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ANALYTICAL MODELLING OF POST-TENSIONED TIMBER BEAM-COLUMN CONNECTIONS IN FIRE

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ABSTRACT: Post-Tensioned Timber (PTT) is a structural system that has many advantages over conventional timber frames, particularly in high-seismic areas such as New Zealand. These include a low-damage, re-centring system and fast construction. Although PTT systems have been shown to perform well in ambient conditions, concerns remain about the performance of beam-column connections in fire. As part of a larger project to address these concerns, a full-scale loaded PTT beam-column subassembly was exposed to a Standard Fire. Connection moment resistance was lost when thread stripping at the anchorage nut. An analytical model was compared to the experimental results. The decompression moment decreased due to the reduced tendon force from heating, the tendon acting eccentrically and the reduced effective cross-section. The analytical connection rotation after decompression was a third of experimental rotation until thread stripping began. This analytical model can be improved by including more complex aspects such as softening of a heated rocking interface and determining the beam stiffness based on the reduced material properties beneath the char layer. A follow up study is being undertaken to address these improvements and to provide a method for designers to evaluate the performance of these connections in fire.

KEYWORDS: Post-Tensioned Timber, Frame, Connections, Fire

1 INTRODUCTION

In high-seismic areas such as New Zealand, building owners and occupiers are also becoming more aware of the downtime and costs of repair or demolition of a building after an earthquake. Instead of conventionally designed structures, they seek low-damage structures which can be reinstated more easily, faster and cheaper. In contrast to conventionally designed structures, where seismic energy is dissipated by plastic hinges in primary structural elements, the primary structure of a low-damage system responds elastically and seismic energy is absorbed by specially designed components, which can be replaced after an earthquake.

Post-Tensioned Timber (PTT) is a technology that meets the demands for both a low-damage structure and timber as the primary structural material. PTT frames utilise prestressed unbonded steel strands or high strength bars inside timber elements, running the length of the frame, with replaceable energy dissipaters at the top and bottom of each beam-column connection (see Figure 1a). Under a lateral load, a gap opens at the rocking beam-column interface and the dissipaters are stretched or compressed, dissipating the seismic energy (see Figure 1b). Moment resistance at the connections is provided by force-couples from the dissipaters in tension and compression and from the tendon and compression of the beam against the column. The stressed tendon re-centres the structure after the conclusion of the lateral action. This technology can also be applied in low seismic areas to achieve longer spans than typically possible with timber elements alone, by using draped tendons and omitting the energy dissipaters (see Figure 1c). The advantageous performance of this structural system is due to the ductile rocking connections which contain the rocking interface and energy dissipation devices (see Figure 1d). The performance of PTT structures under seismic and gravity actions at ambient temperature has been extensively researched at the University of Canterbury [1–7] [1, 2] and a design guide has been published [8]. Several PTT frame buildings have been built in New Zealand and overseas, for example, Trimble Building [9], Young Hunter House and Von Haast [10] in New Zealand and ETH Zurich House of Natural Resources, Zurich, Switzerland [11] (Figure 1e shows a PTT frame during construction).

Although the behaviour of Post-Tensioned Timber box beams in fire has been investigated [12–14], the response of PTT frames and their beam-column connections in fire has not been investigated. Design guidance [8], therefore, proposes two design approaches for PTT structures in fire: the protected and unprotected approach. In the protected approach (see Figure 2a) the tendon is protected from the effects of fire so that the tendon force is maintained. Even though the cross-sections of the timber elements are reduced by charring, the moment resisting connections and moment-frame are preserved. In the unprotected approach (see Figure 2b), the tendon is not explicitly

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protected from the fire. On this basis, it is assumed the tendon force is zero and the beams are designed as simply supported with reduced cross-sections. All PTT buildings built in New Zealand so far have used the unprotected approach, because there is no additional costs to install protection and the design check is simple [12].

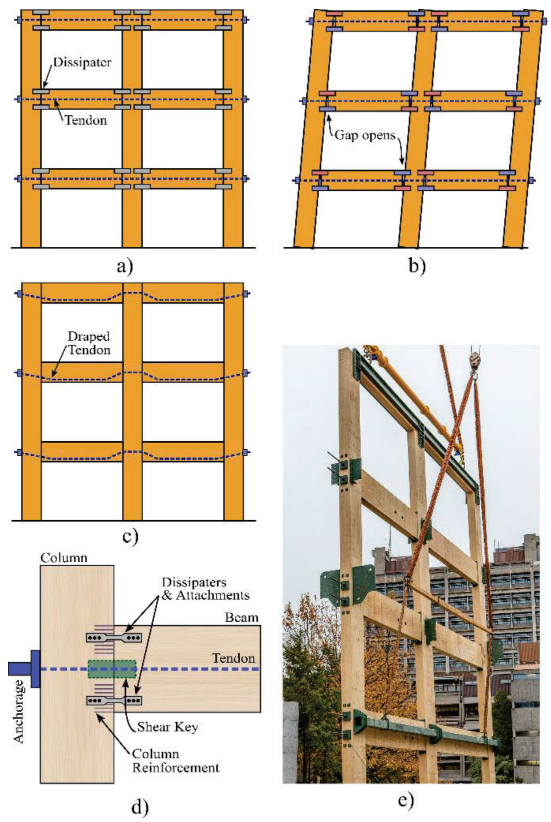

Figure 1. *Illustration of PTT frames a) in a high seismic area at rest and b) under a lateral load and c) in a low seismic area, d) a schematic of a PTT beam-column connection and e) a PTT frame during construction*

While these two design approaches are easily implemented by designers, they do not evaluate the actual response of the connection or structure in a fire. Even in the unprotected case, the tendon will take some time to be heated through an exposed tendon anchorage and for the force in the tendon to decrease. Furthermore, in structures designed for seismic loads, features may already be present that also provide advantageous performance in fire; e.g. larger column sizes for increased lateral stiffness can withstand greater charring in a fire. Although harnessing this advantageous fire performance is an opportunity for increased economy, increased performance in fire is not guaranteed by seismic design and detailing. A previous study has shown that PTT beam-column connections detailed for beneficial seismic performance does not necessarily also improve the connection performance in fire [15].

Previous study [16,17] modified the analytical model for PTT beam-column connections at ambient to the fire scenario and compared results from the analytical model to an experiment of a simple reduced-scale PTT beam-column connection (i.e. had a tendon, anchorage and internal corbel only). The analytical results compared favourably with those from the experiment; the studied connection did not have all the detailing features of a beam-column connection in a multi-storey PTT frame structure (e.g. interface stiffening, creep prevention or an external corbel which is more common) and was also at a reduced-scale. Therefore, questions remain about how representative the analytical model is of "fully" detailed PTT connections.

To address these questions, an experiment of a full-scale PTT beam-column connection is required given the complex thermal and mechanical response of PTT beam-column connections in fire. This paper presents the experimental results from a full-scale PTT connection that was detailed to current design practice in New Zealand and a comparison with an analytical model. Firstly, the components of a PTT beam-column connection and the analytical model are summarised. Secondly, the key results and observations from this experiment are reported and thirdly, the analytical and experimental results are compared.

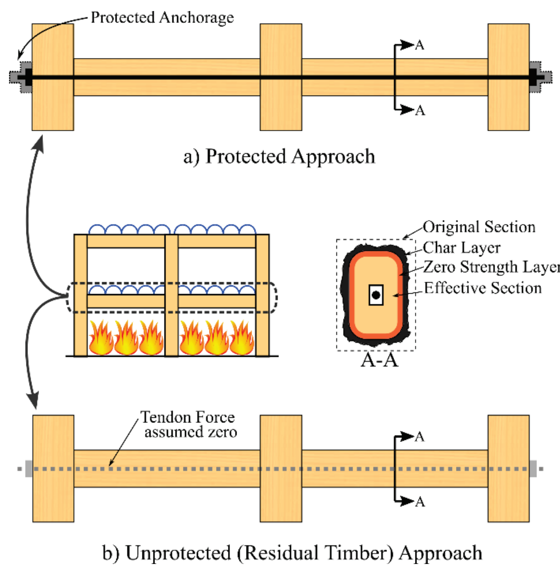

Figure 2. *Illustration of the a) Protected and b) Unprotected approaches to the design of PTT frames*

2 PTT BEAM-COLUMN CONNECTIONS

The advantageous response of PTT structures is directly attributable to the ductile rocking behaviour of the connections while the timber elements remain within their elastic limits. For a beam-column connection in a frame to have this beneficial behaviour, it must contain (see Figure 1d):

1. A tendon passing through ducts in the beam and column.
2. A tendon anchorage at the beam-column connections at each end of the frame.

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3. Dissipaters and their attachments to the timber elements. Dissipaters can take several forms from buckling restrained necked bars to reduced cross-section angles.
4. Corbel or shear key to transfer shear from the beam to the column, which can be external or internal.

However, further detailing of the connection must be considered to ensure the desired stiffness and long-term performance is achieved:

1. The rocking interface must be sufficiently stiff so that drift limits under Serviceability Limit State (SLS) lateral actions are achieved. The interface stiffness depends on the material properties and size of the timber elements, and whether there is a steel interface plate.
2. Perpendicular-to-grain crushing of the timber column must be avoided under large gap angles when the bearing area is small.
3. Creep deformations from the compression of timber column perpendicular-to-grain must be limited so the required tendon force is maintained over the life of the building.

Connection detailing to address these considerations includes screw reinforcement in the column, a steel jacket around the column, steel compression elements inside the column or the column being fabricated with rotated timber blocks at the beam-column zone so the grain is parallel to the compressive forces (see [8]).

The mechanical behaviour of PTT beam-column connections in fire strongly depends on the thermal field established in the joint and the complex interaction between the many connection components. Therefore, the behaviour of PTT connections in fire will depend on their configuration and detailing choices by the designer. While different detailing options to achieve the same objective may result in similar ambient temperature performance, e.g. stiffening the interface with reinforcing screws or rotated timber blocks, they may perform differently in fire. This is particularly true where steel components are introduced which conduct much more than the surrounding timber.

3 ANALYTICAL MODELLING OF PTT CONNECTIONS

The presence of a gap at the beam-column interface violates the assumptions of the Euler-Bernoulli beam theory; therefore, it cannot be applied. The Monolithic Beam Analogy was developed to overcome the same limitation in controlled rocking joints from precast concrete elements during the Precast Seismic Structural Systems (PRESSS) project [18,19]. It was extended to timber elements and called the Modified Monolithic Beam Analogy) several additional complexities had to be addressed including initial joint stiffness and compression block [2,4,5,8]. The MMBA establishes a displacement equivalence between the rocking element and a "monolithic" analogue to determine the equivalent curvature in terms of the gap angle. This additional displacement compatibility equation combined with the two equations of equilibrium allows the calculation of the moment for each given rotation or gap angle. The extension of the MMBA into the fire case is summarised here but explained in more depth in [16] and [17].

The MMBA for an element exposed to fire, is derived by establishing displacement equivalence between the fire-exposed rocking and monolithic cantilever elements of length, ℓ, under axial load, N, (which is the tendon force including the additional force from stretching) and external horizontal load, P, as shown in Figure 3. When the element is exposed to fire on three sides, the centroid moves towards the unexposed side, and so the tendon force acts eccentrically (e), producing a uniform moment (Ne) along the element. This moment contributes to decompress the beam, so less external horizontal force is required to decompression the interface. At decompression, the stress at the extreme tension fibre is zero, so it can be expressed as:

$$\frac{N}{A}-\frac{M_{dec}}{Z}=\frac{N}{A}-\frac{Ne+P_{dec}\ell}{Z}=0 \quad (1)$$

where P_{dec} is the external horizontal force required for decompression. The curvature at the base of the rocking element at decompression is ϕ_{dec}. From the two moments Ne and $P_{dec}\ell$, the sum of the curvatures produced by these moments is: $\phi_{dec}=\phi_{Ne}+\phi_{P_{dec}\ell}$.

The displacement at the top of the rocking and monolithic cantilever elements in terms of the curvature at the base of the element is (i.e., applying Equation 1):

$$\frac{\phi_{Ne}\ell}{2}+\frac{\phi_{P_{dec}\ell}\ell^2}{3}+\theta_{gap}\ell=\frac{\phi_{Ne}\ell}{2}+\frac{\phi_{P\ell}\ell^2}{3} \quad (2)$$

$$\frac{\phi_{P_{dec}\ell}\ell^2}{3}+\theta_{gap}\ell=\frac{\phi_{P\ell}\ell^2}{3} \quad (3)$$

$$\left(\frac{3\theta_{gap}}{\ell}+\phi_{P_{dec}\ell}\right)=\phi_{P\ell} \quad (4)$$

This is the same form as the ambient-temperature seismic MMBA formulation except the curvature required for decompression is reduced by the tendon acting eccentrically.

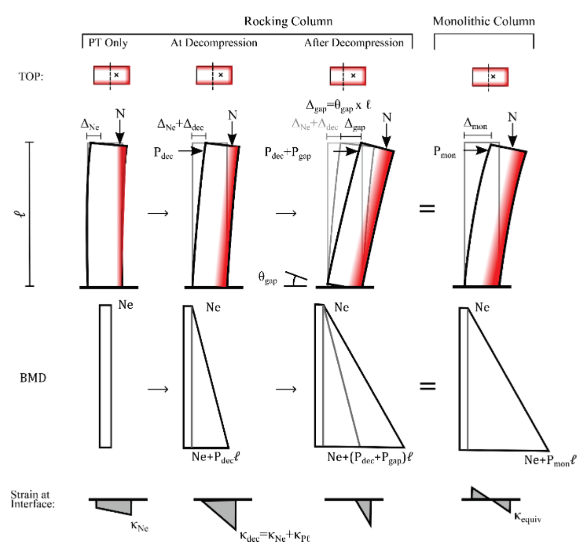

Figure 3: *MMBA for a cantilever element exposed to fire*

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4 EXPERIMENT

4.1 SETUP

A 1:1 scale PTT beam-column subassembly, based on the connection detailing of recently constructed PTT framed structures [10] and published design guidance [8], was constructed, loaded and exposed to fire. The tested sub-assembly (shown in Figure 4) was constructed from LVL13 (E=13.2 GPa [20]) 700x310 mm beam and 800x310 mm column with two 75 mm square ducts for the tendons. These timber elements were fabricated from seven 45 mm thick LVL laminations glued to form the 310 mm thick elements. Bare wire K-Type thermocouples were laid in small grooves on the surface of the seven laminations prior to gluing. The two tendons were 26.5 mm diameter high-strength Macalloy 1030 bars [21] tensioned to 160 kN (28% of UTS) with 200 mm square, 40 mm thick anchorage plates. Two 75 mm square hollow steel sections (5 mm wall thickness) were installed inside the ducts to prevent long-term creep perpendicular to the grain of the column. Reinforcing screws were installed in the column at the top and bottom of the beam-column contact area to prevent crushing of the column perpendicular to the grain under large gap angles in seismic events. Vertical support of the beam at the connection was provided by a steel bracket with central stiffener fixed to the column with 10x M20 coach screws. Thermocouples were placed at the interface between the bracket and the beam and column.

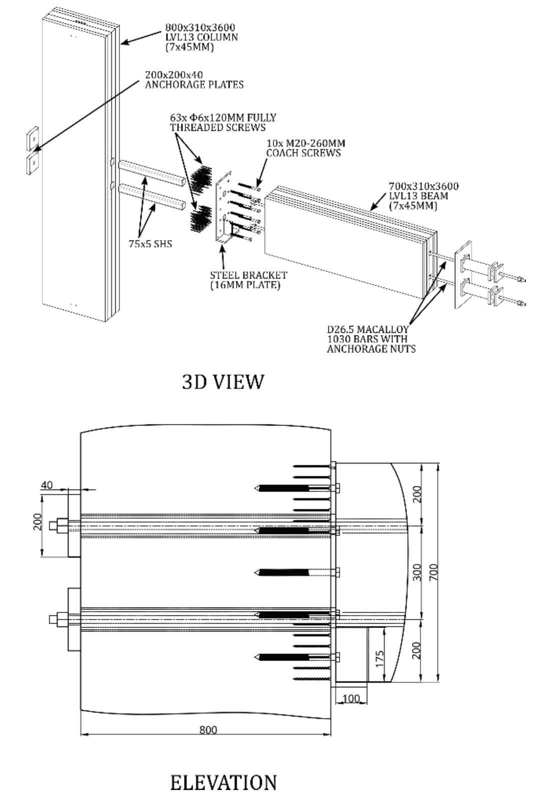

Figure 4: *3D and elevation views of the tested beam-column sub-assembly*

The beam-column sub-assembly was placed in a purpose-built enclosure, so that the beam was exposed on three sides and the column was exposed on four sides below a floor level. The enclosure was made from 13 mm calcium silicate board lined internally with an alkaline earth silicate fibre blanket. The effect of a floor was simulated with a 100 mm thick slab of concrete on the beam. Two viewing windows were installed in the enclosure for visual observations. This enclosure was placed on a 3 x 4 m standard fire resistance testing furnace in its vertical orientation (see Figure 5). A point load of 16 kN was applied to the end of the beam to achieve a total connection moment of 29.6 kNm (90% of the decompression moment). An axial force of 50 kN was applied to the top of the column (limited by hydraulic jack capacity). The furnace was controlled to follow the ISO 834 Standard Fire [22] time-temperature curve (according to AS 1530.4 [23]) until failure of the connection.

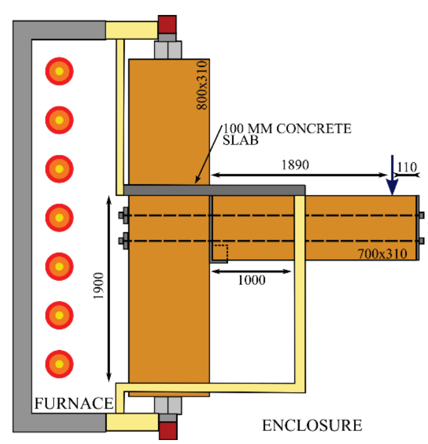

Figure 5: *Schematic of beam-column subassembly in enclosure on furnace*

4.2 OBSERVATIONS

Photos of the connection and the residual cross-section of the timber elements are shown in Figure 6. Corner rounding at the bottom of the beam had occurred and the contoured shape of the column near the connection zone indicates the thermal field was not the same as in the timber elements away from the connection, and charring of the bottom of the column interface which was promoted by conduction through the steel bracket.

Also evident in Figure 12 is the effect of anchorage reinforcement; in this case, the anchorage plates are held off the char surface. Although the 40 mm thick anchorage plates did delay the onset of charring underneath by approximately 15 minutes, without reinforcement under anchorage, the large bearing stress would result in crushing of the timber column perpendicular-to-grain at low temperatures. In this connection, the detailing to prevent long-term creep (the SHS tubes inside the column) formed an alternative load path and supported the anchorage plates as the timber underneath charred. Without anchorage reinforcement to resist the high

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bearing stress from the anchorage, hot wood/char underneath and anchorage plates would be crushed, and the tendon force would decrease from shortening of the tendon. Therefore, if the moment resistance of connections and the frame is to be maintained, the exterior frame connections should be detailed to avoid this failure mechanism.

Figure 6: *a) Photo of the connection after the test and b) residual cross-sections*

4.3 TEMPERATURE RESULTS

The temperature of the furnace control thermocouple probes is shown in Figure 7. Char rates, determined from thermocouples in the beam and column away from the connection (using a 300 °C isotherm), ranged from 0.62 to 0.78 mm/min with a mean of 0.70 mm/min. A previous study of the charring of LVL elements in the Standard Fire reported a mean charring rate of 0.72 mm/min [24]. Only a selection of key thermal results are presented here.

The steel bracket, which supported the beam, was exposed to the fire on three sides and conducted heat through the interface. At 21 minutes, the temperature at the interface between the beam the bearing plate of the bracket exceeded 300 °C. Temperatures measured at the interface between the steel bracket and the column are shown in Figure 8. The temperature developments at locations A-D show that a 2D thermal field was established in the lower third of the interface. Heat was conducted up the interface from the exposed bearing plate and from the exposed sides of the interface plate. In the top two-thirds of the interface plate the thermal field was one-dimensional, decreasing in temperature from the exposed edges to the centre of the bracket.

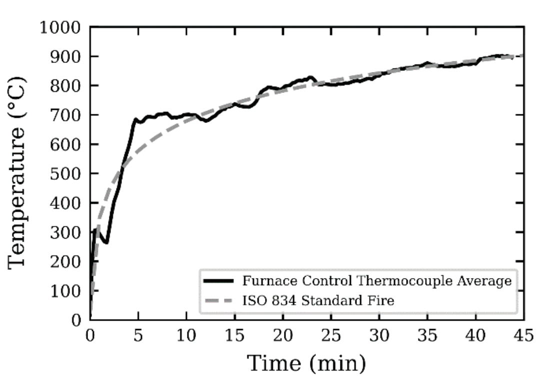

Figure 7: *Temperature of furnace control thermocouple probe*

A large thermal gradient was recorded along the tendon (see Figure 9). The temperature of the tendon underneath the anchorage nut increased quickly and reached a maximum temperature of over 700 °C at the end of the experiment. However, at the same time the temperature 100 mm into the column the tendon was 270 °C and less than 100 °C at 300 mm.

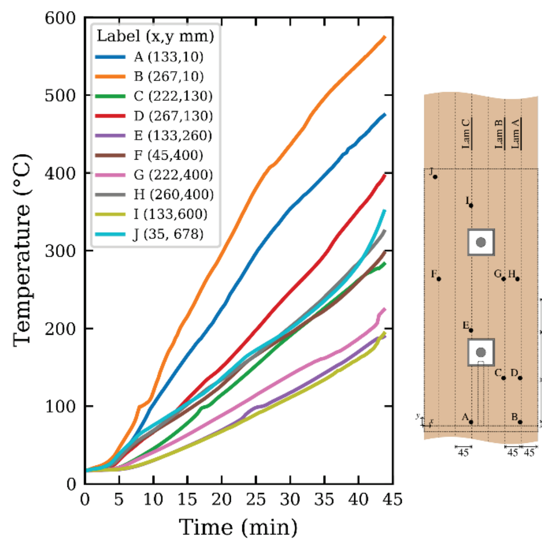

Figure 8. *Temperatures at the interface between the timber column and steel bracket*

The temperature of the tendon was also monitored at the beam-column interface to determine if the tendon was heated by the opening at the interface. The temperature of the bottom tendon did increase slightly (less than 100 °C) as it was heated from the hot steel bracket. The temperature of the top tendon was the same as the bottom tendon except for the final five minutes, when the temperature increased rapidly to 200 °C as hot gases penetrated the gap at the beam-column interface.

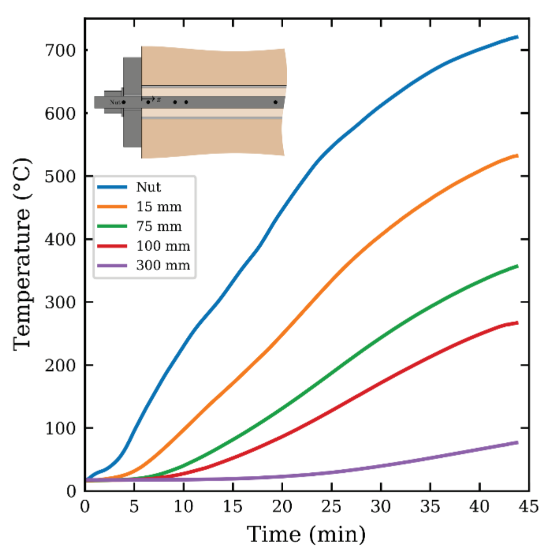

Figure 9: *Temperature of tendon*

4.4 MECHANICAL RESULTS

The connection decompressed after 5 minutes of fire exposure and increased linearly (approximately 0.0014 rad/min) until 37 minutes (a rotation of 0.005 rad). The connection rotation then increased rapidly, and the test was terminated at 44 minutes at a rotation of more than 0.05 rad.

The tendons were initially stressed to 160 kN each although the force in the top tendon increased by around 5 kN and the force in the bottom tendon decreased by the same amount as the beam was loaded. Between 5 and 10 minutes the tendon force in the top and bottom tendons decreased by 13% and 7% respectively. The force in the bottom tendon remained slightly greater than the top tendon until 27 minutes, at which time the force had decreased to 75% of its initial value. After this time, the forces in the tendons reduced at a greater rate. The force in the bottom tendon decreased to 25% of the initial force. The force in the top tendon decreased to 52% at 37 minutes and remained roughly constant for five minutes. In the final minutes of the test the top tendon was unable to sustain the applied force and the interface gap opened to the extent that the bottom tendon was stretched to provide additional moment resistance.

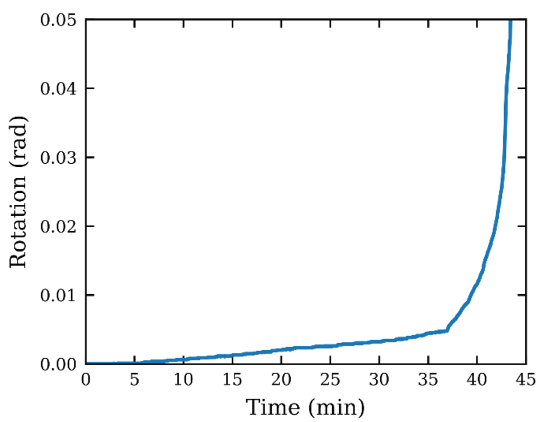

Figure 10: *Connection rotation*

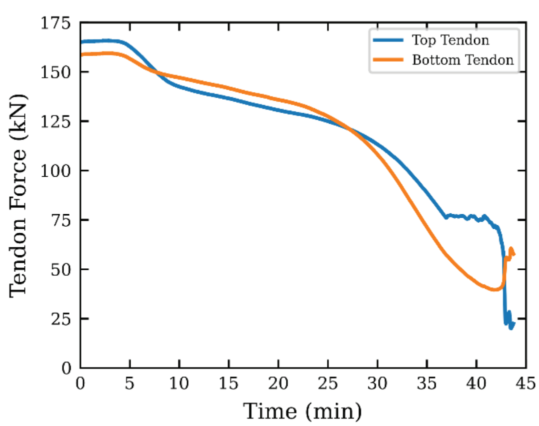

Figure 11: *Forces in the tendons*

The rapid increase in connection rotation (see Figure 10), that began at 37 minutes, coincided with the plateau in the force in the top tendon (see Figure 11). At the conclusion of the experiment, the top tendon had pulled through the anchorage nut (see Figure 12).

For a bar to be pulled a bar through a nut, the threads undergo large plastic deformations, which would explain the constant tendon force and the rapid increase in rotation.

Figure 12: *Tendon anchorages after test*

5 ANALYTICAL MODELLING

The response of the connection in the experiment was modelled using the MMBA procedure that has been modified for fire (see section 3). The effective cross-section of the timber elements was determined using two different implementations of Residual Cross-Section Methods because they are already familiar to practitioners. The two methods were:

1. Char rate of 0.65 mm/min with corner rounding explicitly calculated (radius equal to char depth). This is based on the residual cross-section method in NZS 3603 [25].
2. Char rate of 0.65 mm/min and a 7 mm zero-strength layer. This is based on AS/NZS 1720.4 [26] but the zero-strength layer is considered to develop over 20 minutes [13].

The cross-section was then transformed into an equivalent section with ambient elastic modulus (using modulus ratios) to be used in the MMBA procedure.

The tendon force was reduced by thermal expansion and reduced elastic modulus in the heated portion. For this analysis the temperature profile along the tendon was obtained by fitting a piece-wise linear curve to the tendon temperatures measured in the experiment (Figure 9). Thermal expansion and reduced material properties of prestressing steel were adopted from EN 1992-1-2. The ambient elastic modulus of the tendons was 170 GPa, the secant value provided by the manufacturer [21].

5.1 Decompression Response

The decompression moment is the moment at the connection that corresponds to the point of

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decompression; i.e. stress at the extreme fibre is zero. The decompression moment decreases as the effective cross-section of the timber element and the tendon force reduce. The decrease in decompression moment as the connection was exposed to the Standard Fire is shown in Figure 13 for a constant tendon force case and a tendon relaxation case (thermal expansion and reduced elastic modulus as it is heated). After the first five minutes the decompression moment decreased in an almost linear manner. Following 45 minutes of Standard Fire exposure, a 5% reduction in decompression moment was attributable to the loss of cross-section area and approximately 18% was the result of the reduction in tendon force.

The decompression curvature (in the beam and at the interface) is an important parameter in the MMBA as it relates to the displacement prior to decompression. The reductions in decompression curvature for both the constant and relaxed tendon cases are shown in Figure 14. In the case of the constant tendon force, the decompression curvature continues to increase throughout the exposure, by a factor of 1.34 for Method 1 and 1.44 for Method 2 (at 45 minutes). However, when relaxation of the tendon was considered the decompression curvature increased for the first 30 minutes of the exposure, after which it remained relatively constant. The decompression curvature had increased by roughly half the amount, as in the constant tendon force case, to 1.10 for Method 1 and 1.19 for Method 2 (at 45 minutes).

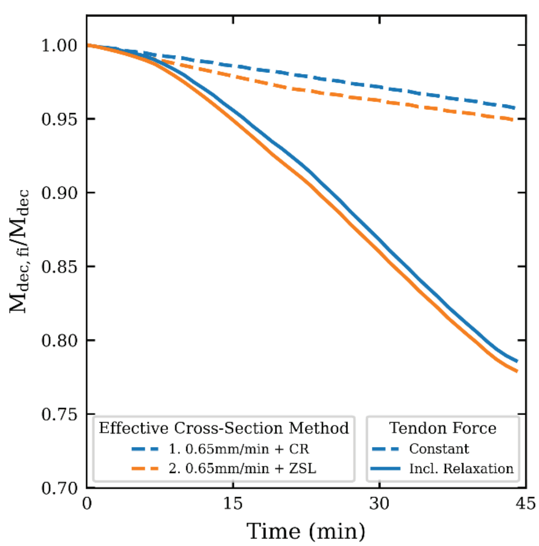

Figure 13: *Decompression Moment when exposed to a Standard Fire*

In seismic resisting PTT applications, the tendons are located so that the centroid of the tendon force and the neutral axis align. However, when the rocking element in a PTT system is exposed to fire on three sides, such as a beam supporting a floor, the neutral axis moves towards the unexposed face as the element is heated and chars. The tendon force then acts eccentrically so an additional moment acts to decompress the connection. Consequently, the moment required to decompress the connection decreases. The additional moment from the tendon acting eccentrically is shown in Figure 15 for both the constant and relaxed tendon cases. The additional moment reached 12-15% of the initial decompression moment after 45 minutes of exposure for the relaxed tendon case, which was about 80% of the additional moment from the constant tendon case.

Figure 16 shows the moment that must be applied to the connection to cause decompression (accounting for the additional moment from the tendon eccentricity and the decreasing decompression moment) as a proportion of the ambient decompression moment. This applied moment for decompression decreases by 17-20% of the ambient decompression moment for the constant tendon case and by a third for the relaxed tendon case.

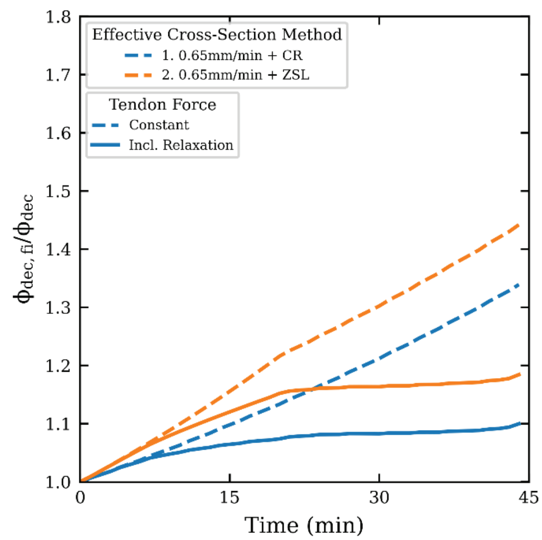

Figure 14: *Decompression Curvature when exposed to a Standard Fire*

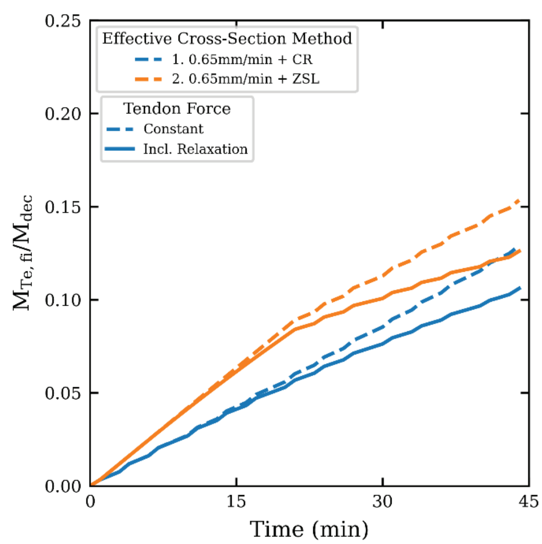

Figure 15: *Additional moment on the connection from the tendon eccentricity when exposed to a Standard Fire*

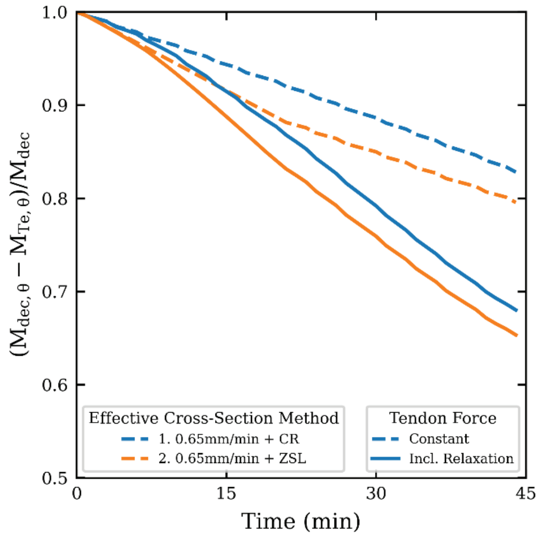

Figure 16: *Moment applied for connection to decompress when exposed to a Standard Fire*

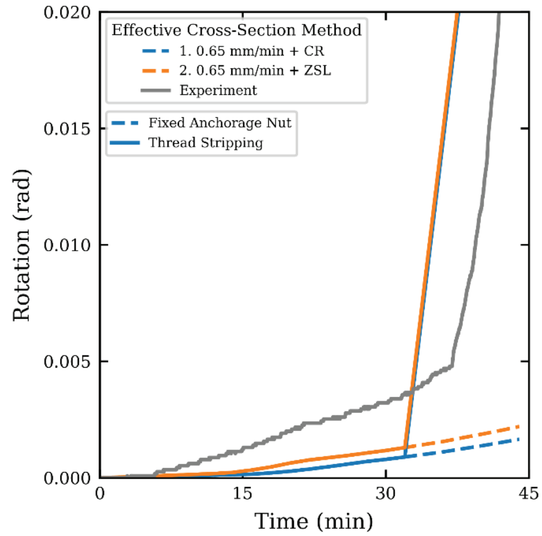

Figure 17: *Analytical and experimental connection rotation*

5.2 Post-Decompression Response

The softening of the interface from the heated interface plate was accounted for by taking the interface stiffness modification factor, k_{gap}, as 0.1 (the value for an unreinforced interface) [9]. Thread stripping of the anchorage nut that was observed in the experiment was included in the model by displacing the anchorage at a constant rate after the strength of the nut was exceeded (32 minutes). This rate assumed the anchorage nut deformation (measured at the end of the experiment) occurred at a constant rate.

Figure 17 compares the analytical connection rotation, with and without thread stripping at the anchorage nut, with the experimental rotation. The connection decompressed at similar times in the experiment and in the analytical models. Like the experimental response, the analytical response had two distinct stages – before and after thread stripping starts. The analytical model predicts only 30% of the experimental rotation before thread-stripping of the anchorage nut began. Thread stripping occurred a few minutes earlier in the analytical model than in the experiment. The thread stripping of the anchorage nut effectively lengthens the tendon and additional connection rotation is required to maintain the tendon force and moment resistance. The different responses of the analytical results with and without thread stripping at the anchorage nut shows that thread stripping dominated the connection response.

6 DISCUSSION

The 1:1 scale experiment of PTT beam-column sub-assembly showed that even when the tendon anchorage is not protected from the fire, the moment-resisting connection has some inherent fire resistance. The unprotected design approach neglects this beneficial performance. The edges and bottom of the steel bracket were exposed to the fire and conducted heat into the connection interface. The elastic modulus and strength of the adjacent wood decreased, softening the connection response. At 37 minutes the top tendon pulled through the anchorage nut. Insulating the anchorage and tendon from the fire will reduce the relaxation of the tendon and delay failure of the anchorage. The internal steel tubes resisted the force from the anchorage when the adjacent timber had charred. Without these tubes, the char and heated layers would have been crushed and the tendon force would have been lost earlier. Similarly, the screw reinforcing in the interface supported the steel bracket when the surrounding timber had charred. Without this screw reinforcement the connection response would have been a lot softer as this timber was crushed and the analytical model would have to be modified to take this into account.

The analytical modelling prior to decompression shows that the pre-decompression behaviour was reduced more by the decreased tendon force (due to thermal expansion and reduced elastic modulus) than the decrease in effective cross-section. Of the two Reduced Cross-Section Methods used to determine the effective cross-section, the method that considered a zero-strength layer always resulted in a softer connection response.

The response of the analytical models compared to the experimental results after decompression indicate that there is a softening phenomenon between 5 and 30 minutes that is missing from the analytical model. This is likely a result of the 3D thermal field in the bottom corner of the beam (due to the heated steel bracket), which is also

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where the greatest bearing stresses would normally occur. Any additional deformation of the interface timber would also reduce the tendon force, so the analytical models would better represent the experimental tendon forces. To examine this further, the 3D thermal field at the connection and heat transfer between adjacent parts should be investigated then the analytical model can be modified to include these effects.

The MMBA is based on an additional displacement condition, so the stiffness of the rocking element (i.e. beam) is a key parameter. The stiffness of a timber element exposed to fire depends on the effective cross-section area, due to charring, and the reduced material properties in the heated layer underneath the char layer. Reduced Cross-Section Methods, such as those used in this analysis, are commonly used by designers to predict the failure times of loaded timber elements exposed to a Standard Fire. However, these methods are developed for a strength criterion, so may not be directly applicable to calculating the reduced stiffness. To alternative methods would be to consider a parabolic temperature distribution in the heated layer or determine the temperature distribution using a numerical technique e.g. finite element analysis. Reduced mechanical properties through the heated layer can be explicitly considered, e.g. using the reduced mechanical properties in EN 1995-1-2.

The thread stripping of the anchorage nut on the threaded bar tendon was not considered adequately in this analysis. Further study is required to develop an appropriate sub-model for this plastic deformation. One option would be to limit the tendon force to the shear yield strength of the threads reduced by the temperature-strength relationships of EN 1992-1-2 [27].

The analytical model presented here captured some of the response of realistically detailed PTT beam-column connections, however, the response is much softer than in the experiment and some important aspects are missing. While some detailing features can improve performance in fire (e.g. interface reinforcement screws), other features degrade connection performance (e.g. partially exposed steel bracket). Further study is required to modify the analytical method to account for the softer response of these connections and to refine how different types of detailing change the connection behaviour.

7 CONCLUSIONS

The performance of Post-Tensioned Timber frames is governed by the behaviour of the beam-column connections and their detailing. A loaded 1:1 scale PTT beam-column sub-assembly, with current seismic detailing, was exposed to the Standard Fire. The connection maintained its moment resistance for 40 minutes despite the tendon anchorages being exposed directly to the fire. This was in opposed to the Unprotected design approach, which neglects the tendon contribution from the start of the fire exposure. The internal SHS tubes in the column and the screw reinforcing at the interface, installed for long-term and seismic considerations, were important to obtaining this performance.

An analytical model for PTT beam-column connections in fire, previously used with medium-scale simple PTT connections, was used to evaluate the response of the connection that was tested. Reduced performance of the connection before decompression was primarily due to a reduced tendon force (thermal expansion and lower elastic modulus in the heated portion) and to a lesser extent, the reduced effective cross-section. The analytical model only rotated approximately a third of the experiment before thread stripping occurred. Once it began, thread stripping of the anchorage nut on the bar tendon dominated the connection behaviour.

The analytical model did not capture some behaviour that was observed in the experiments which this study has shown to be important to the connection response, e.g. softening of the rocking interface as it was heated. To address these shortcomings and improve the analytical model, a follow-up numerical study is being undertaken to investigate the connection's thermal field and the resulting structural response. This analytical model will enable designers to assess the actual response of PTT beam-column connections in fire.

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