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Implications of seismic design detailing on the fire performance of Post-Tensioned Timber frames

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ABSTRACT

Post-tensioned timber (PRES-LAM) frames have significant advantages over traditional timber frame systems especially where a damage resistant philosophy and fast erection are desired. Although, post-tensioned timber beams have been investigated in fire, in practice the post-tensioning has not been used to provide stability in the structural fire load cases. The robustness of seismically designed post-tensioned timber beam-column connections is investigated using sectional analysis and then the influence of detailing is examined. Protection of the tendon, presence of gaps, quantity of exposed steel, fastener type and location and choice of shear key are found to have an influence on the expected performance of these connections.

1 INTRODUCTION

PRES-LAM technology uses unbonded steel tendons or high strength steel bars passing through ducts in LVL timber elements with optional energy dissipaters to create a low damage and re-centering frame or wall structures. Post-tensioned timber buildings offer many advantages over traditional timber construction, especially in seismic areas. Figure 1 illustrates the advantageous flag shaped moment-rotation response of a post-tensioned connection with dissipaters. Post-tensioning not only provides self-centering capability but is a cost-effective and fast method of construction as many moment resisting connections can be made in a single stressing operation (Palermo, Pampanin, Buchanan, & Newcombe, 2005; Pampanin, Palermo, & Buchanan, 2013; T. J. Smith, 2008). This explains the continuously increasing popularity among national and international researchers and practitioners (Brown, Lester, & Pampanin, 2014; Kirstein, Siracusa, & Smith, 2018; Pampanin et al., 2013; Pei et al., 2018; The Framework Project LLC, 2019). As structural fire load cases are typically gravity loads, and walls are often not significantly part of the gravity load path in post-tensioned timber buildings, the primary focus of this study is post-tensioned frames. Post-tensioned timber frames can be classified according to their loading as either "gravity only", seismic only" or "combined gravity-seismic".

Although the seismic implications of post-tensioned timber buildings has been extensively researched (Moroder, 2016; Newcombe, 2011; Pampanin et al., 2013; Sarti, 2015; T. Smith, 2014; van Beerschoten, 2013), only the structural fire performance of post-tensioned timber box beams have been investigated. Spellman (2012) and Costello (2013) conducted fire tests on post-tensioned timber box beams and developed design methodologies for predicting the capacity of these beams after exposure to the standard fire.

When timber is exposed to a fire, char forms on the exposed surfaces which reduces the effective area of the section but acts as a layer of insulation for the remaining section. Under the standard fire timber has a relatively constant and predictable charring rate (Frangi & Fontana, 2003; König & Walleij, 1999). Steel elements heat up quickly when exposed to fire and soften and lose strength when exposed to high temperatures. When steel elements are embedded in timber, they conduct heat into the timber element.

Post-tensioned timber beam-column connections are the key components of the lateral load resisting system so significant energy is spent detailing these connections to achieve the desired performance. These connections may also be a key component of the load resisting system in the structural fire load cases. Therefore, understanding the behaviour of these connections is essential to assessing the full structural response in a fire, especially as the tendon runs through several spans. Sometimes detailing requirements for good seismic and fire performance conflict.

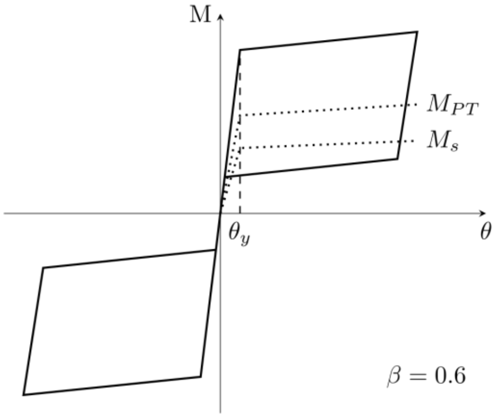


Figure 1: Moment-Rotation Curve for a post-tensioned connection with dissipaters

Figure 3: A beam-column connection using angle dissipaters

Figure 2: A post-tension timber frame being erected (courtesy of University of Canterbury)

In this study the robustness of seismically designed post-tensioned timber connections in fire is examined. Firstly, the behaviour of each component in a beam-column connection is discussed. The Modified Monolithic Beam Analogy (MMBA) is applied to a post-tension timber connection to identify how moment-rotation relationship of the connection changes as the connection is exposed to fire. However, the MMBA method (Newcombe, 2011; Palermo, 2004; Pampanin, Priestley, & Sritharan, 2008) only depends on geometrical and material properties and does not account for detailing. Experimental evidence shows that detailing can greatly affect the fire performance of timber connections (Maraveas, Miamis, & Matthaiou, 2015; Palma, 2016; Peng, 2010). So, the performance of three post-tensioned beam-column connections exposed to fire is hypothesized by accounting for the effects of detailing.

2 APPLICATIONS

Several Post-Tensioned Timber frame buildings have been built in New Zealand including Trimble (Brown et al., 2014), Merrit, St Elmo Courts and Beatrice Tinsley (Kirstein et al., 2018) buildings. An example of a post-tensioned timber frame and connection are presented in Figure 2 and Figure 3. Except for one building, which has a two-way frame and concrete columns, these buildings use post-tensioned frames in one direction. Most use grouted necked steel bars as dissipaters and use five to seven size 15 mm strands for tendons; reduced section steel angles and steel bars have not been used to the same extent. A variety of shear keys are used from stiffened steel angles or plates to SHS dowels. Typically, steel plates reinforced with screws or steel pipe are used to prevent crushing perpendicular to the grain of the column. Only one building has a heel-toe joint at the beam-column interface.

Costello (2013) surveyed post-tensioned timber buildings in New Zealand and found they all adopted an unprotected tendon approach for structural fire design; neglecting the benefits of the post-tensioning. In this approach the tendon is disregarded, resulting in simply supported conditions and the residual section after removing the char layer is used to calculate beam capacity (i.e. Reduced Cross-Section Method).

3 FIRE IMPLICATIONS

There are six components to a post-tensioned connection: tendon, anchorage, dissipaters, shear key, transverse column reinforcement, and the timber elements as shown in Figure 4. The structural fire implications on each component are discussed in Table 1.

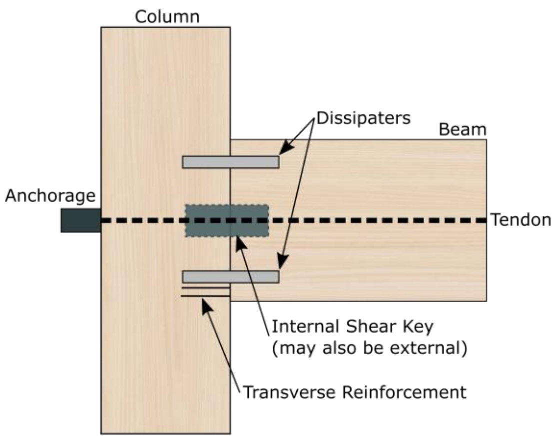


Figure 4: Schematic of a post-tensioned external beam-column connection

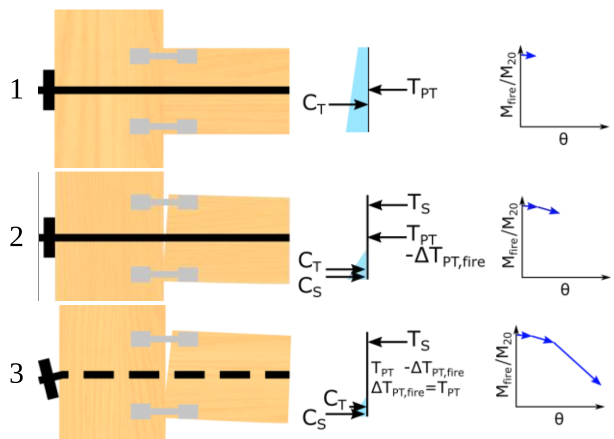


Figure 5: Stages of moment resistance

Table 1: Structural fire implications on components of a post-tensioned timber connection

Component	Structural Fire Implications
Tendon/High strength bar	The tendon provides all the moment resistance in gravity only frames and over half (typically 60%) the moment resistance in seismic frames. Therefore, limiting heat transfer to the tendon to preserve the strength and stiffness of the tendon is essential to maintaining moment resistance at the connection. A gap opening at the beam-column interface may expose a section of tendon to the fire.
Anchorage	In a connection without any gaps the main method of heat transfer to the tendon is conduction through the anchorage. Spellman (2012) conducted fire tests on unprotected anchorages and protected anchorages, and observed that generally tendon force decreased rapidly after a tendon temperature of 200 °C was measured. The unprotected anchorage test showed that 95% of the initial tendon force was maintained for 10 minutes, followed by a rapid decrease in tendon force to almost zero tendon force over five minutes.
Dissipaters	Externally located dissipaters are difficult to protect from fire for two reasons: (1) the dissipater is exposed to fire on most sides, and (2) as the beam chars the embedment strength of the fasteners connecting the dissipater to the beam decreases. If the top dissipaters are on top of the beam, then they have a degree of protection as they may be protected by the floor fire separation (depending on specific detailing).
Shear Key	The shear key is solely responsible for transferring load from the beam to the column. The shear key is of special importance were an unprotected tendon design philosophy has been adopted.
Transverse Column Reinforcement	In the structural fire load case the design loads are less than for ULS at ambient, but preventing crushing at the interface is of no less importance. The effects of any reinforcement on the behaviour of the connection, for example accelerated or localised charring, should be considered.
Timber Elements	Timber elements will char on exposed surfaces resulting in a reduction in the effective cross-section. If possible, there should be no gaps as this exposes more surfaces to the fire (European Committee for Standardization (CEN), 2004; Palma, 2016). The embedment of fasteners will decrease as they conduct heat into the surrounding wood. Timber rapidly losses strength and stiffness at elevated temperatures. By 100 °C the strength and elasticity in tension are approximately 65% and 50% of that at ambient respectively (European Committee for Standardization (CEN), 2004).

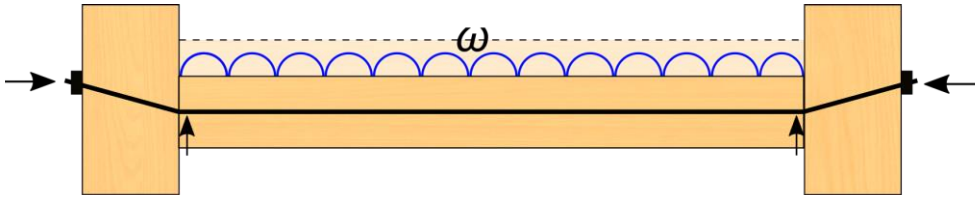


Figure 6: Beam "hanging" on tendon

4 ASSESSMENT

4.1 Structural Fire Behaviour

4.1.1 Connection

Moment resistance of post-tensioned timber beam-column connection is provided by the combined actions of the dissipaters, tendon force and compression of the beam against the column. Fire affects each of these actions uniquely, so losses of moment resistance can be attributed to:

- Tendon losses – Losses due to thermal expansion and reduction in stiffness of tendon due to elevated temperatures, resulting in a decrease in tendon force and elastic modulus as the temperature of the tendon increases. Tendon losses represent the majority of losses as the tendon contributes between 60 and 100% of the moment resistance.
- Section losses – Charring of the exposed timber surfaces results in reduced cross-section and section modulus.
- Dissipater Losses – Losses due to thermal expansion and reduction in strength and stiffness of the dissipater due to elevated temperatures. As a gap opens at the connection, the dissipaters are engaged and provide moment resistance by forming a force-couple. As the elastic modulus reduces a greater connection rotation is needed for the dissipater to provide the same force.

The evolution of moment resistance as the connection is exposed to fire can be described in three stages (Figure 5). (1) In the first minutes of exposure, the moment demand is less than the decompression moment. (2) As the tendon force decreases, the decompression moment is exceeded and a gap opens and the dissipaters are engaged to provide additional resistance. (3) As tendon force decreases to zero, connection rotations increase to maintain dissipater moment resistance. The moment capacity of the connection will eventually be exceeded when dissipaters will not be able to provide sufficient moment resistance.

Shear resistance provided by the shear key is expected to decrease in proportion to the reduction of the effective cross-section. In the case of an exposed shear key, the underside of the beam will char including the portion bearing on the shear key. A downwards displacement will have other effects on the connection, e.g. stretching tendon.

4.1.2 Frame

As a post-tensioned timber frame is exposed to fire, both the rotational stiffness and decompression moment of the connections decrease. Once the moment resistance at a connection at one end of a beam is reached, i.e. decompression moment, moment is further redistributed and the beam is similar to a propped cantilever. Once the decompression moment is reached at both ends, the beam is similar to a simply supported beam.

If the shear key fails then the beam end will displace downwards. Although the tendon is not tight enough to provide moment resistance, it may be possible for the beam to be supported by the tendon (Figure 6).This in turn would provide additional compressive force to the beam. However, this mechanism has not been validated.

4.2 Moment-Rotation Relationship

Van Beerschoten (2013) developed the gravity Modified Monolithic Beam Analogy (MMBA) (Newcombe, 2011; Palermo, 2004; Pampanin et al., 2008) for gravity loaded frames to predict the moment-rotation relationship for a post-tensioned beam-column joint after a gap opens.

Reduced Cross-Section Method (RCSM) is used to design timber members exposed to the standard fire and is contained in many codes (European Committee for Standardization (CEN), 2004; SNZ, 1993). In this approach a thickness, representing the charred timber and hot timber that has "zero strength", is removed from surfaces exposed to the fire. The member is designed using the remaining cross-section which is assumed to be at ambient temperature.

To establish the moment curvature relationship for a Post-Tensioned timber beam-column connection with no dissipaters, the gravity MMBA and RCSM methods were combined. RCSM was used to derive effective beam and column section using char rate of 0.65 mm/min and zero-strength layer of 7 mm (Spellman, 2012).The gravity MMBA procedure was then applied to these members with a protected tendon. If the tendon was not protected then the tendon force would decrease shortly after fire exposure.

The resulting moment-rotation relationship for the connection after various exposures to the standard fire is presented in Figure 7. After 60 minutes the decompression moment has decreased by approximately 10%, whereas the stiffness has decreased by almost half.

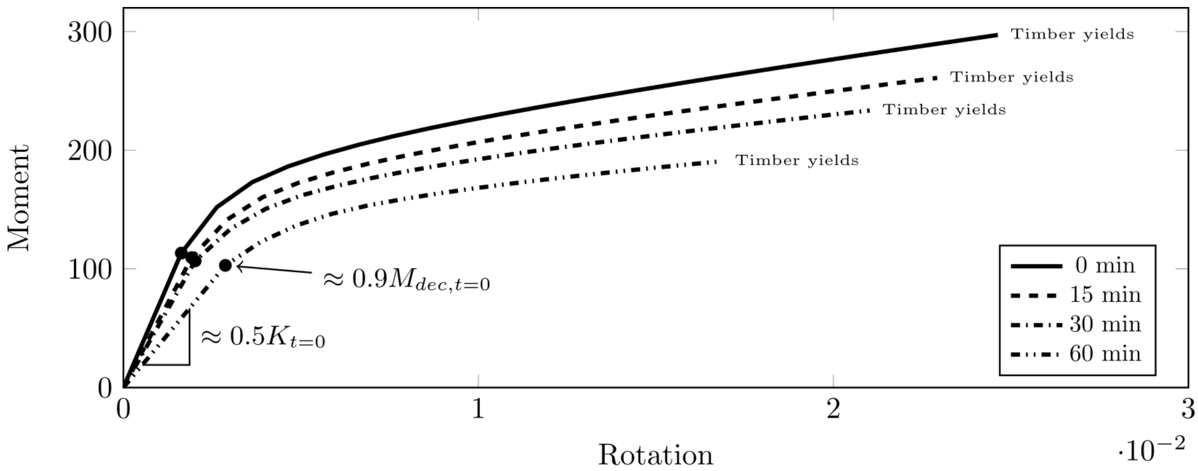


Figure 7: Moment-Rotation of a post-tensioned beam column connection under standard fire exposure

The MMBA method is based only on geometric and material properties of the beam, column and tendon. The effects of detailing are not considered. However, experimental evidence shows that detailing of connections can greatly influence the performance of a connection (Barber, 2018; Maraveas et al., 2015; Palma, 2016; Peng, 2010). From a basic consideration of the behaviour of timber in fire, the following effects are among those worthy of consideration when evaluating the performance of a post-tensioned timber beam-column connection:

1. Reduction of effective timber cross-section – Charring and the zero-strength layer create a reduction in timber cross-section which may also increase the exposure of steel components or decrease bearing area of timber elements. Furthermore, corner rounding may be an important factor if fasteners are located near timber member corners.
2. "slip and slack" created by charring - Bolts and elements that were once tight at assembly may no longer be tight due to charring allowing movement and rotation of elements.
3. Strength and stiffness of fasteners – The strength of bolts and dowels may be sufficient for exposures less than 60 minutes, however, short screws and nails will likely fail earlier as has been observed in tensile joint tests (Lau, 2006; Norén, 1996).
4. Protection of Tendon – The tendon may be exposed to effects of fire through the anchorage and gaps at the beam-column interface which result in a decrease in post-tensioning force.

4.3 Three Examined Typologies

To assess the possible influence of detailing on the fire performance of post-tensioned connections, three post-tensioned timber beam-column connections that represent a range of seismic detailing are assessed for their robustness in fire. Figure 8 shows each connection, including the residual section after a duration of fire exposure, and the characteristics of each of each connection.

Note that none of these connections were designed to have any form of moment resistance in fire.

4.4 Assumptions and Limitations

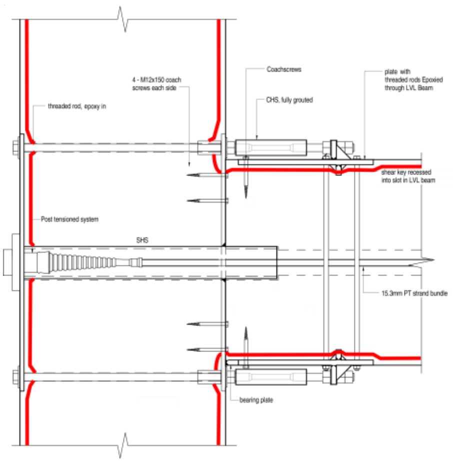
The following assumptions were made in this assessment:

- Valid for standard fire exposure only. The standard fire is specified time temperature relationship intended to allow the comparison the fire resistance properties of different building products. Whereas a real compartment fire has defined growth and decay phases, the standard fire continues to increase in temperature until the element fails. Performance in a standard fire does not guarantee performance in a real fire.
- A lumped mass approach has been adopted for the dissipaters.
- The principles of the RCSM are adopted so the assumptions and limitations of this method apply.
- Linear elastic material properties are assumed.
- Only constant gravity load has been considered.
- Horizontal fire separations have not been accounted for as their performance is system and detail specific and the focus here is on the behaviour of the connection.
- Spellman's test results for an unprotected anchorage are adopted. This is likely conservative for these connections because these connections have larger anchorages, contact area and tendon sizes, and use longer tendons.

4.4.1 Discussion on Steel elements exposed to fire

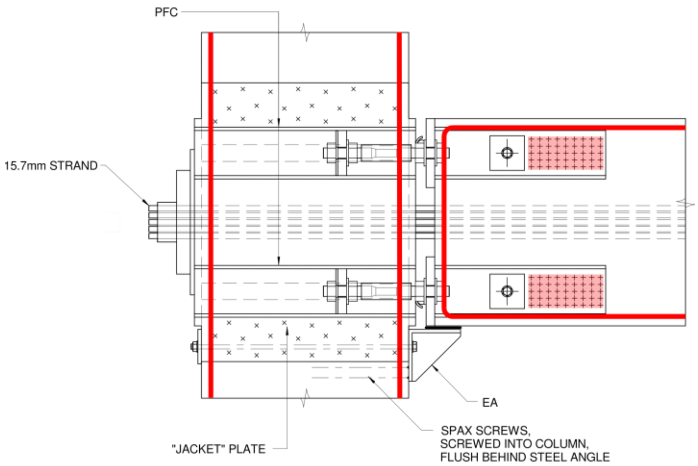
Post-tension timber elements often contain exposed steel elements such as anchorages, dissipaters and steel plates. A lumped mass approximation of an unprotected exposed steel rod indicates that after 30 minutes of exposure it would have approximately 20% of its ambient strength and elastic modulus, and negligible strength and elastic modulus after 45 minutes of exposure. Given the strength of these unprotected exposed elements is low for any practical Fire Resistance Rating the dependability of such unprotected exposed elements should be seriously considered.

A popular method of protecting steel is by applying intumescent paint, which when heated expands and insulates the steel by forming a char layer. Peng (2010), Lau (2006) and Frangi et al. (2010) 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 (2012) 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. Fire protection for connections is based on the fire protection for beams. However, both the actions on the connection and the behaviour of the components of the connections changes under fire exposure. So this approach for protecting connections based on the protection of beams does not offer any guarantee of the performance of a connection when exposed to fire.



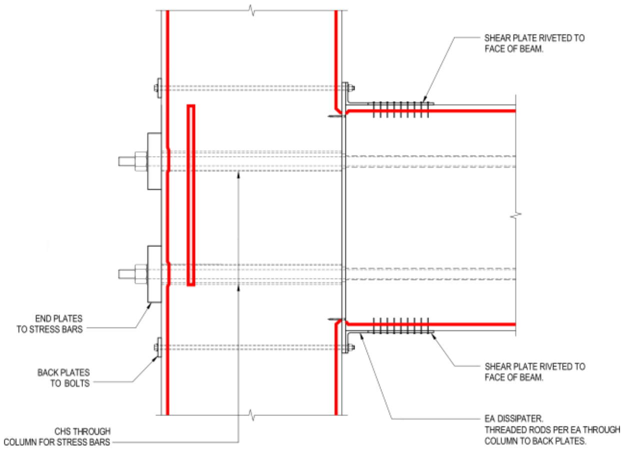
Tendon	15.3 mm strands
Anchorage	Proprietary
Dissipaters	Necked rod grouted in CHS, top and bottom of beam
Corbel	125SHS protrudes into beam
Joint Reinforcement	Steel column face plates, with SHS and coach screws

(a) Connection 1



Tendon	15.7 mm strands
Anchorage	Proprietary anchorage
Dissipaters	Necked rod grouted in sleeve, sides of beam
Corbel	Stiffened steel angle corbel, bolted and screwed into column
Joint Reinforcement	"heel-toe" rocking joints and jacket plates with linepipe

(b) Connection 2



Tendon	2xD36 MACALLOY bar
Anchorage	60 mm steel plate
Dissipaters	Angles with holes
Corbel	Friction and dissipater angles
Joint Reinforcement	Steel column face plate with CHS

(c) Connection 3

Figure 8: Details of connections (red line illustrates residual section after some fire exposure)

4.5 Connection Assessment

4.5.1 Connection 1

The estimated moment capacity of this connection is presented in Figure 9. As the beam chars, the char line will recede below the "shear keys" used to transfer force from the beam to the dissipaters, this mechanism will eventually be lost (shown as i, Figure 10a). As the column chars, the face plates will no longer be bearing on a timber surface, so they will have to bend to maintain contact with timber, resulting in a net increase in the rotations of the connections (shown as ii, Figure 10b).

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 could result from the formation of a longitudinal crack in the "webs" of the beam on either side of the SHS.

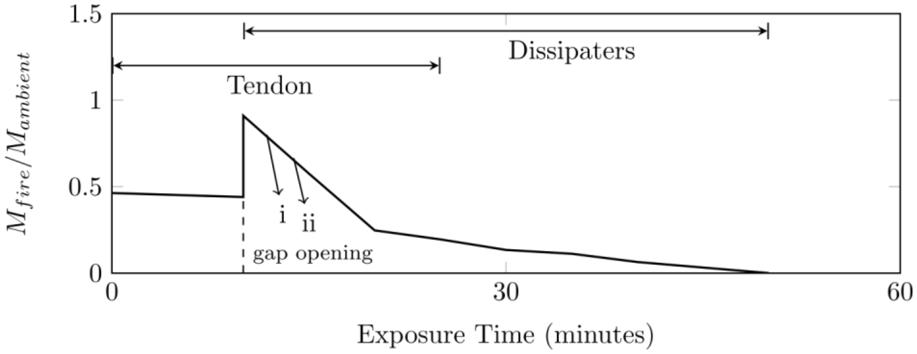


Figure 9: Estimated moment capacities of Connection 1 subject to standard fire exposure

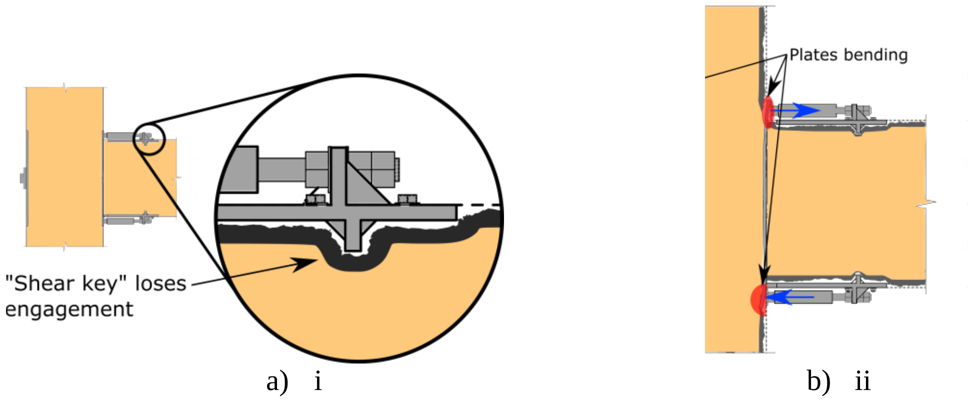


Figure 10: Illustrations of Connection 1 during fire

4.5.2 Connection 2

The estimated moment capacity of this connection is shown in Figure 12. The gap between the beam and column face directly exposes the tendon to the fire and allows charring on all faces of the column and the end of the beam. The beam surface that bears on the shear keys is exposed to the fire. As this surface chars, a downwards displacement must occur to maintain bearing on the shear key. This vertical displacement will induce additional deformations in the connection and create secondary effects reducing moment capacity (shown as i, Figure 13a). The closely spaced rivets, will create superimposed thermal fields in the timber, reducing embedment strength and stiffness, and locally accelerating charring, reducing the load transfer mechanism from the beam to the dissipater (shown as ii, Figure 13b). The jacket plates which were tight at ambient develop slack as the column surface recedes due to charring, resulting in increased deformations to maintain the moment resistance (shown as iii, Figure 13b).

The shear forces are transferred from the beam to the column through the stiffened steel angle that is bolted through into the column, with a bearing strip which will burn shortly after exposure. As the surface of the column recedes, the bolt becomes loose and the angle rotates to maintain resistance to the eccentric loading. Also downwards displacement of the beam occurs to maintain bearing against the shear key. The stiffener is exposed directly to the effects of fire, so deformation of the shear key could occur, although deformation of steel plates in Steel-Wood-Steel connections is not often observed (Maraveas et al., 2015).

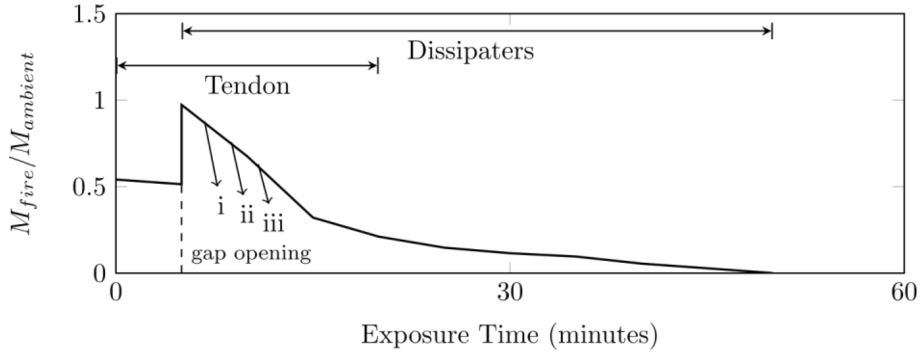
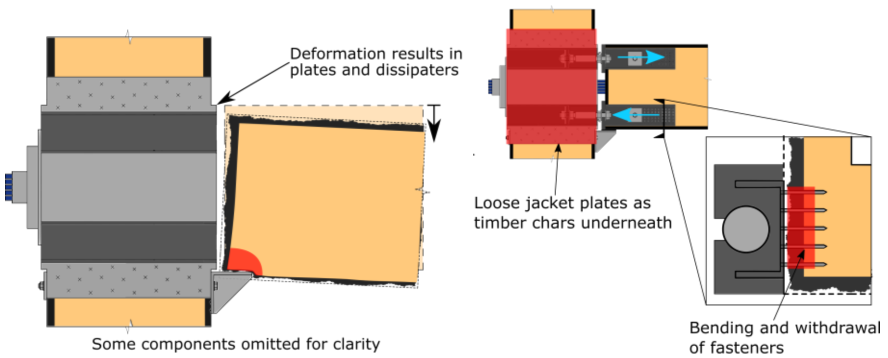


Figure 12: Estimated moment capacities of Connection 2 subject to standard fire exposure



a) i

b) ii and iii

Figure 13: Illustrations of Connection 2 during fire

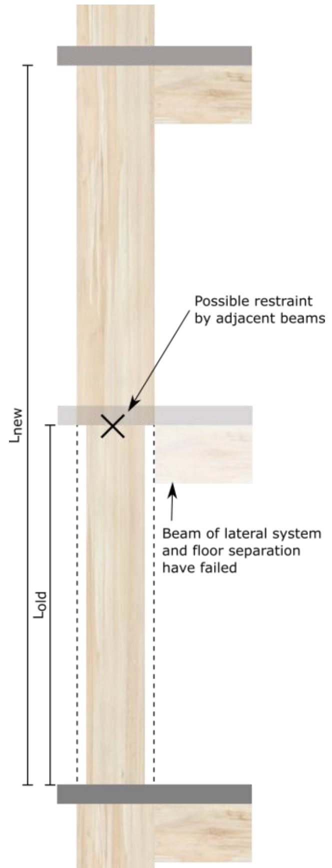


Figure 11: Illustration of doubling of effective length

4.5.3 Connection 3

Other than the compressive force induced by the tendon, the beam in Connection 3 is only significantly loaded under lateral actions. The estimated moment capacity is expected to be similar to that of Connection 1. The bars are only exposed at the end and the dissipaters are fixed to the top and bottom of the beam. Although the angle dissipaters are fixed to the beam with rivets like Connection 2, these will experience bending and withdrawal as charring reduces embedment and will reduce load transfer to the dissipaters, similar to Connection 1. Unlike the other connections, there is an exposed steel plate that passes through the column connecting adjacent beams and braces. This exposed plate will “attract” heat from the fire and conduct it to the centre of the connection, creating a thermal field in the centre of the connection.

Steel bars will not elongate to the same degree as strands as they are stressed to a lower degree, so losses due to change in elastic modulus and thermal expansion may result in a faster loss of tendon force than for strands. Moreover, because steel bars are stiffer than strands they may not exhibit the catenary effect shown in Figure 6, but may have a brittle shear-dowel failure.

4.5.3.1 Interdependency of Gravity and Lateral Systems

In Connection 3, the beam does not carry a significant amount of gravity load, and so can be considered to be only part of the lateral system. However, the column does have an axial gravity load. Often in structural fire design the gravity and lateral systems are assumed to be mutually exclusive, so the lateral system can be ignored. However, this may not be an appropriate assumption.

Using the example of Connection 3, if the beam fails, then the lateral restraint of the column in one direction is lost, effectively doubling the effective length of buckling of the column (illustrated in Figure 11). Furthermore, the floor fire separation may be breached, allowing fire spread to the floor above. If the floor separation fails then diaphragm action may not be able to restrain the column through the orthogonal beams. This example illustrates that although the lateral system may not be load bearing in the structural fire load case, the behaviour of the lateral load system in fire should be considered.

5 CONCLUSION

The robustness of seismic detailing for performance in fire has been assessed. It appears that stiffness of a connection with a protected anchorage decreases with fire exposure at a much faster rate than the decompression moment. Protection of the anchorage is an important consideration for moment resistance of a post-tensioned timber connection subjected to fire exposure. From a structural fire perspective, using internal shear keys, minimizing gaps, using as little exposed steel as possible and using strands rather than bars, are expected to give better performing connections when exposed to fire in the structural-fire load case.

Sectional analysis of post-tensioned beam-column connections during fire is limited because localised damage can vary the overall strength, stiffness and ductility of the connection considerably. Therefore, experimental validation of the performance of post-tensioned beam-column connections is needed to develop a more comprehensive understanding of their behaviour and to account for their realistic contributions to overall frame performance.

ACKNOWLEDGEMENTS

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