminates showed discoloration, and failed at a load of 13.1 kN. The variability in failure load of these beams due to defects was quite large.

It is recommended that future research be conducted to further examine the influence of material defects and direct fire exposure on the mechanical behaviour of glulam beams. The effect of fire on beams of larger cross section than the relatively thin sections tested here should also be investigated as these will be more representative of beams that would actually be used in a large timber building.

There is also a need for additional case study examples in literature that directly address fire safety, and offer structural and architectural designs that explicitly confront fire protection strategies. These further examples will be useful to instill confidence in the public and authorities having jurisdiction, in order for them to deem the safety of a building design.

6 PRELIMINARY CONCLUSIONS

The beams charred in the moment region exhibited an average failure load of 19.3 kN, only slightly below 21.2 kN, which is the average strength of the control beams. On the other hand, the charred beams performed significantly better than the beam whose cross section was mechanically carved, and ultimately failed at a load of 13.7 kN. Examining these results, two observations can be made. First, the mechanically reduced cross section beam may have performed poorly due to material defects. This beam, in addition to other beams that have been excluded entirely from the analysis due to visible defects, suggest that material defects can potentially reduce the strength of a specimen to a degree comparable to that of fire exposure. Second, the moment-region charred beams performed extremely well in comparison to the control beams which should be expected. The beams were charred on their long side, but tested in bending standing vertically. Bending is more impacted by depth and section modulus, shear is more impacted by cross-sectional area. As expected bending wouldn't be impacted much if the short sides didn't see much representative cross section reduction where differences would begin to be due to adhesive degradations. Expectantly, the beams charred in the shear region failed at an average load of 13.9 kN. This is lower than the failure load of the beams charred in the moment region, and considerably lower than that of the control beams. The reduced cross section beam in this instance failed at a load of 18.4 kN. The observation that areas of high shear seem to be more affected by fire exposure than the areas of pure moment warrants the need for further studies involving more representative beam sizes (thickness) as planned by the authors.

The reduction in cross-sectional area to replicate charring was shown to not correlate with the experimental data, suggesting adhesive degradation beyond the char layer does play a role in glulam beam capacity. Despite this, all fire exposed beams tested failed at loads higher than their calculated capacities using the O86 charring equations for glulam beams.

The preliminary analysis presented here of the code prescribed guidance as compared to studies of the actual behaviour of full sized beams under localized (or other) fire loading may provide information that is useful in determining the resilience of the material. Much more fire performance information is necessary however to provide full insight into issues around appropriate application of exposed timber in demonstration buildings. Such data might then be effectively used to formulate principles for design and incorporated into extended example fire safety case studies. These case studies would provide guidance for practitioners, authorities, and general public, in order to be able to fully realize the capabilities of engineered timber.

Acknowledgements

This research was performed utilizing resources from the NSERC Discovery program. The authors wish to thank Adrian Soble for allowing the use of his renderings featured in Figures 1 and 2.

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