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CHALLENGES OF SEISMIC DETAILING ON THE FIRE PERFORMANCE OF POST-TENSIONED TIMBER FRAMES

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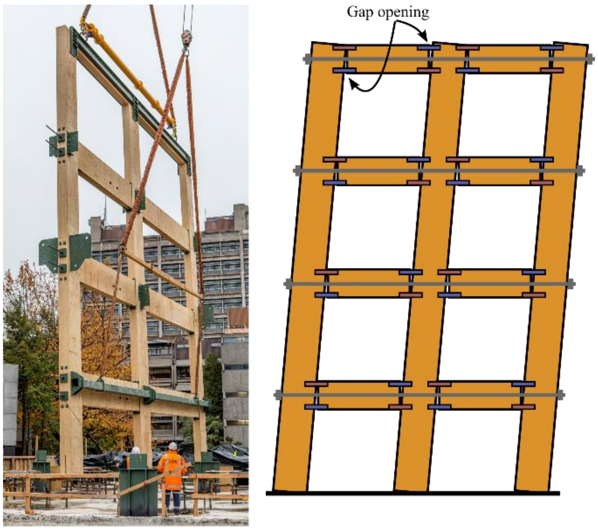
ABSTRACT

Post-Tensioned Timber (PTT) frames have significant advantages over traditional timber frame systems especially where a low damage design and fast construction are desired. New Zealand practitioners designing timber structures for fire are accustomed to applying ambient design methods to an element cross-section reduced by a char depth based on a duration of Standard Fire exposure following NZS 3603 or AS/NZS 1720.4. The behaviour of PTT frames in fire remains a concern because this approach does not account for the actual mechanics of PTT connection and frame response under natural fires that will occur in the structure. This paper examines the individual and interdependent response of PTT connection components (tendon, dissipater, fasteners, etc) to fire. It is shown that the ambient analysis tool for PTT connections, the Modified Monolithic Beam Analogy, cannot be applied to the fire case by only using char reduced cross-sections of timber elements. This approach of combining ambient methodologies with reduced cross-sections does not account for the specific connection detailing, which result in unique damage in fire that may govern the structural response. The responses of two seismically detailed PTT connections are predicted using this approach and compared to a first principles assessment of connection behaviour to demonstrate that failure will occur earlier than otherwise predicted. Numerical thermal analyses of these two connections also qualitatively corroborate the damage that occurs. This investigation establishes that additional studies are required to understand the complex behaviour of these connections when exposed to fire before a design methodology can be developed.

INTRODUCTION

Post-tensioned timber technology uses unbonded steel strands or high strength steel bars passing through ducts in timber LVL or Glulam elements with optional energy dissipaters to create low damage and re-centring frame or wall structures. Post-tensioned timber buildings are increasing in popularity among national and international researchers and practitioners [1–4] because they offer many advantages over traditional timber construction, especially in seismic areas, including a low-damage and self-centring structure, larger spans than otherwise possible in gravity dominated scenarios, and a cost-effective and fast method of construction as many moment resisting connections can be made in a single stressing operation [1,5,6]. Figure 1 shows the installation of a PTT frame and its schematic with energy dissipaters under a seismic load. Figure 2 shows the schematic of a beam-column connection of a PTT frame and the advantageous flag shaped moment-rotation response when rocking occurs at the beam-column interface stretching the dissipaters and the tendon.

For PTT frames under ambient Ultimate Limit State (ULS) loads, the beam and column elements are designed to remain elastic. So the beam-column connections, at which the rocking and energy dissipation occur, are the critical design items that govern the response of the frame. When a gap opens at the rocking interface, the “plane sections remain plane” assumption of Euler-Bernoulli beam theory is no longer valid. Instead the Modified Monolithic Beam Analogy (MMBA) [7–9], which establishes an equivalence between a monolithic connection and a rocking connection, is used to analyse the connection response [1]. Similarly, when a PTT frame is exposed to fire, the response of the connections to the fire will govern the structural response. In PTT frames many connections share the same tendon, so the response of a connection or element cannot be considered in isolation.



a) b)

Figure 1: a) A PTT frame being erected (Courtesy of University of Canterbury), b) qualitative response of a PTT frame under lateral loading.

New Zealand structural designers are accustomed to using a Reduced Cross-Section Method (RCSM) to assess the capacity of heavy timber elements to Standard Fire exposures, as

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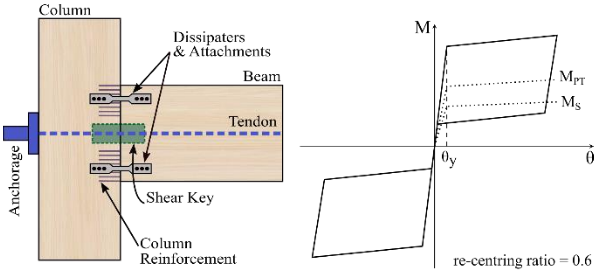
detailed in NZS 3603 Timber Structures Standard [10] and in AS/NZS 1720.4 [11]. In this approach the cross-section is reduced by a char thickness which is the product of a constant charring rate and the duration of Standard Fire exposure. In AS/NZS 1720.4 a further “zero-strength layer” of 7 mm is removed. The capacity of the element at the end of this exposure is evaluated at the ambient design strength of this reduced cross-section. This approach may be acceptable for the midsection of heavy timber beams and columns where the expected thermal field is established. However, at the connections, conduction from steel components, the different exposed surfaces, contact surfaces and mix of components leads to a thermal field that is different to that in the mid-section of the elements. The reduced cross-section method grossly simplifies the thermal field and mechanical response of timber elements and does not facilitate considering the real thermal and mechanical response that may be at the connection.



a) b)

Figure 2: a) Schematic of a PTT beam-column connection with an internal shear key and external dissipaters, b) Typical Cyclic Moment-Rotation curve of a PTT connection with dissipaters.

PTT connections that are part of the gravity load path must be designed to provide adequate performance under both ambient and fire actions. Even if PTT frames are designed only for seismic loads, failure of this system could compromise the fire performance of the gravity structure. When making structural detailing choices for connections, designers consider large displacements and plastic deformations (and in New Zealand particularly with ambient seismic response in mind). However, in a fire, not only do large displacements and plastic deformations occur but also materials reduce in strength and stiffness, and in the case of timber, material is lost as well. If these events are not suitably considered during detailing, a connection can be detailed for excellent seismic performance but may perform poorly in fire.

In this paper the design and performance of seismically detailed PTT beam-column connections that support gravity loads when exposed to fire are investigated. Post-flashover fires, Fire Resistance and structural actions for structural fire design are reviewed for the benefit of primarily seismic engineers and practitioners. Since PTT beam-column connections are made of many components of different materials that will respond differently to fire, firstly the response of each connection component is presented. The response of a PTT connection to fire is assessed using the existing methods designers are familiar with: the Modified Monolithic Beam Analogy and Reduced Cross-Section Method. The response of two seismically detailed connections predicted using this approach are compared to the expected behaviour from a first-principles analysis. The damage to the connections, which degrades their performance, is explored with thermal finite element analyses.

STRUCTURAL ACTIONS UNDER FIRE CONDITIONS

Post-flashover fires (fires where all the contents of the compartment are burning and the rate at which heat is released inside the compartment is governed by how much air can enter the compartment through the openings) are a severe design action for structures. Although the loads on structures in the fire case are generally considered low (usually similar to SLS actions), the thermal exposure from the fire results in significantly reduced structural resistance. Material strength and elastic modulus are significantly lower at elevated temperatures expected in fire [12]. The reduced material strengths and elasticity means deformations are much greater than at ambient, and the utilisation ratio (demand/capacity) of elements in the fire condition is greater than at ambient. The fact that after being designed for earthquake actions, structures may require a form of passive fire protection to maintain load bearing capacity in the fire load case, illustrates the destructive potential of a fire.

The Fire Resistance of an element is the ability of that element to maintain its functions (load bearing function in the case of a structural element) under defined conditions and fire exposure. Standard Fire Resistance is the Fire Resistance where the defined Fire Exposure is the ISO 834 Standard Fire Curve [13] (see Figure 3). The Standard Fire curve is always increasing in temperature and because it has no “end” it is always talked about in the context of an exposure time. Standard Fire Resistance is determined by standardised testing in which a representative loaded specimen is exposed to the Standard Fire. The time since the start of the test at which the specimen fails to maintain its functional requirements is assigned to the element as its Standard Fire Resistance, e.g. 48 minutes.

To decrease the risk of collapse in fire, building codes have implemented minimum Required Fire Resistance for structural elements. The Standard Fire Resistance of structural elements should meet or exceed the period of Required Fire Resistance. The compliance documents to the New Zealand Building Code [14,15] state the Required Fire Resistance as fire resistance ratings, which are durations of Standard Fire exposures in 30-minute divisions. In typical New Zealand practice the structural engineer demonstrates that the structural elements, under appropriate loading, have a Standard Fire Resistance that meets the Required Fire Resistance determined according to prescriptive rules for fire resistance ratings by the Fire Safety engineer. The Standard Fire Resistance can be demonstrated by standard test or for simple scenarios, the structural design standards provide methods for calculating the expected Standard Fire Resistance if it was tested. Often a proprietary protective product is required to increase the Standard Fire Resistance above the Required Fire Resistance threshold.

Designing structures using Standard Fire Resistances assumes the fire in the compartment will be the Standard Fire. The fires that will actually occur in building compartments (real fires) are fundamentally different to the Standard Fire that is commonly used in design. In real fires, there is a finite amount of fuel in the compartment. Once this is consumed the temperature in the compartment will begin to decrease. Therefore, real fires can be split into two phases, a heating phase where combustion of the fuel releases thermal energy within the compartment and a cooling phase where the compartment cools through radiation and convection to the outside environment. Figure 3 shows an example real fire in comparison to the Standard Fire and the two distinct phases of the real fire. The temperature of a real fire depends on the compartment geometry, the openings which limit air supplied to the fire, the compartment linings which absorb thermal energy and the fuel load which determines the duration of burning [12]. The Standard Fire is different from real fires in two important ways: firstly, the Standard Fire is prescriptively always the same, and does not consider the unique aspects of each compartment, and secondly it does not have a cooling phase, instead it is always increasing in temperature. The fire that will occur in a compartment is unique to that compartment and can vary significantly between compartments, even within the same building.

Although Standard Fire tests are not necessarily representative of actual fires, they are used globally as a benchmark for comparison of fire performance between structural elements. However, designing a structure using Standard Fire Resistances (which assume a Standard Fire exposure) does not give any guarantee of performance in a real fire, and is at best an indication of relative response of the structure. This is because the Standard Fire does not represent the real fire that could occur in the compartment and the structural response is dependent on the thermal exposure. The fire that actually occurs in a compartment can be more severe than the Standard Fire exposure, increasing the likelihood of failure. The fire resistance rating durations used in design are only loosely based on the occupancy type and have no reference to the real fire duration or temperatures in the compartment. Furthermore, since Standard Fire Resistances are evaluated on an element basis, an assembly of elements each with a Standard Fire Resistance does not necessarily perform similarly because interactions with the rest of the structure are not accounted for, e.g. restraint of other elements to expansion of a heated element.

The expected fire of a compartment depends on several parameters that are highly variable and may change over the life of the building and the duration of the fire. Therefore, there is significant variability in the fire that will occur. Just as a structure is assessed under a suite of ground motions to ensure a robust response in a time history analysis, the structural response in fire should be assessed against a suite of fires. A simple and common case is using a range of ventilation ratios to construct parametric fire exposures. A robust structure which has been shown to perform against a suite of fires gives more confidence in its actual performance than designing against a single fire, much less a Standard Fire exposure.

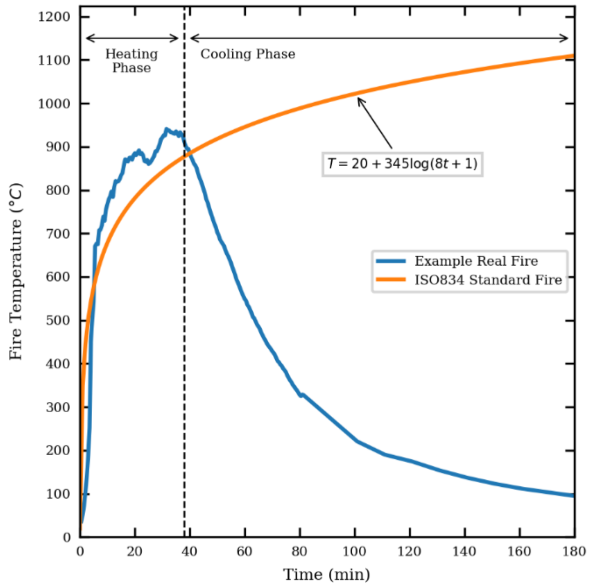


Figure 3: Time-Temperature curves for ISO834 Standard Fire [13] and a real fire (Cardington Office Test) [16].

PTT COMPONENT RESPONSE IN FIRE

A post-tensioned timber frame is an assembly of large cross-section timber beams and columns linked together by post-tensioned connections. Therefore, the behaviour of these connections in fire will govern the frame response in fire.

In the fire scenario, PTT beam-column connections on the gravity load-path are considered to have a combined moment and shear loading. The applied moment is resisted by a force-couple from the tendon and beam-column contact area. However, as the timber element cross-section and tendon force reduce from fire exposure, a gap opens to engage the dissipaters to provide additional moment capacity. The applied shear load is transferred into the column typically by the beam bearing on a shear key. The overall response of a connection is the combination of the moment resisting and shear resisting mechanisms and the interactions between the two mechanisms. The response of each mechanism and their interactions is complex, and so the moment and shear responses should be analysed separately first, before their interactions can be examined.

Although a Standard Fire will not occur in reality, it is used as the fire exposure for the analysis in this paper to remove the complexity of the influence of fire exposures on connection response and is known to practitioners. Research validating design methods for the thermal field and charring of timber elements under non-Standard Fire exposures is ongoing, although there is evidence that greater char thicknesses are observed in fires hotter than the Standard Fire [17,18].

There are six components to a post-tensioned timber connection: the timber beam, column, tendon, dissipaters, shear key and transverse column reinforcement, as shown in Figure 2a. The response of each component in fire must be understood in order to understand the overall connection response. The thermal and mechanical response of each component to fire exposure is unique as discussed in the subsequent sections.

Timber Elements

When a timber element is exposed to fire a distinctive thermal field is established in the element. The low thermal conductivity and enthalpy of vaporization of the moisture in wood creates a steep thermal gradient in the wood. This is complemented by the thermal decomposition of wood which starts at approximately 200 °C [19] and is completed with the formation of char at approximately 300 °C. As the char layer forms, it acts as insulation to the non-charred portion of the element. Therefore, the thermal wave resulting from the fire propagates through the member slowly and it has been observed that during the heating phase of a fire the “core” of a heavy timber element exposed to fire remains close to the ambient temperature [20,21]. The strength and elastic modulus of wood decrease with temperature until char is formed which is considered to have no strength or elasticity. Eurocode 5 Part 1-2 [22] contains bilinear strength and elastic modulus relationships with temperature for timber elements exposed to the Standard Fire. Figure 4 shows these relationships mapped onto the thermal field of a 200 mm thick timber slab exposed to the Standard Fire for 30 and 60 minutes respectively, to illustrate how these properties change with depth below the 300 °C isotherm. This thermal and mechanical response of timber forms the basis of the Residual Cross-Section Method codified in timber codes and standards worldwide (e.g. NZS 3603, AS/NZS 1720.4 & Eurocode 5). In the Residual Cross-Section Method, the original cross-section is reduced to an effective load bearing cross-section. Firstly, the portion of the section which is expected to char is removed; this may consider corner rounding explicitly or use a larger char depth with a rectangular section. Secondly, a further layer of wood, which is too hot to have significant strength, is considered either by evaluating the thermal gradient and reduced material properties explicitly or by removing a “zero-strength layer” and assigning ambient material properties to the remainder. The implementation in NZS 3603 does not consider a zero-strength layer but explicitly includes corner rounding while the implementation in AS/NZS 1720.4 uses a notional charring rate and a zero-strength layer of 7 mm to calculate a residual rectangular cross-section. Figure 4 also shows the material strength distribution considered by the AS/NZS 1720.4 implementation of the RCSM (i.e. zero outside the “zero-strength” layer and 100% in the remainder of the cross-section).

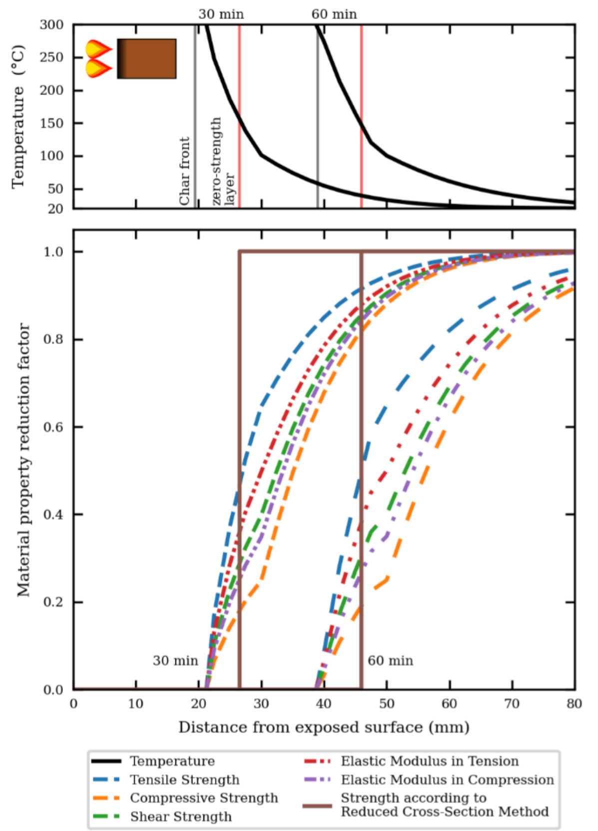


Figure 4: Simulated temperature, strength and elastic modulus distributions through a wood slab after 30 and 60 minutes of exposure to the Standard Fire in comparison to the AS/NZS 1720.4 reduced cross-section method (char rate of 0.65 mm/min and 7 mm zero strength layer).

The Reduced Cross-Section Method assumes the particular characteristic steep and slowly propagating thermal gradient that develops when timber is exposed to the Standard Fire. At connections with partially exposed steel plates or fasteners, the expected thermal field is disrupted as the steel components conduct heat from the exposed surfaces into the interior of the timber. This creates localised accelerated charring around these components and violates the thermal field assumed by the Reduced Cross-Section Method. This method also assumes the timber remains in one piece once it has charred. For engineered wood products, adhesives which achieve this should be used or adhesive failure should be accounted for in design. The Reduced Cross-Section Method should not be used where the assumed thermal field will not occur, without modification to account for the thermal field that will be established.

In the decay phase of the fire, although the peak compartment temperature may have passed, the thermal wave continues to propagate into the core of the section, similar to concrete elements. In experiments by Kinjo et al., the core of the beam, already reduced by charring, reached temperatures between 100 and 150 °C [20]. According to Eurocode 5 Part 1-2 at 100 °C, the strength and elasticity in tension are approximately 65% and 50% of that at ambient respectively [22].

Tendons

Tendons, which are typically cold worked strands or quenched and tempered bars, provide over half (typically 60-80%) of the moment resistance in seismic frames to ensure re-centring and all the moment resistance in gravity only frames (which have no need for dissipative devices). Therefore, to maintain the majority of moment resistance of connections, heat transfer to the tendon should be limited to prevent relaxation thereby retaining the tensioning force and preserving the strength of the tendon and to avoid connection failure. Eurocode 2 Part 1-2 [23] includes strength and elastic modulus relationships for tendon materials.

There are three possible ways for the tendon to be heated by the fire:

1. **Fire exposure to the anchorage**
   The primary method of heat transfer to the tendon will be through the anchorage. Strands use barrels and wedges for anchorages whereas threaded bars use nuts and both use bearing plates to prevent crushing of the column. If these anchorages are exposed directly to the fire they will heat up quickly and conduct heat along the tendon. Furthermore, charring of the timber under the anchorage will result in shortening of the tendon if the anchorage is not otherwise restrained.
2. **Fire exposure directly to the tendon**
   Tendons are normally placed inside the timber elements where they are separated from the fire by a substantial thickness of timber. The tendon will only experience an increase in temperature once heat has been conducted through the timber or by convection across the duct to the tendon. Many PTT structures have the tendon placed in a small duct through a solid timber section with more than 120 mm of wood on each side of the duct. In this case heat transfer to the tendon through the beam will not be significant.
3. **Fire exposure to tendon through a gap opening**
   If a gap opens at the beam-column interface to meet the moment demand on the connection, the tendon could be exposed to the fire through this gap. Eurocode 5 states that fire should be considered to exploit gaps between timber elements greater than 2 mm [22]. Aarnio in [21] found that charring on internal gap surfaces only occurred when the gap was larger than 5 mm. Kordina & Meyer-Ottens in [21] limited gap between timber members in connections to 3 mm and achieved 30 minutes of fire resistance. Therefore, if the gap at the beam interface is expected to be greater than 2mm, exposure of the tendon to fire should be investigated, although the heated length of tendon may be small compared to the full length of the tendon.

When the tendon increases in temperature, the tendon force reduces through a combination of up to three components:

1. Thermal elongation of the heated portion of the tendon,
2. Elastic relaxation of the tendon from the elastic modulus of the tendon, i.e. variation of the mechanical properties, and the whole tendon relaxing as the tendon force decreases, and
3. Plastic strains developing in the tendon, if the temperature at a point results in the tendon yield strength decreasing the stress in the tendon.

To investigate the response of strands and bars, the relaxation of D36 high strength bar (cold worked, 1030 MPa) and 4x15.7 mm strands (cold worked, 1830 MPa) when the anchorages are fixed but one anchorage is exposed to the Standard Fire was simulated. The tendons, based on a three bay (6 m bays) PTT frame, are 20 m long and each tendon was initially stressed to achieve a 250 kN tendon force (24% of their ultimate strength), which requires the bar and strands to stretch 27 mm and 45 mm respectively. The finite element simulation was conducted in Abaqus/Standard using thermal and mechanical properties from EN 1992-1-2 [23]. Figure 5 shows the thermal elongation, elastic and inelastic contributions to total tendon relaxation and the temperature of the tendon at the point of greatest plastic strain. This occurs where the strand/bar connects to the anchorage head/nut. Both the bar and the strand heat up at similar rates and their tendon forces remain relatively constant. Thermal relaxation of the tendon dominates until just before failure when large plastic strains are observed as the yield strength at the hottest portion of the bar falls below the tendon force. Relaxation due to the reduction in elastic modulus of the heated portion of the bar is small relative to the other sources of relaxation.

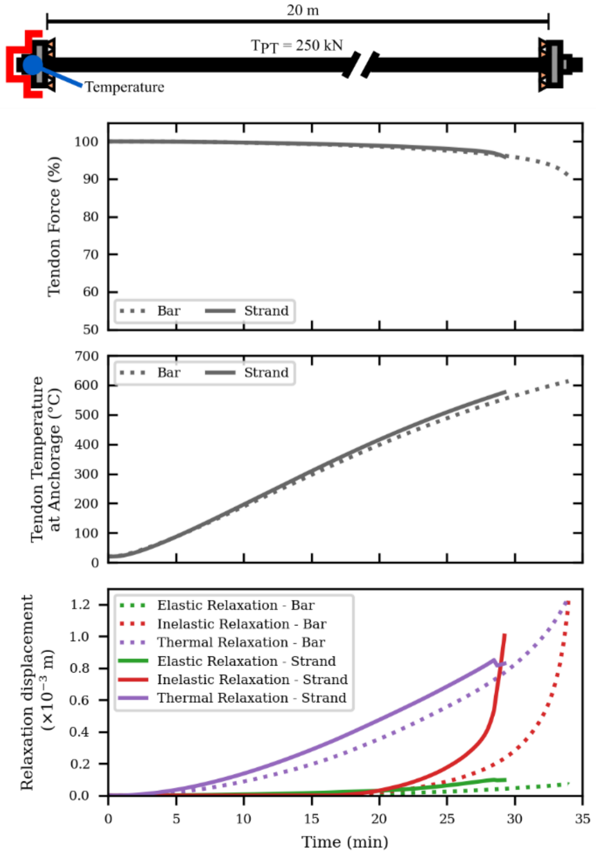


Figure 5: Simulated response of strand and bar when one anchorage is exposed to the Standard Fire.

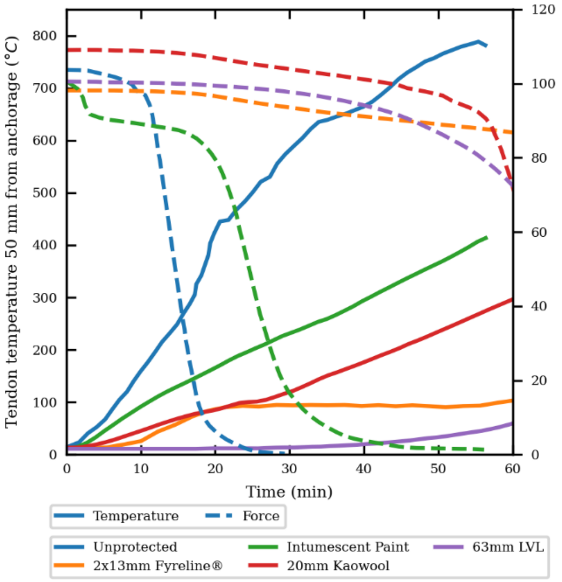


Figure 6: Tendon temperatures and forces for various anchorage protection schemes (from [24]).

Spellman [24] conducted fire tests on 1.5 m long post-tensioned timber beams stressed to approximately 100 kN with one half of the beam, including the anchorage, exposed to fire. Several different protection materials were applied to the exposed anchorage, with the anchorage left unprotected for one test. Temperatures (measured 50 mm from anchorage) and tendon forces are shown in Figure 6. The unprotected anchorage maintained 95% of the initial tendon force for approximately 10 minutes, after which the tendon force decreased to almost zero over five minutes. Intumescent paint provided some insulation to the anchorage, but delayed the rapid tendon force decrease by less than 10 minutes. The other protection options offered significantly better performance; two layers of 13 mm GIB Fyreline® maintained 88% of the initial tendon force for 60 minutes of fire exposure.

Dissipaters and Attachments

Dissipaters, typically from mild steel, can be installed inside timber elements, usually with epoxy, or attached externally, where they can be replaced easily after a seismic event. The moment contribution from dissipaters depends on the strength and stiffness of the dissipaters and the attachments to the timber elements. Internal dissipaters by their nature are insulated from direct fire exposure by the timber and their response depends on the thermal field that develops in the timber around the dissipater and the strength of the attachment (e.g. epoxy) and embedded timber at elevated temperatures. External dissipaters and attachments are directly exposed to the fire unless they are protected and their response is contingent on the thermal field that develops in the dissipaters and attachments and their corresponding reduction in strength and stiffness. Dissipaters are typically made from mild steel; NZS 3404 [25] and Eurocode 3 Part 1-2 [26] contain strength and elastic modulus reduction factors at elevated temperatures for mild steel. Using a lumped mass thermal analysis [26], the temperature, strength and stiffness of a typical grooved mild steel dissipater (F/V of 146 m^{-1}) is shown in Figure 7.

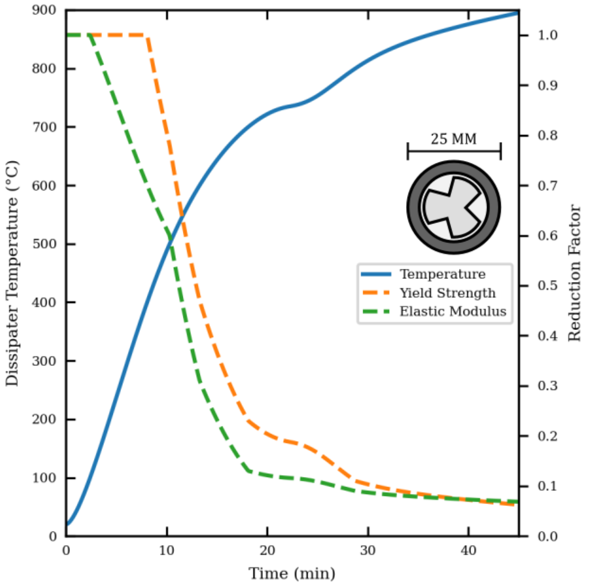


Figure 7: Temperature, yield strength and elastic modulus of grooved dissipaters exposed to the Standard Fire.

To meet seismic performance requirements, the dissipater attachments need to be stiff so the dissipaters are activated at the correct drift and strong enough for the dissipater to develop plastic strain. External dissipaters are typically attached with groups of steel fasteners such as timber rivets or screws. Exposed fastener surfaces conduct heat down the fastener, resulting in reduced embedment strength and accelerated charring around the fasteners, and as the timber surface chars and the eccentricity of the load applied on the fasteners increases.

Shear Key

Although friction at the beam-column interface may be able to provide substantial shear resistance, shear keys are provided to support beams during construction before tendon stressing has occurred and to maintain stability in the unlikely case the tendon force is lost. The shear key is especially important when parts of the tendon or anchorage are exposed to the fire since the beam will become simply supported and vertical support is provided solely by the shear key. Both internal and external shear keys have been used in post-tensioned timber buildings. Internal shear keys are not directly exposed to the fire so their performance depends on the slow-moving thermal field that develops in the timber member. In contrast, external shear keys are exposed directly to the fire. Fasteners used to attach external shear keys will also reduce in strength and cause localised charring.

Transverse Column Reinforcement

The joint panel zone of columns in post-tensioned timber frames is often reinforced to:

1. Control long-term creep as the column is subject to long-term stresses perpendicular to the grain.
2. Prevent crushing perpendicular to the grain of the column when a gap opens and the contact area between the beam and column is reduced.

Several different solutions to these challenges exist including screw reinforcement of the column, fabricating columns with locally rotated grain, external steel jackets around the column and inclusion of steel compression members. The exposure to each of these and their response will be different. The type of reinforcement, if any, will change the interface rotational stiffness in a fire and affect the thermal field within the column. The response of each type of reinforcement is outside the scope of this paper, but the rotational stiffness of reinforcement will reduce in proportion to heat absorbed from the fire.

Protection of Steel Components

Post-tensioned timber connections contain exposed steel components such as anchorages, plates and fasteners. These elements have a high area to mass ratio, and heat up similarly to the exposed dissipaters when exposed to fire. Most importantly, these components are capacity protected to guarantee the dissipation and re-centring capabilities. The lumped mass approximation of a grooved dissipater (see Figure 7) shows that after 30 minutes of Standard Fire exposure it has a temperature of 800 °C and has 10% of its ambient strength and elasticity, and at 45 minutes it has negligible strength and elasticity. For a Required Fire Resistance of 30-minutes, these unprotected steel elements would have lost significant strength. Therefore, the robustness of such unprotected exposed steel elements should be seriously considered.

In New Zealand, the most popular method of protecting steel members is by applying intumescent paint, which when heated expands up to 50 times its original thickness, forming a layer of light char which insulates the steel. When intumescent paint is applied to steel members to increase their Fire Resistance, the paint is applied to the member in sufficient thickness so that the temperature of the steel member does not exceed the failure temperature of the element for that duration of Standard Fire exposure [27]. The failure temperature of steel members is typically in the range of 500-700 °C, which is much greater than the charring temperature of timber at 300 °C. So, a steel element protected with typical intumescent paint thicknesses will still result in accelerated charring to adjacent timber elements. Required intumescent paint thickness are determined from Standard Fire tests on steel I-section members. Steel plates and elements in connections have much less mass than these members, and so will heat up quickly. Carling [21] warns that the extrapolation of results from larger steel sections to small pieces of steel with masses less than 50 kg may not be valid unless confirmed with tests.

Peng [28], Lau [29] and Frangi et al. [30] tested timber connections with steel components protected with intumescent paint and found that thick applications could increase the failure time from 10 to 20 minutes. Spellman [24] found that an anchorage protected with intumescent paint maintained 95% of the tendon force for 23 minutes of fire exposure compared to an unprotected anchorage which maintained 95% of the tendon force for 10 minutes. The char layer produced by the expansion of the intumescent paint is delicate and has been observed to be damaged due to deformations at connections [28]. Use of intumescent paint to protect small steel elements in timber connections should be supported by tests that demonstrate adequate performance of the whole connection will be obtained.

RESPONSE OF POST-TENSIONED TIMBER BEAM-COLUMN CONNECTIONS IN FIRE

Description of Connection Performance in Fire

Under gravity loads in fire PTT beam-column connections are subject to a combined moment and shear load, where the applied moment is resisted by a combination of force-couples from the dissipaters, tendon force and compression of the beam against the column and the shear force is typically resisted by bearing on a shear key. Although the moment demand on the connection is constant, these force-couples change in magnitude and location as the duration of fire exposure and connection rotation increase. The development of moment resistance during exposure to fire can be described in three stages (Figure 8).

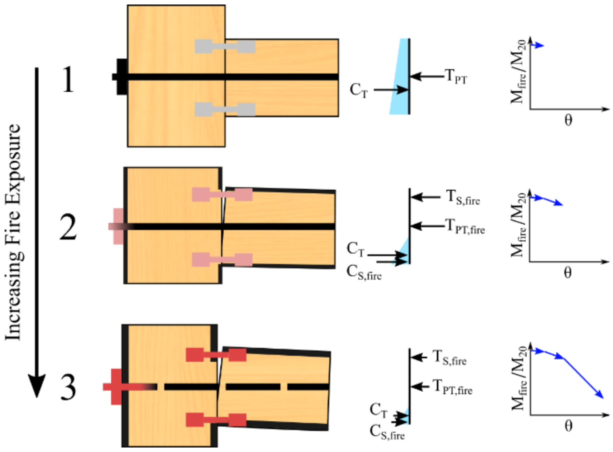


Figure 8: Stages of moment resistance of PTT connection in fire.

1. In the first minutes of exposure, the moment demand is less than the decompression moment. The decompression moment is provided by the force-couple between the tendon force, T_{PT}, and the compression block, C_t.
2. As the tendon force decreases due to the fire exposure ($T_{PT,fire}$), the decompression moment is exceeded, and a gap opens. The tendon is stretched (offsetting some of the reduction in tendon force), the dissipaters are engaged to provide an additional force-couple ($T_{S,fire}$ and $C_{S,fire}$) and the compression block depth decreases.

3. As the tendon and dissipaters heat up, their strength, stiffness and moment contributions decrease. The rotation of the connection increases to maintain dissipater moment resistance by increasing the tendon and dissipater stretch and decreasing the compression block depth. The moment capacity of the connection will eventually be exceeded when dissipaters and the tendon are not able to provide sufficient moment resistance.

Where shear resistance provided by a shear key, it will decrease in proportion to the reduction of the bearing area (which decreases with the reduced cross-section) and the bearing strength of the timber (which depends on the temperature of the timber bearing on the shear key). For internal shear keys, these reductions will occur when the thermal wave reaches the shear key, although splitting of the beam may occur first in this case. In the case of a shear key that is exposed to the fire, the exposed timber surfaces will char, reducing the bearing contact area. If the external shear key is metallic, the timber in contact the shear key will also char which will be crushed as the beam displaces downwards to maintain bearing. This downwards displacement of the beam will have further effects on the connection, e.g. inducing additional deformations in other components and increasing connection rotation.

Basic Moment-Rotation Relationship

To establish the moment-rotation relationship of these connections in fire, designers use the current tools available to them. Accounting for charring of the timber elements using a Reduced Cross-Section Method and using the MMBA to predict the moment-rotation relationship for rocking connections. The MMBA for rocking connections with rectangular sections and gravity loads was developed and validated by van Beerschoten [31].

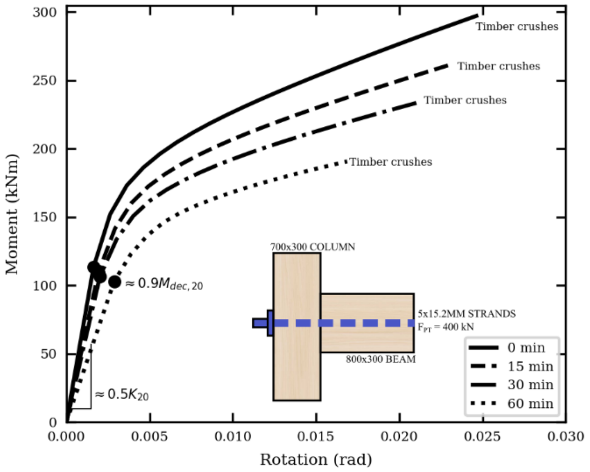


Figure 9: Analytical moment-rotation of a post-tensioned beam column connection under Standard Fire exposure by combining MMBA and RCSM.

The moment-rotation response of a basic PTT beam-column connection without dissipaters for various durations of Standard Fire exposure was predicted using this approach that designers are expected to take. The connection consisted of a 700x300 LVL column, a 800x300 mm beam and a 5x15.2 mm strand tendon tensioned to 400 kN that was fully protected from the effects of fire. The reduced cross-sections of the beam and column were determined based on a nominal charring rate of 0.65 mm/min and a zero-strength layer of 7 mm as per AS/NZS 1720.4 [11] and the initial tendon force was maintained since the tendon was fully protected. Figure 9 presents the moment-rotation relationships for this connection. As the exposure duration increases the initial stiffness and moment capacity for the same rotation decreases. After 60 minutes the decompression moment decreased by 10% and the initial stiffness decreased by almost 50%. The ultimate strength was governed by timber crushing at the extreme compression fibre although the ultimate strength and rotation decreased for longer exposures.

The MMBA is a function only of the geometric and material properties of the beam, column and tendon, so this combined RCSM and MMBA approach does not consider the specific detailing of the connection. When exposed to fire, the specific connection detailing will initiate localised damage which will degrade the connection performance. Fire tests of different types of timber connections show that detailing of timber connections can greatly influence the performance of a connection [28,32–34]. Nor does this method account for the localised change in geometry, strength and stiffness at the connection from fire exposure, e.g. corner rounding, heating of the beam-column interface, etc.

EFFECT OF DETAILING ON CONNECTION RESPONSE

To investigate the shortcomings of the combined MMBA and RCSM approach (called MMBA + RCSM here) and to observe what localised damage at the connection can look like, connection response evaluated using this method was compared to first principles expectations of connection response and a numerical thermal analysis.

Two post-tensioned timber beam-column connections that represent a range of seismic detailing implemented in New Zealand were analysed as a typical designer would be expected to do in practice, using the MMBA + RCSM approach and including the decrease of post-tension force and dissipater strength and stiffness. The expected localised damage from fire exposure at the connections, which impaired overall (joint) connection performance, was assessed from a first principles basis of heat transfer and structural mechanics. A thermal finite element analysis was undertaken to confirm the thermal assessment and investigate some examples of localised damage that can occur at connections.

Methodology

The moment demand on each connection was based on the SLS gravity load case. Typically the SLS connection demand is 75 to 90% of the decompression moment, because the connection should not decompress under serviceability loads and an allowance for long term decrease in tendon force is made depending on the specific connection detailing to control creep [1].

Each connection was exposed to the Standard Fire since in New Zealand practice, structural design for the fire scenario is done using fire resistance ratings (which are durations of Standard Fire exposure). Connections at both ends of the frame were in the one compartment, so for the purposes of tendon force reductions, the effect of two exposed and unprotected anchorages was considered. Although the limitations of the Standard Fire discussed previously apply, the Standard Fire is known to practitioners and the thermal field in timber is well known under the Standard Fire exposure.

For each time-step in the fire exposure, the following methodology was used:

1. The reduced cross-section of the timber elements was modelled using RCSM with a char rate of 0.65 mm/min and a zero-strength layer of 7 mm (AS/NZS 1720.4 [11]) that is established over 20 minutes [22].

2. The reduced tendon force was determined from a simplified time-tendon relationship in which the tendon force decreased by 10% during the first 30 minutes then decreased to zero over 5 minutes. This was taken from the analysis presented previously in this paper (see Figure 5) and applied to both anchorages. No additional stretching of the tendon from gap opening was included beyond this time because the reduction in tendon force comes from inelastic deformations.
3. The interface decompression rotational stiffness from the compression of the column due to the post-tension force was calculated using multiple linear springs as described by van Beerschoten [31]. The rotational stiffness decreased as the beam cross-section charred.
4. The temperature of the unprotected dissipaters was modelled using the Eurocode 3 lumped mass approach. The yield strength and elastic modulus at that temperature were adopted from the reduction factors in EN 1993-1-2 [26]. This model overestimated the dissipater temperature because it neglected heat being conducted away.
5. The MMBA procedure for gravity frames with rectangular element under gravity loads [31] was used to establish the relationship between connection rotation (including any gap opening) and the tendon and dissipater stretch. For a given gap angle the moment induced in the connection was calculated.

The thermal and mechanical first principles analysis considered the thermal field that is established in timber when it is exposed to fire, conduction along metallic components and into the surrounding timber and the superposition of where components are close. The strength and stiffness of timber decreases until it is charred when it has no structurally significant strength or stiffness.

Three-dimensional thermal finite element analyses were undertaken for each connection in Abaqus [35] to simply demonstrate what unique connection damage can occur and how this damage would affect a MMBA analysis. The components of the connection were modelled as separate parts (5 mm element size) with perfect thermal contact between all the contact surfaces. Material properties were taken from EN 1993-1-2 [26] and EN 1995-1-2 [22] for steel and timber (density of 570 kg/m^3 for LVL13 [36]) respectively. A Standard Fire exposure was applied to all external surfaces of the connection with a convective coefficient of 25 W/m^{2}K [37] and the emissivity of steel and timber surfaces were 0.7 [26] and 0.8 [22] respectively.

Connection 1

Connection 1 (Figure 10a) consisted of an 800x315 mm LVL beam and column, a 7x15.3mm strand tendon stressed to 60% of its ultimate tensile strength. Four external necked dissipaters (A= 314 mm^2) were fixed to the top and bottom of the beam with rods through the beam and column and force is transferred between the dissipaters and the timber beam with "shear stubs" (as annotated Figure 10a). The internal shear key was an epoxied square hollow section which the tendon anchorage also bears on.

The rotation and moment capacities of this connection following the methodology above is presented in Figure 10b. The decompression moment, which is a function of beam height and post-tensioning force, decreased by 15% over the first 30 minutes of exposure, 60% of this decrease was from the reducing tendon force and 40% from the reduction in beam height. The rotations increased slightly during the first 30 minutes as the decompression rotational stiffness decreased by 30%. There is a slight bilinear relationship due to the zero-strength layer growing during the first 20 minutes. The connection decompressed at 21 minutes, and the dissipaters were slightly engaged to provide a small amount (< 3%) of additional moment resistance. After 30 minutes when the tendon force decreases accelerated, the connection rotations became larger to stretch the hot dissipaters further and increase their contribution to moment resistance. At 34 minutes the moment demand exceeded its moment capacity. The yield moment resistance decreased at a slightly greater rate than the ultimate moment resistance because the dissipaters have a greater contribution to the yield moment than to ultimate moment resistance where tendon contribution is greater due to the tendon stretching as gaps open.

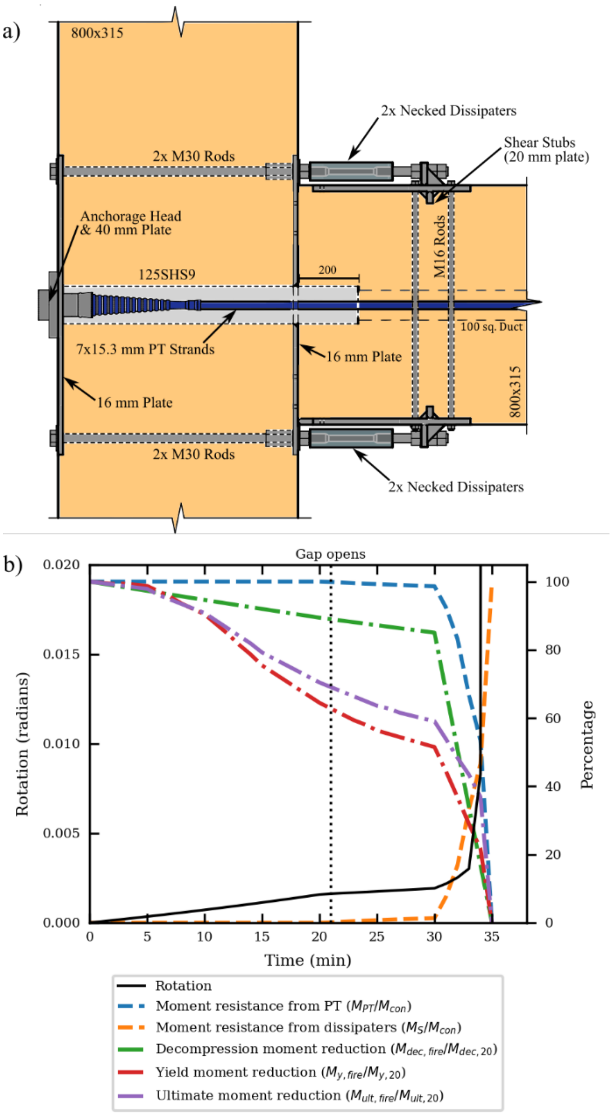


Figure 10: a) Elevation of Connection 1, b) Connection 1 response to Standard Fire estimated using MMBA + RCSM.

The first principles consideration of the actual thermal and mechanical response of the connection components to maintain a load bearing connection, identified three cases where connection performance will be degraded (shown indicatively in Figure 11a):

1. As the beam chars, the "shear stubs" engage less sound timber. The shear keys are 40 mm deep and exposed to the fire, this transfer mechanism will be lost while the dissipaters still have some strength.
2. Dissipater forces are transferred into the column through a steel plate bearing on the column. As the timber surface beneath these plates chars, the plates will bend to maintain bearing contact with sound timber.

3. Conduction through the steel plate at the interface will heat and soften the surrounding wood and initiate corner rounding. The softer interface wood and reduced contact area from corner rounding will result in a softer connection response.

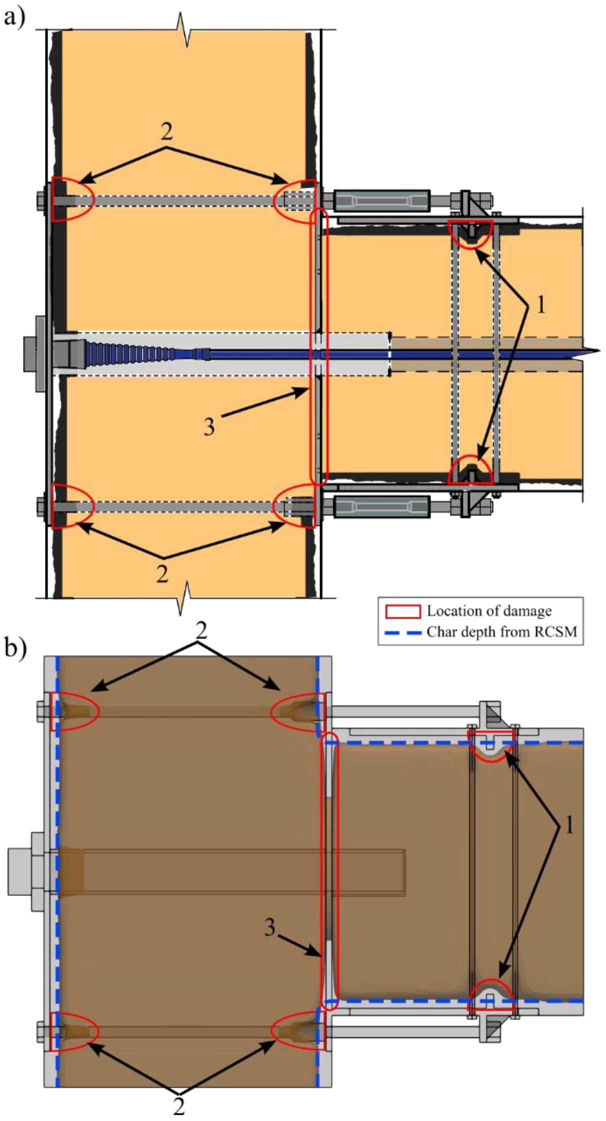


Figure 11: a) Locations of damage when Connection 1 is exposed to fire, b) Render of the 300 °C isotherm of Connection 1 after 60 minutes of Standard Fire exposure from thermal FEA (material hotter than 300 °C is shaded) compared to char depth from RCSM (0.65 mm/min).

Shear resistance at the connection is provided by a SHS that extends through the column into the beam. Dowel action is used to transfer the shear force from the beam into the column. Failure results from the formation of a longitudinal crack, propagating from the end of the beam, at the location of the SHS as the section of the beam is reduced.

A thermal FEA of this connection was conducted to corroborate the expected thermal field in the first principles assessment. The 300 °C isotherm, which is closely correlated with the char front, of the connection after 60 minutes of Standard Fire exposure is rendered in Figure 11b (material hotter than 300 °C is shaded) along with annotations marking the three cases of connection degradation identified previously. This isotherm confirmed the charring around the "shear stubs" and thermal penetration along the bolts and at the beam-column interface identified in the first principles assessment, which deteriorate connection response. The 300 °C isotherm was compared to the char thickness determined from the RCSM with a charring rate of 0.65 mm/min (blue dashed lines in Figure 11b). The char thickness predicted by the RCSM is appropriate for timber directly exposed to the fire, where the assumptions of the RCSM are valid. However, around partially exposed metallic items, where the RCSM assumptions are invalid, the predicted char thickness is underpredicted. Determining mechanical response of this connection based on the RCSM would not reflect the actual response of the connection because the RCSM doesn't capture the localised connection damage. For example, as the wood around the "shear stubs" chars, the embedment strength is reduced and decreases to zero when the "stub" only engages char.

Connection 2

Connection 2 (Figure 11a) had 600 x 315 mm LVL column and beam, a 6x 15.7 mm strand tendon tensioned to 65% UTS and four external necked dissipaters (A = 380 mm^2) fixed with timber rivets to the sides of beam and to a steel jacket fitted around the column. The beam was supported by a rubber pad on a stiffened steel shear key bolted through the column with screw reinforcing behind the shear key.

The rotation and moment capacities of Connection 2 exposed to the Standard Fire evaluated using the MMBA + RCSM method is presented in Figure 12b. The rotation of Connection 2 during the fire exposure was similar to Connection 1, although it was slightly greater because Connection 2 has less interface rotational stiffness as a result of the beam being 25% shallower. Connection 2 decompressed at 19 minutes which was 2 minutes earlier than Connection 1. The decompression, yield and moment responses were like that of Connection 1, as the basic connection mechanics were the same. Although, the moment contribution from the dissipaters was less for Connection 2 because the dissipaters were 21% larger than in Connection 1, the lever arm is less than half of that in Connection 1, so the total moment resistance provided by the dissipaters is less. A smaller decrease in the yield moment of Connection 2 was observed for the same reason.

The first principles considerations of this connection's response identified three potential causes of impaired performance (shown indicatively in Figure 13a):

1. Closely spaced timber rivets fix the dissipater attachment to the beam. When exposed to fire, superimposed thermal fields will develop in the timber from conduction along the rivets, reducing their embedment strength and stiffness, and causing localised charring (similar to that observed for nails in [30]). The load transfer mechanism from the dissipater attachment will be reduced and will likely fail earlier than the dissipaters themselves.
2. The steel jacket around the column is tight at ambient, but it will develop slack as the column surface recedes due to charring, resulting in increased deformations to the jacket and the connection to maintain the moment resistance.
3. The beam bears on the rubber pad and steel shear key which is exposed to the fire. As the rubber pad burns and the timber bearing surface chars, a downwards displacement will occur as the char crushes to maintain bearing on the shear key. This vertical displacement of the beam will induce additional deformations in the connection and create secondary effects reducing moment capacity.
4. Conduction through the steel plate at the interface similar to that noted for Connection 1.

The shear forces are transferred from the beam to the column through the stiffened bolted steel shear key. As the surface of the column recedes, the bolts become loose. Screw reinforcing behind the angle acts to prevent rotation of the angle until they buckle from the increasing unsupported length and reduced elastic modulus.

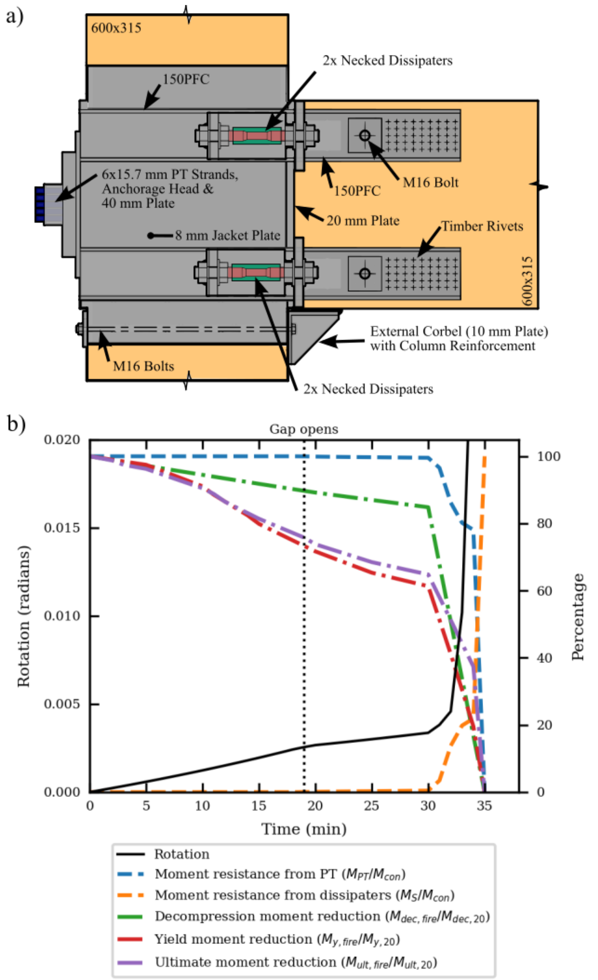


Figure 12: Elevation of Connection 2, b) Connection 2 response to Standard Fire estimated using MMBA + RCSM.

The 300 °C isotherm of the connection after 60 minutes of Standard Fire exposure is rendered in Figure 13b (material hotter than 300 °C is not shown) along with annotations marking the three causes of impaired connection performance identified previously. Charring underneath the steel jacket plate and on the shear key, conduction at the beam-column interface and accelerated charring around the dissipater attachment fasteners is evident from the 300 °C isotherm from the finite element analysis. This result supports the mechanisms for degraded connection performance identified in the first principles analysis. Similar to Connection 1, the RCSM modelled reasonable char depths for directly exposed timber, but where the assumptions of the RCSM were violated, such as around fasteners, charring occurred where it was not predicted. This localised damage, which is not captured by the RCSM, results in poorer connection performance than otherwise predicted, for example, less force can be transferred through the dissipater attachment fasteners when the accelerated charring around them is considered.

Discussion

The connections examined here illustrate that although the existing tools available to designers, the MMBA and RCSM, can be used to create a moment-rotation model of PTT connections exposed to fire, when the connection is exposed to fire it may undergo greater rotations and fail earlier than this model suggests. This difference between modelled and expected response is because the MMBA + RCSM approach does not consider the specific damage that occurs to that connection when it is exposed to fire. The response of PTT connections is a combination of the responses of the individual components (dissipaters, tendon, timber elements, etc). The response of these components depends on their specific detailing and that of the connection. When exposed to a fire, unique damage will occur to these components, which will grow in magnitude as they absorb more thermal energy from the fire.

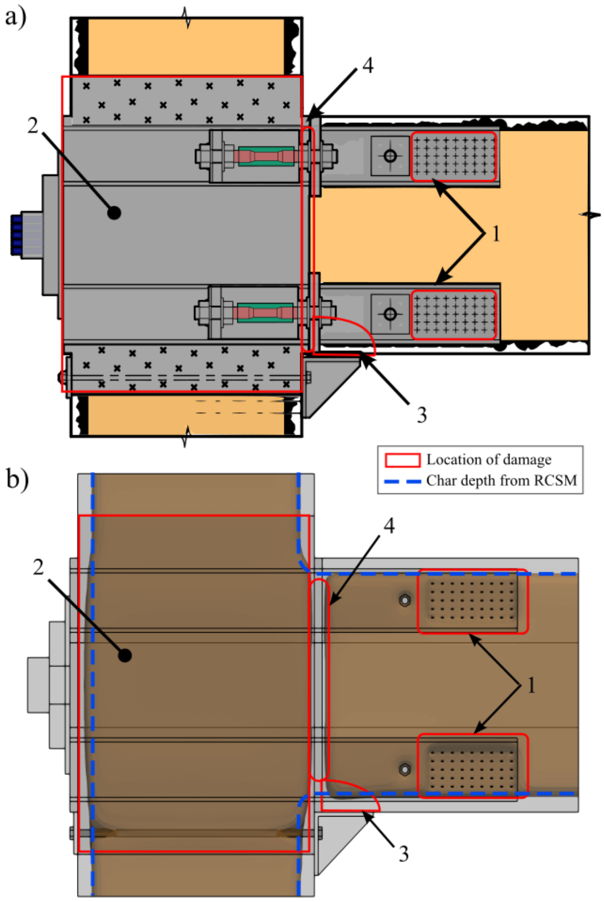


Figure 13: a) Locations of damage when Connection 2 is exposed to fire, b) Render of the 300 °C isotherm of Connection 2 after 60 minutes of Standard Fire exposure from thermal FEA (material hotter than 300 °C is shaded) compared to char depth from RCSM (0.65 mm/min).

The MMBA which relates gap angle to the moment, assumes a rigid rocking interface after decompression and only depends on geometric and material properties of the beam, column, dissipater and tendon. In the connections presented here, localised damage included degraded strength and stiffness of dissipater attachments, softening beam-column interface from conduction through steel plates, the resulting damage from vertical beam displacements to maintain bearing capacity was identified that would result in poorer performance than expected from applying the MMBA. The MMBA overpredicted connection performance; it does not consider the localised damage when these connections are exposed to fire, which depends on the specific detailing of the connection. Although most of the components are capacity protected their material degradation can compromise the overall capacity design principles assumed in the ambient conditions.

The RCSM determines an effective load-carrying section used to calculate the strength of an element after it has been exposed to the Standard Fire for some duration. The 3D thermal field that develops at connections and around steel components is different to the 2D thermal field the RCSM is based on. The two connections demonstrated that there were effects including conduction through partially exposed steel plates, accelerated charring at closely spaced fasteners and charring underneath steel plates, that the RCSM neglects.

The analytical and numerical work presented here shows that the analytical methods currently available are unable to account for the realistic response of these connections. A better analytical method that can accurately model the connection the response of these connections is needed. The response of PTT beam-column connections is complex because of the combined moment and shear loading on the connection and the overall connection behaviour is the combination of the responses of the many components. An understanding is required for response under only an applied moment and only an applied shear load, before they can be combined to model the overall connection response. The response of these connections to these actions should first be studied separately without the complication of interactions and to investigate connection response where shear or moment loads dominate. Given the complex response of PTT beam-column connections to fire, experimental studies are required to verify these new analytical methods. These new methods could incorporate the MMBA for sectional analysis once a gap has opened, but also include factors or other means to account for the connection damage degrading performance. Furthermore, while there is much guidance on connection detailing for robust seismic performance, there is no research or guidance on detailing these connections for good performance in fire.

Improving Fire Resistance of PTT Connections

The two examined connections illustrated the challenges with designing PTT connections for good performance in both seismic and fire actions. The following are apparent from these examples:

1. External steel shear keys lead to charring of the timber bearing surface. To maintain loadbearing capacity the beam will displace downwards. Unexposed steel shear keys do not cause this localised damage.
2. The moment contributions from the exposed dissipaters in fire is governed by the strength and stiffness of the attachment mechanism to the timber beam and column (fasteners or stubs etc). Internal or more deeply fixed dissipaters will maintain moment contributions for longer.
3. Exposed steel plates that are tightly fitted at construction will become loose when the timber surface recedes due to charring. This will lead to “slack” which must be overcome with further deformation in order for the load bearing capability to be maintained. This could be mitigated for some time by installing reinforcing screws underneath the steel plate.
4. Protect the tendon anchorage from the fire such that the tendon force is maintained.

Although only Post-Tensioned Timber Beam-Column connections have been examined, these principles can be applied to other forms of timber connections where fire resistance is required.

CONCLUSIONS

Although the Standard Fire is currently the most widely used benchmark for empirically quantifying an elements fire resistance, to be confident that in the event of a fire there is a low probability of failure or collapse, structures must be designed for their real performance in fire. In order to do this design methods must reflect the actual thermal and mechanical response of the structure under a realistic fire exposure. Designing using Standard Fire does not achieve this objective since the Standard Fire will not occur in real life.

PTT beam-column connections have a complex behaviour in fire because each component of the connection has a different thermal and mechanical response when exposed to fire, and each component interacts with the others. The existing Reduced Cross-Section Method (for determining the residual cross-section of heavy timber sections exposed to fire) and the Modified Monolithic Beam Analogy (for determining the response of a rocking connection) methods cannot simply be combined to predict the response of a PTT connection in fire. These methods only consider the element geometry and elastic material properties and do not include localised damage from fire (which was shown qualitatively in a thermal FEA) which can degrade the overall strength and stiffness of the connection considerably. Careful consideration needs to be given to the detailing of timber connections to ensure that the desired performance under fire is achieved and is not compromised by seismic or other detailing choices.

Since the existing methods available to designers are not satisfactory, better analytical methods are needed which can account for the actual thermal field that will develop in the connection and the mechanical response of each of the components and of the connection overall. Furthermore, these new methods should consider the localised damage that occurs at connections which depends on the specific detailing of the connection. Verifying these new methods with experimental investigations and providing empirically based recommendations for detailing to achieve good fire performance, will increase confidence in the performance of multi-storey Post-Tensioned Timber structures in fire.

REFERENCES

1. Pampanin S, Palermo A and Buchanan A (2013). “*Design Guide Australia and New Zealand: Post-Tensioned Timber Buildings*”. Structural Timber Innovation Company.
2. Brown A, Lester J, Pampanin S and Pietra D (2014). “Re-building Trimble Navigation’s offices using a damage-limiting seismic system”. *World Conference on Timber Engineering*, 10 August, Quebec City, Canada, 2941–50.
3. Kirstein A, Siracusa J and Smith T (2018). “The new timber von Haast replacement building in Christchurch”. *2018 NZSEE Conference*, Auckland, New Zealand.
4. Pei S, van de Lindt J, Barbosa A, Berman J, Dolan J, Zimmerman RB, et al. (2018). “Full-scale shake table test of mass-timber building with resilient post-tensioned rocking walls”. *2018 World Conference on Timber Engineering*, Seoul, Republic of Korea.
5. Palermo A, Pampanin S, Buchanan A and Newcombe M (2005). “Seismic design of multi-storey buildings using laminated veneer lumber (LVL)”. *2005 NZSEE Conference*, 11 March, Taupo, New Zealand, Paper 14.
6. Smith T (2008). “*Feasibility of Multi-Storey Post-Tensioned Timber Buildings: Detailing, Cost and Construction*”. Master’s Thesis, University of Canterbury, Christchurch, New Zealand.
7. Palermo A (2004). “*The Use of Controlled Rocking in the Seismic Design of Bridges*”. PhD Thesis, Politecnico di Milano, Milan, Italy, 277.
8. Pampanin S, Nigel Priestley MJ and Sritharan S (2001). “Analytical modelling of the seismic behaviour of precast concrete frames designed with ductile connections”. *Journal of Earthquake Engineering*, **5**(3): 329–67. https://doi.org/10.1080/13632460109350397

9. Newcombe M (2011). “*Seismic Design of Post-Tensioned Timber Frame and Wall Buildings*”. PhD Thesis, University of Canterbury, Christchurch, New Zealand.
10. Standards New Zealand (1993). “*NZS 3603: Timber Structures Standard*”. Standards New Zealand, Wellington, NZ
11. Standards New Zealand and Standards Australia (2019). “AS/NZS 1720.4 Timber Structures Part 4: Fire Resistance of Timber Elements”.
12. Buchanan AH and Abu AK (2017). “*Structural Design for Fire Safety*”. 2nd ed., John Wiley & Sons.
13. ISO (1999). “*ISO 834-1:1999 Fire Resistance Tests - Elements of Building Construction - Part 1: General Requirements*”.
14. Ministry of Business, Innovation & Employment (2020). “*C/AS2 Acceptable Solution for Buildings Other than Risk Group SH*”.
15. Ministry of Business, Innovation & Employment (2020). “*C/VM2 Verification Method: Framework for Fire Safety Design*”.
16. Kirby BR (1998). “*The Behaviour of Multi Storey Steel Framed Building Subject to Fire Attack - Experimental Data*”.
17. Lange D, Boström L, Schmid J and Albrektsson J (2014). “Charring rate of timber in natural fires”. *8th International Conference on Structures in Fire*, 11 June, Shanghai, China, 521–8.
18. Werther N (2016). “External and internal factors influencing the charring of timber – An experimental study with respect to natural fires and moisture conditions”. *9th International Conference on Structures in Fire*, 8 June, New Jersey, USA, 650–8.
19. Frangi A and Fontana M (2003). “Charring rates and temperature profiles of wood sections”. *Fire and Materials*, **27**(2): 91–102. https://doi.org/10.1002/fam.819
20. Kinjo H, Hirashima T, Yusa S, Horio T and Matsumoto T (2016). “Fire performance, including the cooling phase, of structural glued laminated timber beams”. *Journal of Structural Fire Engineering*, **7**(4): 349–64. https://doi.org/10.1108/JSFE-12-2016-024
21. Carling O (1989). “*Fire Resistance of Joint Details in Loadbearing Timber Construction - A Literature Review*”. SR018, Building Research Association of New Zealand.
22. European Committee for Standardization (CEN) (2004). “*Eurocode 5—Design of Timber Structures, Part 1-2: General—Structural Fire Design*”. CEN, Brussels, Belgium.
23. European Committee for Standardization (CEN) (2004). “*Eurocode 2—Design of Concrete Structures, Part 1-2: General—Structural Fire Design*”. CEN, Brussels, Belgium.
24. Spellman PM (2012). “*The Fire Performance of Post-Tensioned Timber Beams*”. Master’s Thesis, University of Canterbury, Christchurch, New Zealand.
25. Standards New Zealand (2007). “*NZS 3404: Steel Structure Standard*”.
26. European Committee for Standardization (CEN) (2005). “*Eurocode 3—Design of Steel Structures, Part 1-2: General—Structural Fire Design*”. CEN, Brussels, Belgium.
27. Standards Australia (2014). “*AS 1530 Methods for Fire Tests on Building Materials, Components and Structures Part 4: Fire-Resistance Tests for Elements of Construction*”.
28. Peng L, Hadjisophocleous G, Mehaffey J and Mohammad M (2012). “Fire performance of timber connections, Part 1: Fire resistance tests on bolted wood-steel-wood and steel-wood-steel connections”. *Journal of Structural Fire Engineering*, **3**(2): 107–32. https://doi.org/10.1260/2040-2317.3.2.107
29. Lau PH (2006). “*Fire Resistance of Connections in Laminated Veneer Lumber (LVL)*”. Master’s Thesis, University of Canterbury, Christchurch, New Zealand.
30. Frangi A, Erchinger C and Fontana M (2009). “Experimental fire analysis of steel-to-timber connections using dowels and nails”. *Fire and Materials*, **34**(1): 1–19. https://doi.org/10.1002/fam.994
31. van Beerschoten W (2013). “*Structural Performance of Post-Tensioned Timber Frames under Gravity Loading*”. PhD Thesis, University of Canterbury, Christchurch, New Zealand.
32. Palma P, Frangi A, Hugi E, Cachim P and Cruz H (2016). “Fire resistance tests on timber beam-to-column shear connections”. *Journal of Structural Fire Engineering*, **7**(1): 41–57. https://doi.org/10.1108/JSFE-03-2016-004
33. Maraveas C, Miamis K and Matthaiou ChE (2015). “Performance of timber connections exposed to fire: A review”. *Fire Technology*, **51**(6): 1401–32. https://doi.org/10.1007/s10694-013-0369-y
34. Barber D (2018). “Fire tests to ASTME119 on full-size glulam beam to column connections”. *10th International Conference on Structures in Fire*, 6 June, Belfast, United Kingdom.
35. Dassault Systemes SIMULIA Corp. (2019). “*Abaqus 2020*”. Dassault Systemes SIMULIA Corp., Johnston, RI, USA.
36. Nelson Pine (2016). “*Nelson Pine LVL - Specific Engineering Design Guide*”.
37. CEN (2002). “*Eurocode 1—Actions on Structures Part 1-2: General Actions—Actions on Structures Exposed to Fire*”.