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ABSTRACT. Modern architecture is striving for large open spaces, which has resulted in the development of design methodologies such as the Improved Travelling Fires Methodology (iTFM). This methodology is only applicable in large open spaces where flashover may not occur. Ongoing research focuses on improving iTFM through large experiments and on improving the design of advanced material constructions. This paper studies the effect of various design fires, including traditional uniform fires and for the first time, iTFM on flat-plate unbonded post-tensioned (UPT) concrete, a structural system which allows long flooring spans (>8m). A structural model of high temperature unbonded prestressing tendon relaxation was previously developed and predictions were compared for the first time with previous fire experiments. A novel case study based on a real building was then analysed using the model to propose an acceptance criterion based on the longest tendon length where rupture and extreme stress relaxation is prevented. Two types of steel, fabricated according to BS 5896 and AS/NZ4672, were analysed for various unbonded lengths (between 8.15m and 65.2m). Emphasis is on the localised high temperatures of travelling fires and the unique deformation mechanisms present when prestressing steel is heated locally. The relaxation model has been validated for negligible slab deflection, allowing for discussion on the difference between the impact of travelling and uniform fires on the design of UPT concrete. The tendon rupture and relaxation analysis have novelty in showing that slow moving fires are more onerous structurally than fast moving or uniform fires thereby providing the practitioner with provisional guidance to defining critical design thermal boundaries. Results also demonstrated that tendons in similar designs require consideration to be limited in unbonded length. The development of new acceptance criteria herein is the first step in producing a generalized criteria for better informed structural fire design of UPT concrete exposed to fire.

Keywords: Travelling fires, post-tensioned, prestressing steel, concrete, fire

1 INTRODUCTION

Contemporary architecture demands structural engineering systems which can permit large open spaces (examples include airports, theatres, stadia, condos, offices, etc). A number of real fire events have shown that fires in large open spaces will not result in uniform thermal exposure to the structure as is conventionally assumed. In reality, fires can exhibit characteristics that demonstrate a "travelling" behaviour. In the transition towards objective- and performance-based design in many jurisdictions, it is becoming more important to model and understand structures exposed to a range of likely and expected design fires. This is typically done by selecting realistic design fires (based on engineering judgement, local practice or probabilistic studies) to ensure a robust structural design for fire.

The fire resistance design of structures is predominantly based on the adoption of the ISO 834 standard fire test (or equivalent ASTM E119) that has its known origins more than 100 years ago. The adoption of the standard fire makes it possible to establish a qualification-based fire resistance metric for classification purposes, and the comparative performance of structures and other building components (doors, fire stops etc). The authors acknowledge that it does have its place for fire design purposes, however, it also has been recognised that the standard fire does not necessarily resemble the real fire behaviour that structures can experience. When considering the real fire behaviour expected in real structures, a range of responses should be expected depending particularly on the architectural geometry which differs from the compartments used to develop the standardized fire exposure.

The abundance of larger open compartments led to the development of design methodologies such as the Improved Travelling Fires Methodology (iTFM) [1]. In this types of methodology, a travelling fire is a localized fire that moves within the large open compartment, composed of a leading edge, which is governed by flame spread, and a trailing edge, which gradually burns out as the fuel is consumed. The iTFM methodology assumes a non-uniform temperature distribution through the compartment and assumes that the fire travels in a one-dimensional direction through the compartment, at a constant spread rate. The size of the fire can vary depending on the fuel and ventilation parameters and therefore a family of fires (based on deterministic or probabilistic selection) is always considered to identify the most critical case when travelling fires are used in real design. The compartment is divided into two moving regions to calculate the non-uniform, transient temperatures: the near-field and the far-field. The near-field is where the flames are impinging on the ceiling and the compartment experiences the peak temperature. A flapping angle, described as the angle from the main axis of the flame, is included to capture the possible fluctuation of the impinging flame. The length of this region identifies the fire size, usually expressed in percentage of floor space or a flame spread rate. The far-field is the smoke region which corresponds with the pre-heating or cooling of the structural elements. As the distance increases from the near-field, the temperature of the smoke decreases due to the mixing with the air within the compartment. The far-field model is the analytical expression of Alpert's correlation with the limitation that it cannot exceed the near-field temperature.

Research studies on composite steel concrete structures have shown that “short, hot” and “long, cool” uniform fires can lead to different responses [2] which cannot be observed in standard fire tests. Rackauskaite *et al* [3, 4] compared the influence of travelling fires and uniform fires in a steel multi-storey building and determined that travelling fires can introduce a range of responses. Thermally, travelling fires are more severe, and structurally, it depends on the specific heating scenario. Similar conclusions were reached by Law *et al* [5] for a reinforced concrete structure subjected to uniform and travelling fires. For post-tensioned concrete (PT) structures, Gales *et al* [6] have shown experimentally that a localised fire can be more onerous in comparison to a uniform fire (an assumption of the standard fire test) due to the stiffer restraint of the prestressing steel from the unheated areas.

Methodologies capturing the possible behaviour of fires within large compartments are greatly needed in Fire Safety Engineering. In North America, where the design of steel connections is rationally being considered, there have been preliminary considerations of travelling fires [7]. Figure 1 details the evolution of travelling fire models, with a focus on the methodology used for this paper, iTFM. The development of these methodologies has led to the incorporation of travelling fires into the design process, shown through the UK standards PD7974-1 and -3 which now reference travelling fires as representative of industry practice [18,19]. The origins of travelling fires stem from the observation of fires in large, open-plan enclosures, where the fire was seen to travel within the available space and last for extended durations. One of the first approaches at simulating a travelling temperature-time curve was developed by Clifton [20] in New Zealand. This methodology divided each compartment, or fire cell, into four distinct regions (preheat, fire, burned out, and smoke logged) that were subjected to parametric fire curves individually and sequentially. The Travelling Fires Methodology (TFM), as known today, was developed by Stern-Gottfried and Rein [12,1] to capture the highly non-uniform and transient heating within large compartments, building on the research of Rein *et al* in 2007 [8]. The TFM methodology was developed further to become iTFM in 2015 by Rackauskaite *et al* [4]. Dai et al. [21] developed an Extended Travelling Fire Methodology (ETFM) that combined Hasemi’s localized fire model with a calculation for the smoke layer. This method accounts for the accumulation of hot gases within the compartment.

The iTFM methodology was chosen for this study as it was developed for design purposes and is based on conservative assumptions. It is currently the most applied methodology for large compartments used in industry, with many new iconic buildings using it as part of their design. It is purposefully not aiming to be predictive due to the limited experimental evidence from large compartments. A very limited number of experiments have been undertaken in large enclosures and therefore, fire dynamics in such spaces require further research and a number of experimental campaigns are currently pursued by researchers internationally. It should be noted that, at this point, the iTFM design methodology, or any other design fire methodology for large enclosures, are not yet validated. The authors’ study acknowledges other travelling fire methodologies could be considered. It is acknowledged that the choice of building and design fires are illustrative with the focus on thermal performance.

To date, the real structures that have been designed using travelling fires methodologies are open plan offices of largely steel composite or traditional reinforced concrete configuration – mostly because the structural fire model validation exists for these structure types, i.e. the Cardington Studies of 1995 and 2003 [2]. However, other structural systems which also enable large compartmentation have seen limited application of the travelling fires methodology in practice. Post-tensioned concrete, which uses highly stressed prestressing steel tendons to achieve long spans, is such an example that has received limited research attention to this type of design fire. This is natural as it is difficult to study due to effects from Load Induced Thermal Straining (LITS) [6]. The application of iTFM could be critical as previous experimental research has indicated that PT concrete has specific vulnerabilities to localised heating that can cause its high strength steel reinforcing to rupture – specifically if the steel is left unbonded to the concrete and rather than bonded as demonstrated in a previous literature review on real fire case studies and experimental evidence [6]. Additionally, corresponding reductions in stress relaxation and tendon failure hampers the structures ability to balance applied loading over long spans. By including travelling fires within the family of design fires and identifying the most critical heating scenarios, it becomes possible to design PT concrete structures so that failure mechanisms through tendon rupture or stress relaxation are minimized.

The authors' focus is on the development of simple analytical tools. Analytical tools are simplified tools for practitioners that allow for quick simple but conservative analysis to be conducted. For a complex structural system such as unbonded (U)PT, an analytical tool would allow for a more conservative and certain design until further research and complex design guidance can be produced that improve our understanding of these structures in fire conditions. This study holistically aims to develop the capabilities in modelling PT concrete for prestressing steel tendon relaxation and rupture by proposing a limitation on tendon length in design, rationally identifying the critical design fires necessary to ensure this type of structure's fire resilience, producing an analytical tool and concluding with a priority listing of research needs for PT concrete structures for industry and researchers moving forward.

2 RESEARCH INTO UNBONDED POST-TENSIONED CONCRETE IN FIRE

Unlike PT concrete structures, the behaviour of reinforced concrete structures at elevated temperatures has been extensively studied in previous literature. Khoury [22] provides an overview of the concrete material response at elevated temperatures as well as of the methods for achieving fire resistance for concrete structures. Dotreppe *et al* [23] present an experimental study on the fire behaviour of concrete columns, and Lim [24] presents experimental and numerical studies of two-way slabs. A numerical model for the fire resistance of reinforced concrete beams is presented by Kodur and Dwaikat [25]. A multiaxial constitutive model for concrete at elevated materials has been developed by Gernay *et al* [26], and Molkens *et al* [27] have examined numerically the compressive membrane action developed in concrete slabs. However, research on PT concrete exposed to fire, and more specifically unbonded PT, has received little attention and therefore has developed a degree of documented uncertainty towards its performance in fire. A series of fire test for one-way spanning unbonded PT concrete were performed by Bailey and Ellobody [28] with the aim to understand its fundamental behaviour.

One of the most comprehensive structural fire experiment programmes that permits the study towards rational design of PT concrete in fire is the set of experiments performed by Gales *et al* between 2011 and 2013 in the United Kingdom (see [6]). These experiments included testing of three multiple bay, realistically-restrained concrete slabs heated locally with radiant heaters that permitted a highly controlled and repeatable heat exposure between experiments. Two experiments were unbonded (noted as A and C) and one was bonded (noted as B). An unbonded tendon is free to deform along the length of the installation in the slab and is easier to construct, whereas bonded is more difficult to construct and can only deform locally. To study the effect of Load Induced Thermal Strains (LITS), two of these slabs were heated twice (A and B) while the third slab (C) was only heated once [29]. LITS occurs in concrete structures when they experience high temperatures for the first time and can cause an upward deflection in concrete slabs which would otherwise seem counterintuitive given that the deflection is contrary to the deflection of the slab due to the applied loads. Including a secondary heating has been shown to result in no additional LITS strain effects observed, provided that the stress and heating levels of the first heating are not exceeded. The experimental specimens were exposed to a long-cool fire under expected service load conditions. The first heating cycle included heating the slab until the prestressing steel reached approximately 350°C, a commonly used critical temperature in design standards, using a constant radiant heat flux of about 35 kW/m^2. For the slabs receiving a second heating cycle, the slabs were heated using the same heat flux until the prestressing steel reached 427°C after they had cooled to ambient temperature. A schematic of these experiments is illustrated in Figure 2. These slab experiments (A through C) did not show tendon failure, but they did show significant stress relaxation (nearly 50%) which has an effect on the load balancing capabilities of the concrete slab – particularly in the unbonded tendon configuration. The details of the PT concrete experiments can be found in [6]. The reader is referred to that work for details on the experiment procedure and its results. It is beyond the scope of this study to provide specific experimental details of that research programme as these can be found elsewhere in [6, 29-31]. The published accounts of these experiments advocate for modelling to be considered, but to date, limited attempts have been made to utilize these tests for modelling purposes to the knowledge of the authors, mainly due to the highly complicated nature of LITS observed in the tests which also deserves additional study.

Recent industry reports, such as NIST 1188 [32] and ASCE Manual of Practice (MOP) [33], have called for additional research into the performance of PT concrete in fire as well as a more complete understanding of the realistic thermal boundary these structures are expected to encounter in fire. The industry consensus research needs from the ASCE MOP are highlighted in Table 1 and are largely adapted from NIST 1188 guidance that followed the proposed research needs identified within reference [6]. The study herein starts to address these critical research needs, focusing on a valid acceptance criterion (tendon failure and relaxation) for UPT concrete in fire against a range of realistic fires. The emphasis of this study is not to investigate spalling or other thermal mechanisms, such as LITS. Instead, the focus is on research needs 2 and 3 as identified in the document and Table 1: local and global behaviour; and analytical tools and analysis to lead to novel design tools. It is expected that future research will aim to address these needs and indeed various other organisations, such as RILEM, are currently studying these phenomena [33]. Unbonded PT concrete fire research is an important and needed area of investigation which has received little contemporary attention, in part due to its difficult to study nature. If prestressing steel tendons do fail, they have the potential for significant damage and life safety concerns, as shown in the case studies examined in [6].

3 TENDON DEFORMATION MODEL

This study begins with a thermal and tendon rupture computational model validation against the previously mentioned experimental data set available in literature [6]. This consisted of a three span unbonded PT concrete flooring system exposed to localised heating (A and C denoted in Section 2). The analysis assumes a thermal boundary condition representative of heating conditions experienced during all three slab experiments (see [6]). There was some minor variation in heating along the slabs, therefore a representative thermal boundary condition was developed which adequately describes the adiabatic temperature-time exposure to the slabs induced in all experiments. The boundary represents a best-fit, least-squares approximation of the average of eight thermocouples used on the exposed soffit in each of these experiments (see Figure 3 for extent of heating). Using this heating curve and a heat transfer analysis through concrete (detailed below), temperature profiles through concrete slabs, including at the position of the tendon, can be determined.

The heat transfer analysis in the concrete was performed using a finite difference elemental energy balance, based on fundamentals described in [34]. The input parameters considered carbonaceous aggregates, a 95 mm thick slab, and a 4% moisture content. The analysis incorporates the variations of thermal properties of the concrete and the effects of moisture due to spatial and temporal variations of temperature. The thermal properties – thermal conductivity, density, and specific heat – were calculated for the different depths within the slab for each time step during the fire exposure. This led to changing temperatures at each location with time, which affects the properties that are dependent on temperature, referred to as thermal properties. The reinforced concrete slab is assumed to be horizontally infinite, allowing for the edge effect to be ignored. At the fire-slab and slab-air (unexposed face) interfaces, the three modes of heat transfer are considered (conduction, convection, and radiation). It should be noted that the steel reinforcement is assumed to have no influence on the heat transfer behaviour herein, both for the heat transfer through the slab and the longitudinal heat transfer. This has been validated for bonded steel reinforcement but has also been shown to be very minimal in [35]. The through thickness slab temperatures were estimated and used to determine tendon temperatures (at axis distance). The temperatures obtained from the thermal modelling were compared to the experimental thermocouple temperature observations of the average surface temperature of the exposed soffit, tendon temperature, and the top of the slab at mid span in Figure 3a. The analysis indicated difficulties in heat transfer modelling with the moisture in concrete at the early stages. Since the heat transfer model does indicate good agreement at temperatures known to be important for creep of prestressing steel (i.e. in the range of 300°C), the thermal modelling is satisfactory for predicting the temperatures the prestressing steel embedded in concrete will experience. Using the tendon stress relaxation (which accounts for thermal relaxation and creep) and rupture modelling (degradation of strength) found in detail within the validated models of Gales *et al* [36], the expected stress relaxation was computed for BS 5896 prestressing steel for Slabs A and C. It should be noted that other stress relaxation models and parameters exist in recent literature, however, as this work builds upon reference [6], only the parameters and material studies from that research are utilised herein. As illustrated in Figure 3b, the stress relaxation calculated was then compared to the tendon stress observed in the experiment.

Slab A showed a maximum difference of 47 MPa (5% error) between calculation and experimental results of stress relaxation, while Slab C showed a maximum difference of 27 MPA (3% error) during the first three hours of testing. The observed accuracy of the structural stress relaxation model is in line with that demonstrated in Gales *et al* [36] (i.e. less than 6% error). The experimental data from Slabs A and C show little stress relaxation at the start of the experiments, whereas the structural modelling predicts clear relaxation during this period. This could be due to tendon elongation caused by thermal bowing at the start of the experiment, which would counteract (by increasing stress) stress relaxation induced by heating. However, this is not accounted for in the current stress relaxation modelling since the model does not consider structural deformation response. As shown in [6], deflection (measured as 12 mm maximum) was relatively small considering the spanning length (the length of the heated span was 4140 mm) and therefore the resulting elongation of the tendon from deflection would be expected to be small and within the error observed. The comparison between modelled and experimental tendon stress relaxation of Slabs A and C however supports the use of the utilised creep parameters from [36], allowing the stress relaxation model described to be used on the case study described in Section 4 and subjected to limitations described in Section 5.

4 THERMO-MECHANICAL ANALYSIS OF AN UNBONDED POST-TENSIONED CONCRETE STRUCTURE EXPOSED TO FIRE

4.1 Building Case Study

The structural design of the case study considered is based on an existing PT concrete building, designed by ARUP in accordance to UK building standards. Figure 4 illustrates the 2-storey structure and Figure 5 illustrates its structural layout. It should be noted that only the structure's specific features necessary to this paper are shown. The authors' state that the reader should assume the structure to be hypothetical in nature. Additional structural details would need to be considered for this building in order for it to be built, and the analysis is hypothetical based on several changes in the building's design. The ultimate design as-built and as-recorded is not within the scope of this paper.

The structure has column-to-column spans of 8.15m, with over 100 prestressing steel tendons varying in length from 6 to 32m. The height of the compartment to the soffit is 3.6m. Figure 6 shows the parabolic tendon profile within the typical 250mm thick slab, which has a 35mm tendon axis distance at the supports and mid-span. To address the needs and limitations of this study, certain design aspects of the structure have been modified by the authors from those of the actual structure. The slabs are not representative of a real building, however, they do provided value as research tools to investigate various physical responses.

The two key variables in this study are the design fires and the tendon length. For each analysis, the authors considered a fire origin from the west side of the structure (right façade of the structure shown as Grid N) unless the fire was specified as uniform. The tendon length was varied in each analysis, originating from Grid N and projected the tendon length along Grid 5 for each analysis. The study considered eight simulated tendon lengths for each design fire. The tendon length was defined as starting at 8.15m and using the column-to-column spacing (8.15m) as the interval. The longest tendon considered was 65.2m, which was considered by the authors as an upper bound limit for PT construction.

Two tendon types (I and II) as defined in Section 4.3 are considered. A total of 128 fire-tendon fire scenarios were analysed and discussed herein (Section 4.2 discusses the design fires used). Table 2 illustrates these scenarios.

There are limitations that need to be considered prior to the presentation of the case study's results. First, the authors do not attempt to investigate the thermal-mechanical response of the concrete slab. Very limited progress has been made in literature on this subject owing to a lack of available experimental programmes to allow its validation. However, it is encouraged that researchers and practitioners attempt these validations in future research using the experiments as in [6]. Transient thermal straining, more specifically Load Induced Thermal Strain (LITS), has received some attention by researchers in recent literature [37]. This straining mechanism appears to be important in describing the deformation response of PT concrete slabs which thereby make performance criteria difficult to establish [32, 33]. Tendon rupture and stress relaxation as a function of tendon length and fire exposure is the current focus of this investigation, analysed through the case study herein. Future research could consider other performance criteria and what criteria govern the design depending on the structural layout.

4.2 Design Fires

The case study structure presented herein was thermally evaluated for a range of design fires, including the uniform ISO 834 fire, uniform parametric fires and non-uniform travelling fires, to determine the concrete's thermal response as a function of time (see Table 2 for configurations considered). All fires were selected qualitatively based on experience from previous studies by the authors to reflect a typical design process and a broad range of what the compartment may experience. The heat transfer analysis of structural members was calculated using the nonlinear finite difference calculation as defined in Section 3. The inputs to the calculations, such as a moisture content of 1.5%, are standard values obtained from EC1. It was assumed that explosive spalling is unlikely to occur (though should always be of attention) as the XC1 class is considered with moisture content less than 3% and the concrete strength is below 55 MPa, in accordance with EC2-1-2. The most critical fires that may cause tendon rupture, based upon region specific material usage, are identified using the validated and highly conservative region-specific tendon rupture models from [36]. These models are further described in Section 4.3 and are used to identify at which temperature the tendons are expected to rupture.

Seven varying design fires were considered in this analysis to cover a range of different structural behaviour: a short hot parametric fire, a long cool parametric fire, and four travelling fires, and the standard fire (ISO 834). Although the standard fire is a prescriptive exposure that is not representative of reality, it was included due to its frequent use in design and, more importantly, to provide a benchmark for comparison. The design fires were selected based on what is considered current practice in the industry, following the recommendations in the UK standards PD7974-3:2019 [19]. For all design fires, fire load density and heat release rate per unit area were taken for office type accommodation in accordance with EN1991-1-2 and PD6688-1-2 [38] (511 MJ/m^2 and 290 kW/m^2 respectively). As Rackauskaite *et al* [3] have shown, travelling fires of different speeds can lead to different thermal and structural responses. As a result, for the purposes of this study, four representative travelling fire scenarios were qualitatively considered of two slow, a medium and a fast travelling fire (2%, 5%, 20% and 40% fire sizes with respect to the compartment size respectively) to ensure a range of responses. A flapping angle of 6.5° was adopted [3]. Figure 7 illustrates the time-temperature curves for the four travelling fires across the entire structure's floor. The two parametric fires used were determined to be representative of different structural behaviour, with a hot-short fire and a long-cool fire, found by varying the available ventilation as discussed by Lamont *et al* [39]. The thermal exposure of the two parametric fires are shown in Figure 8. The standard fire was limited to a two-hour thermal exposure, as this would be the typical fire rating prescribed for this system.

4.3 Tendon Analysis

After the thermal boundary conditions (from design fires) and concrete temperatures were calculated (from the same nonlinear finite difference calculation method introduced in Section 3), the corresponding tendon temperatures were extracted from the thermal analysis. These temperature profiles were then inputted into a stress relaxation model described in reference [36]. The model was developed by calculating the thermal creep strain increase that occurs when the steel is exposed to a changing high temperature during a set time interval over a set length of the steel. The model then sums the strain increases over the length of the tendon to calculate the overall relaxation of the unbonded tendon stress. Two types of steel were analysed: Steel I (fabricated according to BS 5896) and Steel II (fabricated according to AS/NZ 4672). The prestressing stress and strength analysis considered both these steel types to assess whether region specific influences in manufacture may control the response of the unbonded prestressing steel. Once the prestressing steel stress relaxation was calculated using thermal properties from [36], significant differences in behaviour emerged with respect to relaxation and the introduction of the travelling fire as the design thermal boundary. Figure 9 and Figure 10 briefly illustrates the response of one common 32.6m long unbonded prestressing steel tendon exposed to the travelling fires examined.

The stress relaxation results first imply that Steel II, in all cases, has less stress relaxation than Steel I. Figure 11 illustrates the comparison of this for a 32.6m tendon. When various travelling fires are considered, they seem to converge by two hours to a below-acceptable stress state regardless of the make of steel (I or II). The acceptable stress relaxation was assumed to be 50%, in this case 600 MPa (defined by stress relaxation without collapse observed in reference [6], this value should be used with caution as describe in Section 4.4). For most cases, the stress relaxation calculated due to travelling fires is more significant than a uniform fire exposure (parametric and standard) for the unbonded prestressing steel tendon. This is likely as the travelling fire leads to longer heating durations and therefore higher steel temperatures, but further studies are required to assess the influence of travelling fires in experimental reality. Based on the acceptable stress state described, the temperature of the steel which would cause this stress state can be identified. Figure 12 illustrates the temperature at which stress failure occurs with varying tendon lengths.

When tendon strength reduction is considered, the more onerous fire emerges as the small travelling fire (5%) with respect to tendon rupture. In strength reduction, tertiary creep is simplified in the calculations and rupture is implied to occur during secondary creep when tendon stress exceeds time dependent strength. The omission of tertiary creep in the calculations produces conservative results since only considering secondary creep decreases the stress relaxation and therefore increases the stress level and predicts tendon rupture sooner. To fully define strength failure including tertiary creep, further research is needed, however, the creep model has been shown to be accurate in [36]. Rupture is not predicted to occur for Steel I but is predicted for Steel II between 342°C and 400°C. Strength reduction due to increasing temperature was calculated using the recommended reduction factors in Table 3.3 in EC2-1-2 [39], which are considered typically conservative with respect to available literature [36]. However, Robertson *et al* [41] indicated very significant strength reductions as a function of heating time (even when specimens had no load during heating). It is prudent that, if a long travelling fire is considered, the corresponding strength reduction should also be considered more carefully. This is because, as prestressing steel is heated for a longer period at the same elevated temperature, it can lose additional strength. Figure 13 shows the temperature at which tendon rupture is assumed to occur with varying tendon lengths.

Additional analyses were performed with the tendons starting on Grid M (instead of N) and the fire igniting along Grid N for travelling fires (same as previous analysis). This was done to examine the effect on tendons being offset to the start of the fire, which could occur in a realistic structure. The results, however, did not vary significantly, with no differences for the strength failures and differences of less than 5°C for stress failures. The uniform fires did not result in any differences since the location of the tendon has no impact on the thermal exposure it will receive. Since the results are not significant, these analyses were not included in Table 2 and any further discussion.

4.4 Acceptance Criterion for Structural Design

Figure 12 and 13 can be used for a simplified basis of establishing an acceptance criterion for cases similar to those examined. These figures identify the temperatures which would cause failure of the structural system, using the assumptions that stress failure occurs when 50% stress relaxation is reached, and tendon rupture occurs when the stress exceeds the strength of the tendon. The criterion can be appropriately defined by the stakeholders. The authors assumed a 50% stress relaxation as definition of stress failure as this has been shown not to cause collapse of a slab within experimental tests [6]. It is anticipated in reality that the value may be considered higher but the practitioner can easily follow the methodology herein to establish a separate set of contours for a higher allowable stress relaxation state.

Figure 13 illustrates the strength failure temperature versus the length of strand, showing that travelling fires are more critical than parametric and uniform fires (which did not cause tendon rupture) for this specific case study and the design fires examined. Only Steel II experienced tendon rupture, and only when exposed to travelling fires. Figure 13 illustrates that, as the tendon length increases, the most critical failure temperature decreases. This is expected as the fire becomes less uniform, but the defining critical temperature can drop below 340°C. When designing PT concrete slabs, the Eurocode uses 350°C as the critical temperature which would cause failure of the slabs while the North American guidance uses 427°C [42]. The concrete design is done so that the steel tendons only reach these critical temperatures once the specified fire rating is reached. Figures 12 and 13 allow the designers to rationally decide the longest tendon length that should be supplied using the contours created (based upon the criterion of 50% stress loss and Eurocode strength reductions). The authors subsequently advise that if the critical temperature of 350°C is used for prestressing steel, the maximum tendon length specified for Steel II should be 42m. If a critical temperature of 427°C is assumed, as in North

American standards [42], the maximum tendon length is less than 8m due to tendon rupture. Since Steel I does not rupture, the standard chosen would be of no consequence.

For this specific case study and the design fires examined, a slow travelling fire (5%) is the critical design fire that is required for tendon rupture. The result is expected as the fire leads to longer durations that the concrete will be exposed to heat. While a 2% travelling fire would lead to a longer thermal exposure, the concrete is exposed to the near-field (peak temperature) for a very short period of time and allowed to cool in the far-field for longer durations.

5 LIMITATIONS AND GAPS IN LITERATURE

The results obtained as part of this study are only the first steps to defining the critical design thermal boundaries for UPT concrete flat-plate slabs. Additional research is required, which will require additional tendon modelling to confirm. These include but are not limited to travelling fires moving perpendicular to the current analysis since the tendons span multi-directional (east west, north south, and diagonal); tendons that are multi-banded; bond type; various tendon profiles; and sensitivity of concrete properties. The undertaken structural modelling does not consider the thermal-mechanical relations of load induced thermal strains (LITS), and thermal gradients which invoke complicated deflection mechanisms or load shedding from tendon failure. All of these research advancements are beyond the scope of the current study but necessary to consider when formulating generalized design considerations for highly complex prestressing steel.

More experiments are needed to fully understand the tendency of spalling and its effects on the tendons within the slab [43]. Prestressing steel tendons, compared to mild steel reinforcement, can experience more damage under high temperature. If spalling were to occur, the tendons would be exposed directly to the severe heating and the significance of such heating is still unknown. A more complete understanding is needed to inform a more in-depth tendon deformation model.

Further follow-up studies from this work could consider the consequences to the reduction in flexural and shear capacity of UPT concrete slabs as a result of localised tendon rupture. This may also require further experiments to validate, particularly in defining the amount of required bonded non-prestressed reinforcement to prevent collapse. Preliminary insight into stress relaxation and tendon rupture effects on shear and flexure can be found elsewhere in Gales *et al* [44].

Future research will inform practitioners how to structurally design these assemblies to prevent tendon rupture and improve upon probabilistic fire design approaches for this structure type. The research can lead to the needed acceptance criterion to help enable objective- and performance-based strategies, meeting ASCE MOP goals. The current tendon stress relaxation model has only been validated for unbonded prestressing steel, using the experiments in [6]. The research within, therefore, cannot be applied to bonded PT concrete. Once a model is developed and validated for bonded prestressing steel, the same method described herein could be used.

Another key research need is additional large-scale experiment validation of fire dynamics in large compartments. This would allow for an improved modelling of the thermal boundary and subsequent heat transfer. Those efforts are underway by multiple researchers, which will allow for further probabilistic investigations.

The research herein has shown that the quality of steel is very important. The stress relaxation of Steel II was shown to be less than Steel I (AS/NZ 4672 and BS 5896 respectively). The reason Steel II performs with less relaxation is because the stock considered in reference [36] had a sizably larger concentration of chromium than the stock of Steel I, which can improve creep properties [36]. The origin of the steel is inconsequential; it is the quality of the steel that is important and should carefully be checked should a performance-based fire design and analysis be performed on a UPT structure. There is no guarantee that this addition of chromium was intentional or if it is consistent. Hence, the authors recommend conservatism to prevent an extensive and expensive material study performed for every performance-based design.

6 CONCLUSIONS

This study has built upon the previous endeavours in PT concrete research. For the first time, the authors utilise iTFM on a UPT concrete structure to define critical design thermal boundaries and resulting unbonded tendon behaviour. A comparison of the effect of uniform and travelling fires for UPT structures is novel since it has not been reported in literature before. For the first time in peer reviewed literature, the Gales *et al* stress relaxation model [33] was compared and validated to real test data [6], which, once further validated with additional experimental data, could then be used in a novel fashion to assess a realistic building floor plan as opposed to a purely hypothetical structural configuration (where structure size, slab thicknesses, and span dimensioning may not be consistent with as built reality) to develop acceptance criterion as defined by the ASCE Manual of Practice [33].

Designers are in need of simple analytical tools which can enable rational fire design. When a family of fires is considered, it allows for the possibility to optimize the safety of a PT structure in order to facilitate the specification of a maximum tendon length that will not relax nor rupture. However, UPT concrete structures are of high complexity and require additional development of these analytical tools.

The results of the study establish the first steps towards the definition of the critical design thermal boundaries for UPT concrete flat-plate slabs. For this specific case study and the design fires examined, a slow travelling fire (5%) is the critical design fire that is required for tendon rupture. The result is expected as the fire leads to longer durations that the concrete will be exposed to heat. The analysis of this case study suggests that Steel II would fail due to tendon rupture between 342°C and 400°C. For this reason, a maximum tendon length of 42m can be considered when using steels that exhibit similar behavioural characteristics as Steel II in similar sized compartments, but should be used with caution when considering the limitations of this study. Steel I showed no vulnerabilities to strength failure (tendon rupture) but illustrated severe stress relaxation potential. In relation to stress relaxation, uniform fires are more critical for steels that exhibit the characteristic of both prestressing steels (I and II) compared to travelling fires. It was found that, by simulating a range of traditional and travelling fires, the tendon rupture temperature was higher than the critical temperature specified in the Eurocode when Steel I was utilised (minimum temperature of tendon rupture being 425°C to a critical temperature of 350°C), from which it can be assumed there is no risk of tendon rupture.

The development of the critical thermal boundaries allows for simpler and safer design for UPT concrete exposed to fire. The research herein has shown that the quality of prestressing steel is imperative to consider in the design, and that to ensure the most severe fire loads are considered, travelling fires must be included in the design fires. The localised heating effect and longer duration of slow travelling fires create more onerous conditions within the structure.

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Table 1 Research Needs from ASCE Manual of Practice [33]

Topic	Milestone
1. Concrete Cover Spalling	a. Determine acceptable limits for spalling
	b. Quantify the beneficial effects of specific mitigation techniques
	c. Develop appropriate acceptance criteria for concrete cover spalling
	d. Develop new spalling mitigation recommendations
2. Local and Global Behavior	a. Define acceptable limits for out-of-plumb concrete column behavior and slab deflection, particularly when related to realistic restraining mechanisms in actual buildings
	b. Develop appropriate local and global behavior acceptance criteria which considers the structural system performance of reinforced and prestressed concrete structures
3. Analytical Tools and Analysis	a. Develop acceptance criteria for reinforced and prestressed concrete structures under fire exposure that consider realistic fire scenarios

Table 2 Prestressing Steel Tendon Analysis Scenarios Considered

Fire Scenario	Tendon type	Tendon Starting Grid	Fire Starting Grid	Number of tendon lengths analysed [a]
Standard (ISO 834)	I and II	N	Full compartment	16
Parametric (Short)	I and II	N	Full compartment	16
Parametric (Long)	I and II	N	Full compartment	16
Travelling fire (2%)	I and II	N	N[b]	16
Travelling fire (5%)	I and II	N	N[b]	16
Travelling fire (20%)	I and II	N	N[b]	16
Travelling fire (40%)	I and II	N	N[b]	16

[a] Eight tendon lengths of increasing size were considered for each tendon type.

[b] Additional analyses were done starting on Grid M but resulted in no significant changes and are therefore not considered.

Figures

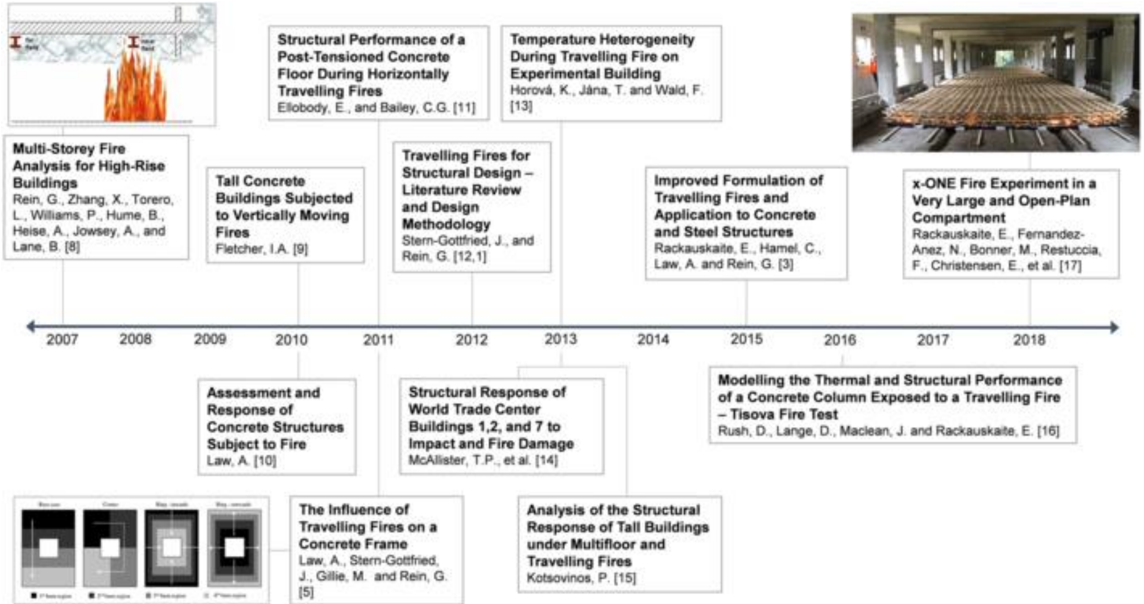

Fig. 1 Brief contextual contemporary history of the Travelling Fires Methodology in literature as related to this study (post 2007)

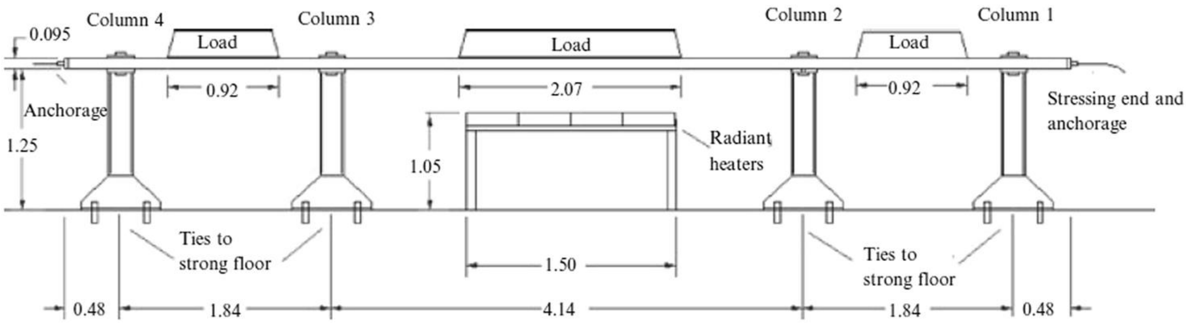

Fig. 2 Experimental three-span unbonded post-tensioned concrete setup, adapted from [6], exposed to localised heating under the middle span (dimensions in metres)

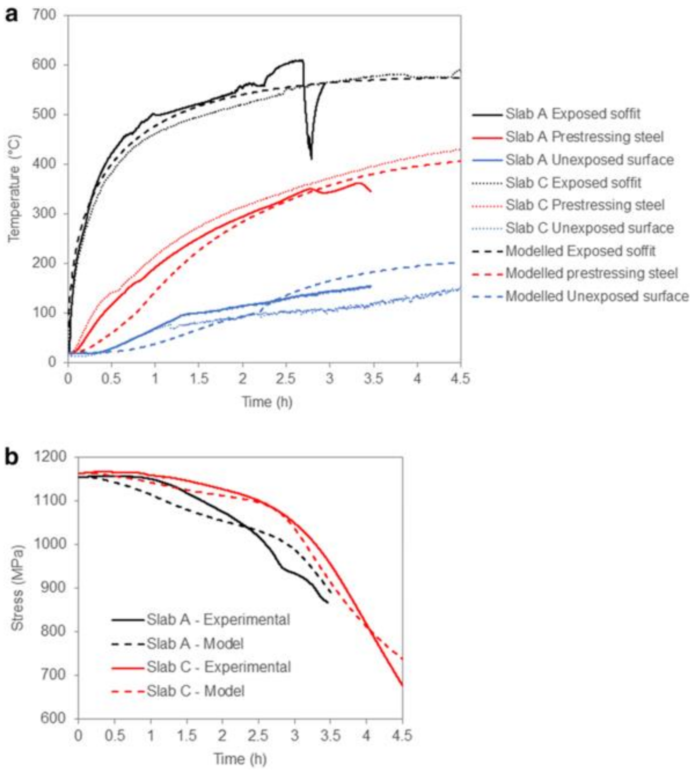

Fig. 3 (a) Thermal response and (b) prestressing tendon response of UPT multi-span experiments A and C (see [6]) showing good correlation between the experimental data and the model. The maximum difference between calculation and experimental results for the stress relaxation (b) is 47 MPa (5% error) for Slab A and 27 MPa (3% error) for Slab B for the first three hours of testing

Fig. 4 Exterior corner of the two-storey building used for the case study. This is a post-tensioned concrete structure designed by ARUP with over 100 prestressing tendons of various lengths

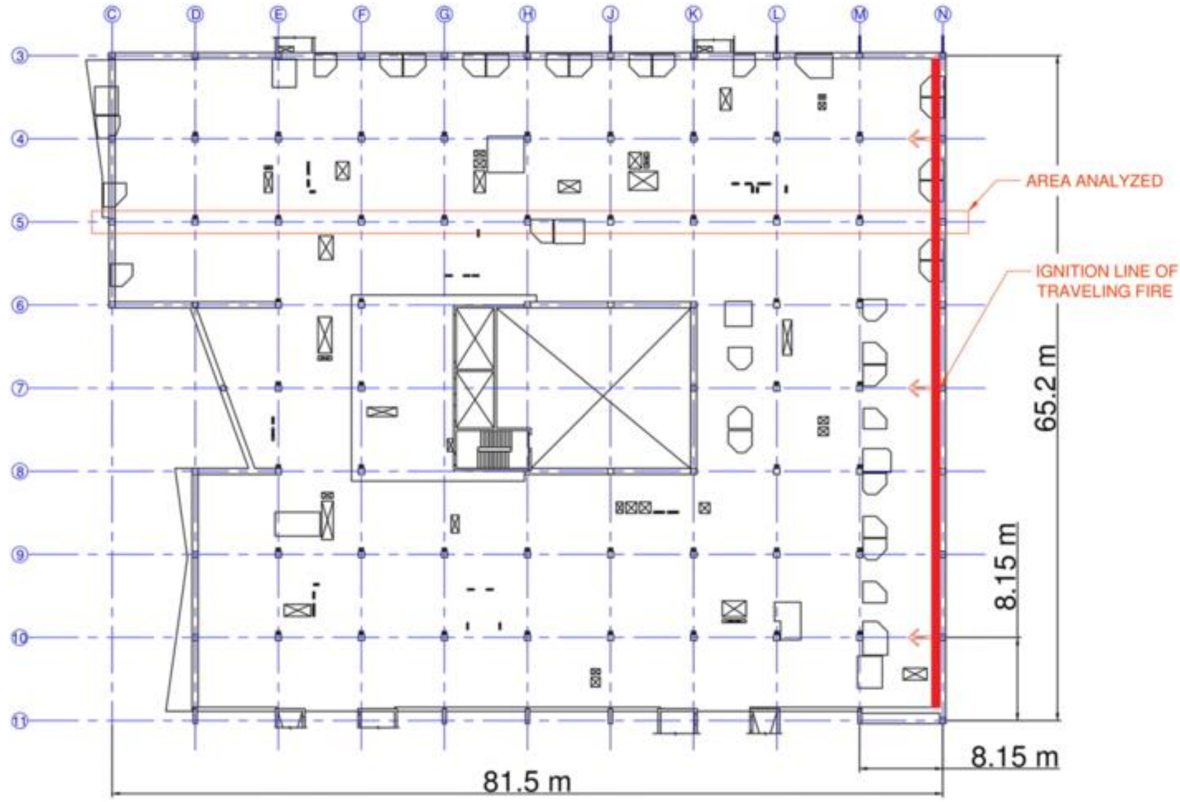

Fig. 5 Structural layout for the case study building, showing column grid spacing of 8.15m. The ignition line for the travelling fires analysed is shown on the right, as well as Grid line 5 along which the tendon length is varied

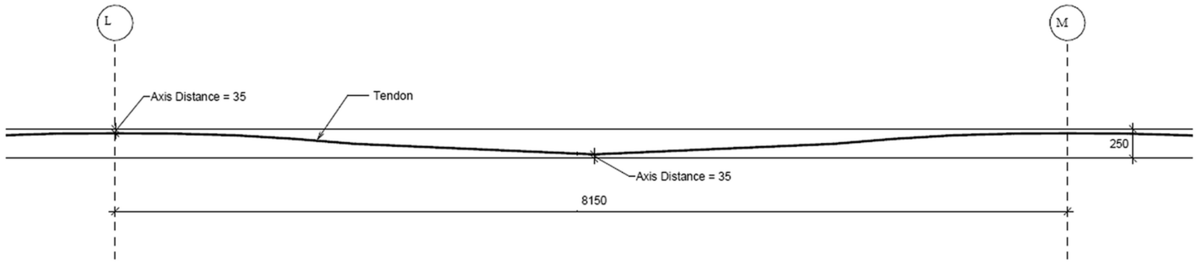

Fig. 6 Cross section of the slab between Grid M-L, along Grid line 5, showing the concrete cover and parabolic prestressing steel tendon (dimensions in mm)

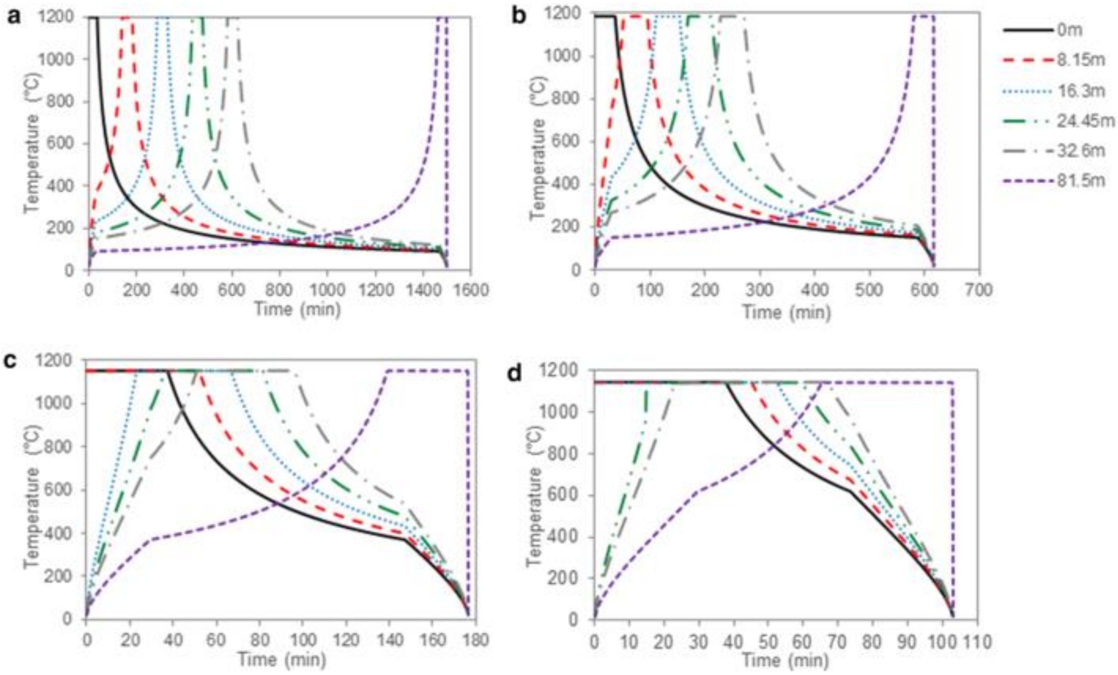

Fig. 7 Thermal exposure across gridline N-C for the four varying sizes of travelling fires examined: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. The graphs illustrate the thermal exposure at regular intervals in the compartment

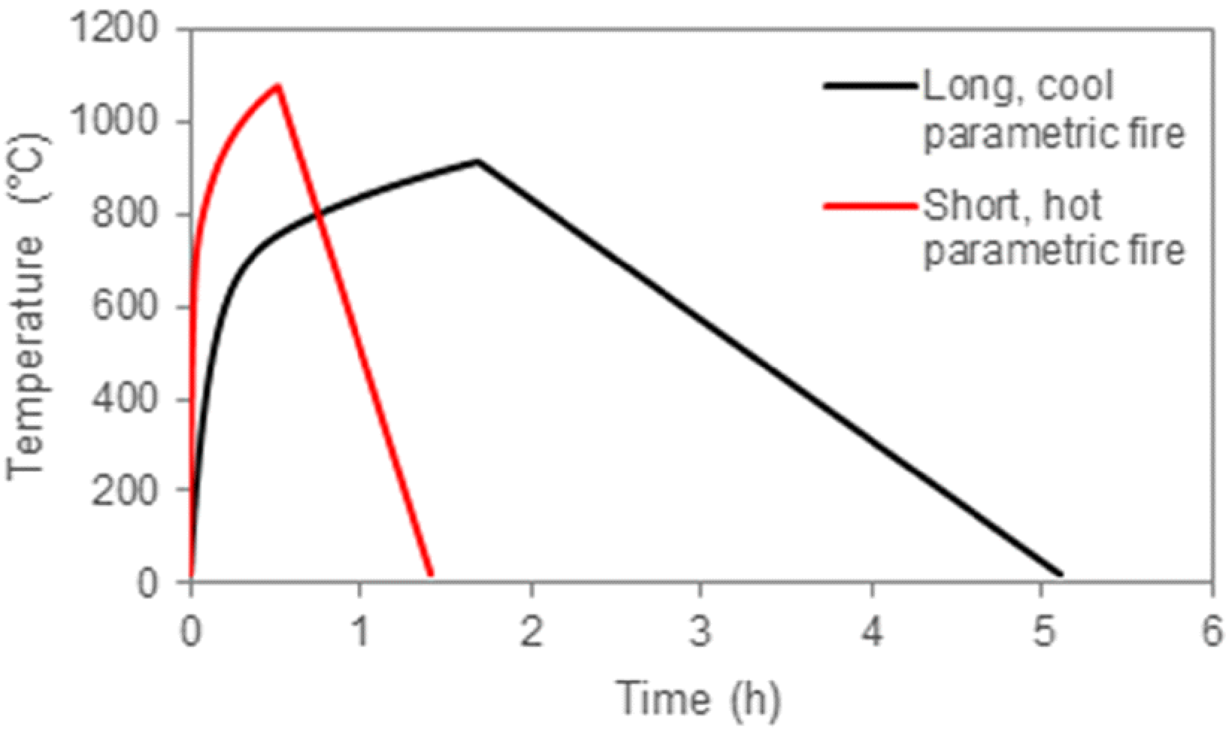

Fig. 8 Thermal exposures of the two parametric fires: a short, hot fire and a long, cool fire. These were found by varying the ventilation until the two extremes of parametric fires were found

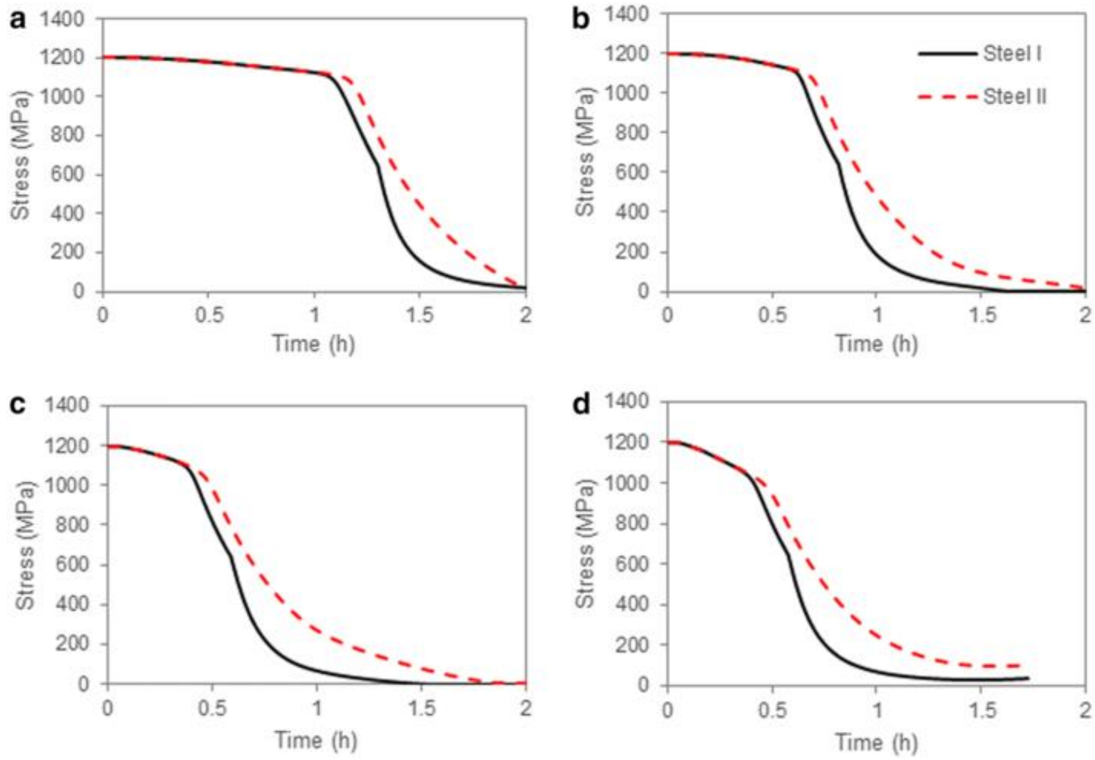

Fig. 9 Tendon stress relaxation model of a 32.6m tendon exposed to the four travelling fires: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. This figure illustrates that Steel II (fabricated according to AS/NZ 4672) experiences less stress relaxation over time than Steel I (fabricated according to BS 5896)

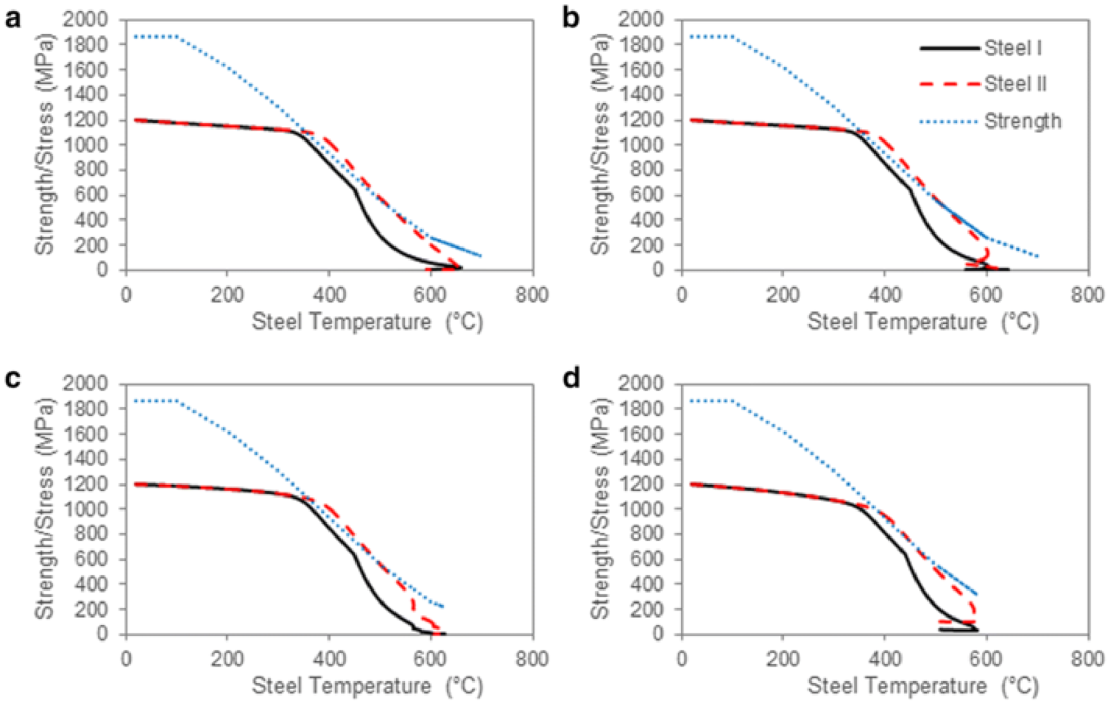

Fig. 10 Tendon strength reduction with temperature of a 32.6m tendon exposed to the four travelling fires: (a) 2%, (b) 5%, (c) 20%, and (d) 40%. This figure illustrates that Steel II (fabricated according to AS/NZ 4672) would fail due to tendon rupture (where the stress surpasses the strength of the tendon)

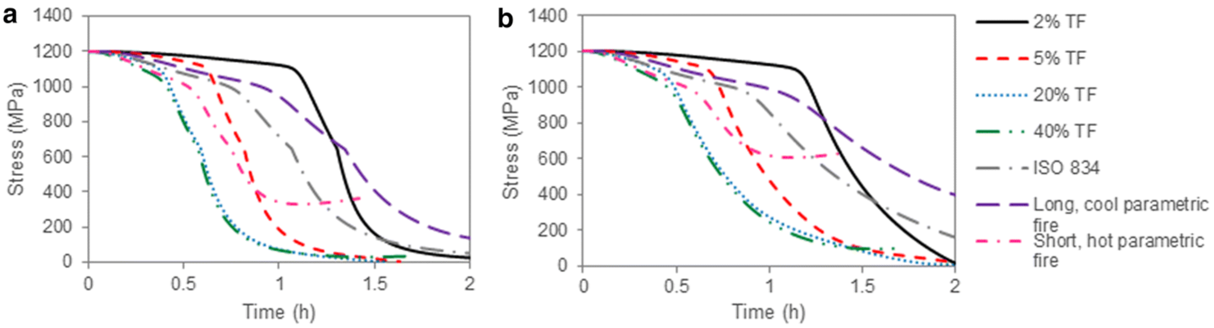

Fig. 11 Tendon stress relaxation model of a 32.6m tendon for (a) Steel I (fabricated according to BS 5896) and (b) Steel II (fabricated according to AS/NZ 4672) under the various design fires examined, illustrating that travelling fires cause greater stress relaxation. This figure also shows that for all cases except the short, hot parametric fire for Steel II, the tendon would fail due to stress relaxation (<600MPa)

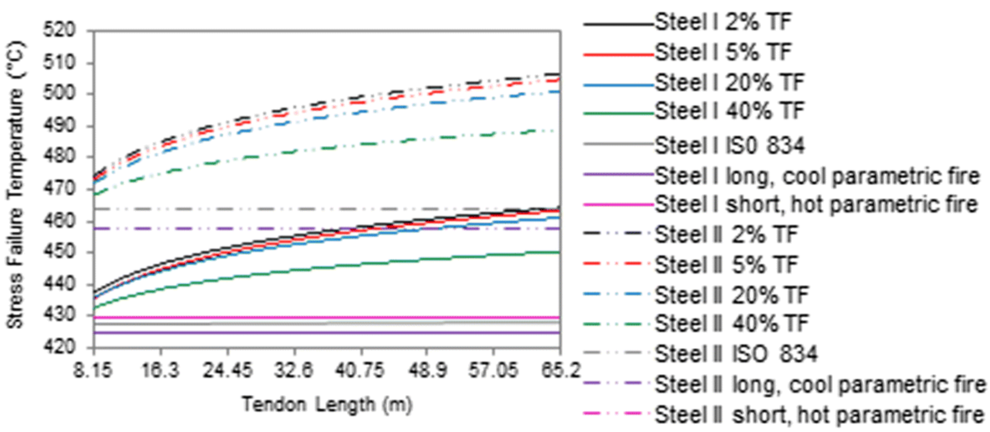

Fig. 12 Contour plots showing the temperature at which varying prestressing tendon lengths experience a stress failure (stress in tendon < 600MPa) for all design fires considered. The figure illustrates that the steels would fail at temperatures higher than the critical design temperature specified (350°C for the Eurocode and 427°C for the North American standard)

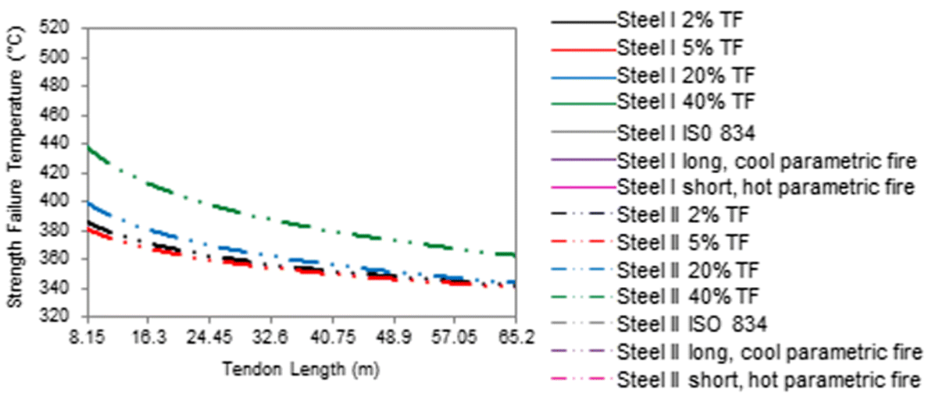

Fig. 13 Contour plots showing the temperature at which varying prestressing tendon lengths experience a strength failure (where the stress surpasses the strength of the tendon) for all design fires considered. The figure illustrates that Steel II would fail prior to reaching the Eurocode critical design temperature (350°C) for tendon lengths greater than 42m, and would fail to meet the North American standard (427°C). Steel I does not fail due to strength failure