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Unbonded Post Tensioned Concrete Slabs in Fire – Part II – Modelling Tendon Response and the Consequences of Localized Heating

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ABSTRACT
This is Part II of a two part paper dealing with the current state of knowledge of the fire-safe structural design and construction of unbonded post-tensioned (UPT) flat plate concrete structures. Part I provided detailed results of nineteen transient high temperature stress relaxation tests on restrained UPT tendons of realistic length and parabolic longitudinal profiles. Experimentation identified several credible concerns for UPT concrete structures in fire, most notably the potential for premature tendon rupture due to localized heating, which may result from a number of possible causes in a real structure. The real world response of continuous UPT tendons both during and after heating is largely unknown, and is dependent on factors which are not currently accounted for either in standard fire tests or by available prescriptive design guidance. This second part of the paper presents and applies a numerical model to predict the time-temperature-stress-strength interdependencies of stressed UPT tendons under localized transient heating, as may be experienced by tendons in a real concrete building in a real fire. The model is used, along with previously developed and validated computational models for heat transfer and prestress relaxation in UPT tendons, to assess existing prescriptive concrete cover requirements for UPT slabs. It is shown that localized heating of UPT tendons is likely to induce premature tendon rupture during fire, and that current prescriptive code procedures based on concrete cover alone are, in general, insufficient to prevent this. Based on the data presented it appears that minimum code prescribed concrete covers for UPT structures require revision if premature tendon rupture during fire is to be avoided.

Key words: Prestressing steel; Post-tensioned slabs; Unbonded construction; Fire endurance testing; Spalling; Concrete; Concrete cover; High temperature creep; Stress relaxation

1. INTRODUCTION

As discussed in Part I [1], real building response during and after fire for a continuous, multi-bay, unbonded post-tensioned (UPT) concrete structure is largely unknown; in particular the UPT tendons' behaviour at high temperature is not well understood. Part I presented 19 transient high temperature tests of restrained UPT tendons. Each tendon was exposed to localized heating of a varying duration and severity. Part I concluded that longer heated length ratios resulted in greater overall prestress relaxation, but also greater prestress recovery on cooling. Furthermore, tendons with shorter heated length ratios experienced less thermal expansion in proportion to their total unbonded length, which resulted in a higher likelihood of tendon rupture under localized heating. This suggested that critical temperatures (with respect to tendon rupture) for UPT tendons depend also on their heated length, and that smaller heated lengths have lower critical temperatures.

Fires in real buildings have shown that damage due to localized heating of a prestressing tendon affects the structural capacity of adjacent bays which are not directly exposed to fire [2]. There are several scenarios in which localized heating of the tendon may occur in a real structure. These include: single bay fires in a continuous multi-bay structure; travelling fires (which have been observed, for instance in Building 7 of the World Trade Centre complex [3]); ceiling jet fires (particularly within parking structures [4]); spalling of the concrete cover (which is more likely for modern concretes and pre-compressed elements); and transverse and longitudinal cracking during heating (which has been observed in many fire tests and real fires with UPT construction [5]). Whatever the cause of localized heating of a UPT tendon, it is clear that a more complete understanding of the potential consequences of localized heating on UPT tendon response is needed to ensure fire safe design and construction of UPT buildings.

No fire tests have ever been performed on realistic, continuous UPT flat plate concrete slabs. However badly needed these tests may be, full scale testing of model or actual UPT multi bay structures is not likely within the foreseeable future. Recent research has been targeted on furnace testing of isolated components with associated computational analysis [6,7]. Available computational procedures and tools, however, have not been sufficiently validated against real multi-bay UPT concrete structures in fire; their ability to model the transient behaviour of unbonded tendons, discrete cracking of concrete, and shear failure modes remains questionable. It is crucial that computational models prove their capability to simulate all relevant structural behaviours, including tendon prestress relaxation due to high temperature creep and strength degradation during heating and cooling. The current paper deals specifically with the fire performance of continuous UPT tendons in an effort to establish, experimentally and computationally, the potential consequences of localised heating. This is done with a view to developing the ability to defensibly model full UPT buildings in fire. Validation and use of a computational heat transfer and prestress relaxation model are presented to assess and predict the response of a UPT tendon in a concrete building exposed to fire.

2. OVERVIEW & OBJECTIVES

As discussed in Part I of this two part paper [1], the interdependencies of time (t), temperature (T), stress (σ), and strength (f_{pu}) for prestressing tendons at high temperature can lead to premature tendon rupture or to irrecoverable prestress loss (relaxation) during fire. A computational model for predicting high temperature deformation and tendon stress variation of cold-drawn prestressing steel has been developed and partially validated previously by the authors [8,9]. This model is further validated herein and then applied to assess tendon stress variation or premature rupture due to localized heating. The model predicts prestress variation by considering transient thermal creep, $\varepsilon_{cr} = f(t, T, \sigma)$, reversible thermal relaxation $\varepsilon_T = f(T)$, the reduced mechanical properties of steel $\varepsilon_\sigma = f(t,T,\sigma)$), and time to tendon rupture (based on the degradation of strength with temperature, $f_{ps} = f(T)$) for any assumed spatial or temporal tendon heating scenario.

The potential consequences of localized heating of UPT tendons are discussed with the aid of the computational model. The model is used to study two specific issues of interest:

1) *Prescriptive concrete cover requirements*: The fire resistance of UPT structures is typically ensured in design by prescribing minimum concrete covers to the prestressed reinforcement. The required cover is generally stated in tables which give the minimum concrete cover to the prestressed reinforcement (or minimum *axis distance*, which is the distance from the surface of the concrete to the centroid of the tendon) which is necessary to achieve a given fire resistance rating [10,11]. These cover requirements are nominally based on an assumed critical temperature for the prestressed reinforcement required to prevent collapse in a standard furnace tests of an isolated structural element – they are often defined as the temperature causing an approximate 50% ultimate strength reduction for the tendons [12]. Such requirements are based on tests on short, uniformly heated tendons and does not account for tendon

rupture due to localized heating. The computational model is used herein to compare fire resistance times and concrete covers prescribed by two widely used design codes (Eurocode 2 and North America's International Building Code [10,11]) to tendon rupture times predicted by the model during a number of credible fire scenarios.

2) *Potential impacts of cover spalling*: UPT flat-plate structures are particularly susceptible to explosive concrete cover spalling during fire [5]. This is due to a combination of high concrete compressive strength, slab pre-compression under service loads, and (typically) small amounts or (sometimes) a complete lack of bonded steel reinforcement (see [1]). Explosive cover spalling in real structures is essentially impossible to predict quantitatively, and despite considerable research efforts it has not yet been accurately modelled. While spalling models are advancing [e.g. 13], the authors believe that it will be some time before spalling can be accurately accounted for in design. Local concrete cover spalling during a fire would obviously cause localized heating of the prestressed reinforcement in the region near the spall. In the current paper, a preliminary modelling exercise is provided to highlight the possible impacts of cover spalling for premature tendon rupture during fire.

3. COMPUTATIONAL MODELLING

The computational model predicts stress relaxation at high temperature in locally heated UPT tendons. The model is capable of capturing the transient time-stress behaviour of an unbonded tendon exposed to any spatial and temporal heating regime, initial loading level, and time-temperature history. The model assumes the unbonded tendon to be discretized into finite thermal regions of constant uniform temperature within a given time step. This implies that, during a specific time interval, the tendon physically moves through the heated regions when it locally expands; the thermal regions remain spatially stationary with time.

The temperature of the tendon within each thermal region is input into the computational model via a one-dimensional finite difference heat transfer model coded previously by Bisby [14] for concrete slabs exposed to standard fires from below. Alternatively, the model can accept tendon temperatures measured from tests (as is necessary in validating the model against the experiments presented in Part I of the current paper).

The stress relaxation model is based on prior research on high temperature transient creep of steel by Dorn [15] and Harmathy [16]. The longitudinally discretized tendon is considered to be restrained, with fixed total length, such that any deformation along the tendon, whether due to mechanical, thermal, or creep deformation during heating sums to zero over the entire tendon length. Any increase in mechanical strains, creep and/or thermal deformation will be proportionally followed by a decrease in mechanical stress, which in turn will cause stress relaxation of the tendon (and vice versa in the event of thermal contraction during cooling). Figure 1 presents a simplified schematic of the calculation procedure – full details of the model's mechanics and procedures are available elsewhere [8,9,17].

3.1 Model Validation

Table 1 presents selected predictions from the computational modelling as compared against the results of the 19 restrained, unbonded, locally heated tendon stress relaxation tests presented in Part I [1]. In comparing the tests at different heated length ratios at a soak of 400ºC (Test 4 and Test 14), the computational model tends to show proportionally greater recoverable prestress for larger heated length ratios, where the shorter heated length ratios maintained a higher overall stress level but experienced greater irrecoverable losses. The predicted and experimental stress-time histories for selected cases are illustrated in Figure 2. The resulting predictions are generally consistent with the observed response, however the model tends to overestimate the prestress loss for both heated length ratios due to an over-prediction of creep during the constant temperature soak phase. To remedy this problem additional steady-state creep tests are needed to provide the necessary model inputs at stress levels above 600MPa (parameters are only currently available up to about 690MPa for cold-drawn prestressing steel [9,16]). This point is highlighted by examining tests 4 and 5, which had different initial prestress levels. Test 5 used an initial prestress of 600 MPa, in comparison to approximately 1000MPa used in Test 3. Modelling inaccuracy was a maximum of 3% for Test 5 and 18% for Test 4. Figure 3 illustrates the results of both tests in comparison with the predicted results. This figure confirms that creep is modelled more accurately at lower tendon stress levels using the available creep parameters and it may need refinement.

In a real building during a fire, deformations (thermal bowing) of the concrete in a UPT slab and the effects of gravity loads may actually cause stress in the tendon to increase rather than decrease [18]. Such potential effects have been conservatively neglected in the current analysis and in subsequent computational studies of tendon response in a flat plate UPT structure (see below). Importance is given primarily to modelling the strain and deformation behaviour of the tendon reacting to high temperature alone, and interactions with the deformation of the rest of the structure are currently ignored. Further research is necessary to account for other interactions in a continuous concrete slab and to eventually develop a defensible modelling approach that can account for all relevant actions.

3.2 Modelling the Behaviour of a Tendon in a Flat Plate Structure during Fire

To study the potential implications of localized heating of an isolated UPT tendon of realistic length and parabolic profile within a concrete structure during a standard fire, with particular emphasis on the likelihood of premature tendon rupture during heating in light of the conclusions of Part I of the current paper, illustrative calculations were performed with the computational model for a typical multiple bay UPT concrete slab. The example structure is a three bay flat plate continuous UPT concrete slab. The default slab dimensions and tendon profile used in the analyses are shown in Figure 4 and are based on a design example taken directly from the Portland Cement Association (PCA) [19]. The example structure represents a typical design which might be applied in North America for a multi-storey residential occupancy building.

The example slab is assumed to be cast from carbonate aggregate concrete. The slab thickness is 203mm and the tendon prestress is 1200MPa after losses for a No. 13 (99mm^2 nominal cross sectional area) low relaxation steel prestressing strand. The default minimum concrete clear cover at midspan of the central (restrained) bay is assumed as C_m = 19mm, and in the edge (unrestrained) spans it is taken as 38mm. This configuration gives a prescribed restrained fire resistance rating of 120mins according to the International Building Code (IBC) [10], but only 60mins according to the Eurocodes [11] (for reasons discussed below). The default total length of unbonded tendon is 25.6m.

Using the computational model developed by the authors [8,9] the tendon stress variation was predicted for exposure of the slab to an ASTM E119 [20] fire restricted to the central bay. This represents heating over about 36% of the total length of the structure, notwithstanding variable heating of the tendon within the central bay resulting from its parabolic profile within the slab (hence variable concrete cover). The moisture content of the concrete was conservatively assumed as 2% by mass for the purposes of the heat transfer analysis, and the tendon temperature was assumed to be uniformly the same as the surrounding concrete (i.e. on the basis of axis distance).

Tendon rupture was assumed to occur in the computational model if, at any instant during the analysis, the ‘current’ tendon stress level and temperature combination fell above the ultimate tendon strength limits given in Cl. 5.2(6) of EN 1992-1-2 [11]. The Eurocode 2’s tendon strength reduction curve is known to be conservative as far as tendon rupture is concerned, as discussed in Part I [1]. It is worth noting that tendon strength reduction factors have also been presented by Abrams and Cruz [12]; these are less conservative than those given by the Eurocodes and assume 50% loss of ultimate tensile strength at 426ºC (800ºF) – these curves are responsible for the 426ºC critical temperature assumed for prestressing steel in North American design codes and the resulting prescriptive concrete covers for a given fire resistance for an unrestrained UPT slab [21]. The Eurocode strength reduction factors are considerably more conservative, and the Eurocode explicitly states (in Cl. 5.2(5) of EN 1992-1-2 [11]) a critical temperature of 350ºC for prestressing steel. This is apparently based on a 45% reduction of the characteristic 0.1% proof-stress due to temperature. While allowance is made in EN 1992-1-2 [11] to increase the critical temperature of prestressing steel based on its load ratio, Cl. 5.2(9) of EN 1992-1-2 [11] states a critical temperature for unbonded tendons of 350ºC, and warns that higher temperatures should not be used unless accurate methods of determining the effects of slab deflection, and more specifically prevention of tendon rupture (Cl. 4.1-3), are accounted for in design. This suggests that a performance-based design for a UPT slab should be taken by modelling the behaviour of prestressing steel in fire and accounting for the effects on the entire structure’s response, although no obvious method of accurately doing this is given.

To illustrate typical output from the computational model for the example structure, Figure 5 shows the predicted variation in tendon stress levels for the default analysis with the central bay exposed to an ASTM E119 fire [20] from below. Also included in this figure are the tendon strength reduction curves suggested by Abrams and Cruz [12] and EN 1992-1-2 [11], as well as the predicted temporal variation in temperature of the tendon at its smallest axis distance (i.e. midspan of the central bay). This figure clearly demonstrates the complex interplay between stress, creep, and strength at elevated temperature, and also shows the importance of what otherwise appear to be subtle differences in strength reduction equations suggested in different sources [11,12]. As already noted with reference to the test data given in Part 1 and tabulated in Table 1, prestress relaxation is predicted to accelerate rapidly at temperatures above about 300°C. For the default analysis case the tendon is predicted not to rupture according to the Abrams and Cruz [12] strength reduction curve up to maximum temperatures exceeding 500°C (beyond two hours), whereas the EN 1992-1-2 [11] curve conservatively predicts tendon rupture at 323°C, after only 42 minutes of exposure to fire. The significance of these differences is highlighted in the following sections.

4. POTENTIAL CONSEQUENCES OF LOCALIZED HEATING

Cold drawn prestressing steel is more sensitive than mild steel to deterioration of mechanical properties at elevated temperature, and it therefore requires larger concrete cover for fire protection [11,21]. The Eurocodes [11] recognize this and explicitly require that an additional 15mm of concrete cover axis distance be provided for prestressed steel reinforcement in comparison with mild steel reinforcement, both for continuous (including flat plate) and for simply supported slabs [22].

In general, concrete cover requirements for achieving a given fire resistance are given in international design codes for *restrained* and *unrestrained* flexural members (sometimes distinguished as *continuous* and *simply-supported*). However, none of the available codes explicitly distinguishes between *bonded* and *unbonded* construction. EN 1992-1-2 [11] appears to be aware of some of the hazards specific to UPT construction but provides no direct design guidance to account for this. For instance, EN 1992-1-2 [11] (Cl. 4.1(3)) states that "sudden failure caused by excessive steel elongation from heating for prestressed members with unbonded tendons should be avoided," but it provides no guidance on how this can be achieved in practice. EN 1992-1-2 [11] also cautions (Cl. 5.2(9)) that "for unbonded tendons critical temperatures greater than 350°C should only be used where more accurate methods are used to determine the effects of deflections," although again no practical guidance is given and the reasons for focusing on deflection rather than tendon rupture are not clear. In the absence of specific design guidance it is reasonable to assume that designers will simply adhere to the minimum prescriptive (tabulated) concrete cover requirements given for prestressed flexural elements; it is shown below that these may not be sufficient to prevent tendon rupture.

4.1 Prescriptive Concrete Cover Requirements for UPT Construction

To investigate the implications of currently prescribed concrete cover requirements for UPT slabs, the typical example slab shown in Figure 4 was used to perform a number of computational simulations assuming different axis distances at midspan (C_m) for the UPT tendon based on covers required to achieve prescribed fire resistance ratings (from 30mins to 240mins). Both EN 1992 1-2 [11] and IBC [10] requirements were considered. The IBC clear cover requirements have been adjusted to axis distance to allow a fair comparison. This has been achieved by adding 3mm for the tendons' sheathing and half the bar diameter (6.5mm) [23].

The tendon temperatures used in the model runs (including the default analysis in Section 3) were assumed as the temperature of concrete at the level of the tendon's centroid, rather than at the bottom of the tendon or at the centroid of the single wire closest to the heated face. This approach is consistent with the *axis distance* definition used by Eurocode 2 [11].

4.1.1 Eurocode 2 Concrete Cover Requirements

The axis distance concrete cover requirements in Eurocode 2 are based on validated computational heat transfer calculations that nominally assume a critical temperature of 350ºC for the tendon. The heat transfer analysis used in the authors' stress relaxation model was used to verify that the Eurocode axis distances are representative of the 350ºC isotherm for the respective fire ratings quoted in the code for simply supported members. For continuous, two-way, or flat plate UPT slabs, prescribed axis distances are reduced in the Eurocode to account for fire resistance enhancements arising from other actions (i.e. restraint, continuity, membrane actions, etc.) although the magnitudes of these axis distance reductions are not rationalized in any obvious way. It appears that they are based on observed load bearing capacities of isolated elements in standard furnace tests on reinforced concrete slabs [24], and subsequently increased by 15mm to account for the lower assumed critical temperature of prestressing steel as compared with reinforcing steel [22]. It has also been stated in the literature [25] that the concrete covers suggested in the Eurocodes have been influenced by a series of tests on continuous one way UPT concrete slabs performed by Van Herberghen and Van Damme [26] (discussed in Part I of this paper).

4.1.2 IBC Concrete Cover Requirements

The IBC concrete cover requirements are based on an assumed critical temperature of 426ºC rather than 350ºC. In general, the IBC therefore requires considerably less cover than Eurocode 2, even when the clear covers are adjusted to axis distance (refer to Table 2). The IBC covers are apparently based on measured reinforcement temperatures observed in standard furnace tests on isolated structural elements [5,27]. The required covers in the IBC appear also to have been influenced by a 1973 report by Gustaferro

[27] in which measured tendon temperatures in standard furnace tests were used to argue for reduced required cover requirements. Gustaferro used data collected from six standard fire tests on post-tensioned concrete slabs to compare the temperatures of tendons at a given cover depth to the temperature of the concrete at the same depth, and found that the tendons were cooler than the surrounding concrete by amounts corresponding to between 3/8” (10mm) and 9/8” (29mm). On this basis, Gustaferro successfully argued for a reduction of “at least 3/8” (10mm)” in the required concrete cover for post-tensioned slabs. This concept was subsequently adopted in some North American standards [28], resulting in concrete cover requirements which seem likely fail to ensure that tendon temperatures will be maintained below 427ºC even for the simply supported (unrestrained) case. It should be noted, however, that in making the comparisons between measured tendon temperatures and temperatures in the concrete at the same level, Gustaferro [27] used the *average* recorded tendon temperature, even for parabolic tendons with variable concrete cover. This approach is hard to defend, particularly in light of the discussion in the following section.

The IBC makes no distinction between one way and two way slabs, nor does it refer to continuous slabs. Rather, it uses the terminology *restrained* to refer to these situations. Required concrete covers are further reduced for restrained cases, but again it is difficult to rationalize these reductions since they appear to be based on observations from standard furnace tests of isolated structural elements. For unrestrained slabs the IBC covers have remained essentially unchanged since 1959 [29]. For restrained slabs the covers have not changed since at least 1973 [27], despite the tests by Van Herberghen and Van Damme [26] which provided compelling evidence that cover requirements should be increased for UPT slabs.

Finally, it is noteworthy that mild steel reinforcing bars are a single, homogeneous structural component, whereas prestressing steel is fabricated from a helix of seven individual wires plus grease and sheathing. A conservative assumption in defining concrete cover for fire protection of UPT tendons would be to assume the tendon temperature based on the clear cover alone (i.e. the tendon temperature is equal to the temperature at the bottom of the tendon); this is essentially the approach used by the IBC [21], albeit with considerably smaller required covers than the Eurocodes. In the case of an axis distance definition for cover it could reasonably be argued that, for prestressing tendons consisting of multiple wires, the temperature should be taken as that of the most heated wire rather than the tendon’s centroidal temperature. Thus, the axis distance of the most heated wire would be used, resulting in a 4mm increase in the required cover for a 13mm nominal diameter tendon. Nevertheless, the correct, conservative definition of cover depth for prestressing tendons warrants further study, particularly in light of the importance of cover for preventing premature tendon rupture during fire, as discussed in the following section.

4.2 Concrete Cover and Tendon Rupture

To study the effectiveness of current prescribed concrete covers for preventing premature tendon rupture during a localized fire, code-prescribed concrete cover requirements for both restrained and unrestrained cases were assessed using the computational model, applied to the example structure with the central bay subjected to a standard fire (i.e. the time to tendon rupture was predicted by the model for an assumed midspan tendon axis distance (C_m) in the central bay). Tendon strength reductions were assumed according to Eurocode 2 [11].

Figure 6 plots the predicted time to tendon rupture versus midspan axis distance for three heated length ratios. Smaller heated length ratios were simulated by ‘adding’ additional bays to both sides of the default structure while still considering only the central bay exposed to fire (i.e. by effectively elongating the total length of unbonded tendon while maintaining the same fire exposure). The heated length ratios were 36% (9.1m heated out of a total tendon length of 25.6mm for the default case in Figure 5), 21% (a total tendon length equal to 43.7m), and 13% (a total tendon length equal to 71.1m). The curves are truncated in cases where the concrete cover was sufficiently large that no tendon rupture was predicted (i.e. for 21% and 36% at times greater than 120mins).

Also shown in Figure 6 are the 350ºC and 426ºC isotherms predicted by the heat transfer model at the tendons’ axis distance; these are significant because they nominally represent the tendon critical temperatures assumed by Eurocode 2 [11] and the IBC [10], respectively, for simply supported UPT flexural elements.

Comparison of the predicted tendon rupture times versus these isotherms indicates that the IBC’s critical temperature of 426ºC is difficult to justify and requires revision since premature tendon rupture will occur well before a temperature of 426ºC is reached in real UPT structures. Of course, this assumes that it is essential to avoid tendon rupture to achieve adequate fire resistance, which might not be the case provided that sufficient bonded steel reinforcement is present or if moment redistribution or membrane actions can be relied upon to prevent collapse. However, given the unknowns associated with these secondary behaviours, it would seem prudent to design UPT slabs for fire with prevention of tendon rupture taken as an explicit design objective.

The Eurocode's critical temperature of 350ºC is far more defensible, although for shorter heated length ratios even this may not be sufficiently conservative to prevent tendon rupture due to localized heating before the code-prescribed fire resistance is achieved. For example, for 120 minutes fire resistance and a 13% heated length ratio, about 54mm of axis distance is needed to keep the tendon below 350ºC, whereas about 61mm is needed to prevent tendon rupture. While the prescriptive requirements of Eurocode closely reflect the 350ºC isotherm, they do not account for the stress-strength-temperature-interaction that develops under smaller heated lengths as are likely to be experienced in a modern UPT structure.

The IBC prescriptive cover requirements [10] for simply supported UPT slabs are slightly conservative with respect to the 426ºC isotherm (compare the data in Table 2 against the 426ºC isotherm in Figure 6). However, a limiting temperature of 426ºC is inadequate to prevent tendon rupture within the prescribed fire resistance time. For example, at 120mins an additional 12mm of concrete cover is required simply to equal the 350ºC isotherm equivalent of the Eurocode [11] requirements, and an additional 18mm is required to prevent tendon rupture for smaller heated length ratios.

Table 2 provides a tabular summary of the analysis results and code requirements, comparing prescribed concrete cover axis distances against covers required to prevent tendon rupture for the example structure at 36% and 13% heated length ratios. This table confirms that the Eurocode [11] covers for the simply supported case essentially represent the 350ºC isotherm, but that they are unconservative with respect to tendon rupture for shorter heated lengths. The table also confirms that the IBC [10] covers are considerably less than required to prevent tendon rupture on the basis of the model predictions.

In both codes the required covers for restrained slabs are somewhat reduced, since collapse prevention is assumed to be aided by restraint and continuity and is not wholly dependant on the tendon temperature. While this approach may be defensible on the basis of available standard furnace test data, it means that tendon rupture, well before the prescribed fire resistance time is met, is virtually guaranteed to occur in any real, multiple span continuous or flat plate UPT structure during a standard fire.

It is important to reiterate that the tendon rupture predictions in Figure 6 and Table 2 were made using the conservative ultimate strength reductions suggested in Eurocode 2 [11]. Using the less conservative strength reductions suggested by Abrams and Cruz [12] would mean that no tendon ruptures are predicted for any of the above cases. It should also be noted, however, that the model has previously been shown [9] to predict stress relaxation in a conservative matter (i.e. it somewhat overestimates the amount of creep/relaxation), such that the tendon stress is likely to be higher than predicted by the model, making tendon rupture more likely in a real structure. Furthermore, thermal bowing, vertical deflections due to gravity loads, and lateral thermal expansion during heating will all act to increase tendon stress during the early stages of a fire [18]; also making tendon rupture more likely. It is therefore plausible that the tendon stress reductions predicted by the current model could be less than in a real structure, and that tendon rupture may occur even earlier than predicted.

Additional experimentation and modelling are underway to investigate the above issues. Nonetheless, the current analysis provides compelling evidence that the IBC's [10] prescriptive concrete cover requirements are unconservative for preventing tendon rupture during fire. It is suggested that the IBC [10] adopt the prescribed concrete covers given by EN 1992-1-2 [11].

In light of the above discussion it is interesting to consider what effect tendon rupture and full loss of prestress might have on the structural capacity of a real flat plate UPT slab. Total loss of prestress in a continuous two-way UPT flat slab at ambient temperature has previously been investigated experimentally at the University of Texas [30]. In 1975, a nine bay (3 span × 3 span) scaled two-way UPT flat slab structure was damaged by manually de-stressing all of the tendons passing through the central bay. The slab remained stable under its self weight. It was then loaded with an imposed design live load, and again remained stable. However, the imposed live load was applied only over the *exterior* spans; the central bay was subjected to self-weight only. While this test shows that a two-way continuous UPT flat slab can withstand total loss of prestress without collapsing at ambient temperature under self weight only, in a fire scenario it is typical to assume loads of at least self weight plus 50% of the live load. Furthermore, the test at ambient temperature would not include damage due to thermal bowing, thermal stress cracking, and material property degradation (both steel and concrete) at elevated temperature. Additional research to study these issues is warranted.

4.3 Effects of Cover Spalling on Localized Heating and Tendon Rupture

All of the prescriptive code requirements discussed in the previous section assume that no cover spalling occurs. Previous analysis by the authors [5] provided clear evidence that the assumption of zero cover spalling is hard to defend, either on the basis of standard fire tests performed on UPT concrete structural elements or based on evidence from real fires in UPT buildings. It was therefore desired to use the prestress relaxation and tendon rupture model to investigate the potential consequences of different amounts of cover spalling on the likelihood of tendon rupture during fire.

To simulate the effect of spalling on the example slab, the computational heat transfer algorithm coded by Bisby [14] was modified to simulate any desired location, time, length and depth of concrete cover spalling. The analysis assumes one-dimensional heat transfer analysis based on a finite difference algorithm essentially identical to that proposed by Lie [31]. Spalling is manually imposed in the model by killing elements in the model to the desired depth. Once the spalled elements are killed in the analysis the heat transfer equations assumed at the new exposed surface are changed to reflect a radiative/convective boundary defined by gas temperature in accordance with the Standard Fire rather than a conductive boundary. The time, location, length, and depth of spalling are thus prescribed by the user. It must be noted that this is in no way an attempt by the authors to model when and how cover spalling will occur; we believe that this would be premature given current understanding of the factors influencing spalling despite published attempts by others [32] to do this. The current analysis is merely intended to illustrate the possible effects of spalling on tendon rupture for UPT slabs.

The default spalling analysis considered spalling to occur between minutes 10 and 11 of a standard fire, to a depth of 10mm over a length of 1000mm centred on midspan in the central bay of the example structure, shown in Figure 4. This is the time when the concrete at a depth of 10mm reaches 350 to 375ºC according to the heat transfer analysis; this temperature is roughly the value used by Hertz [33] for the critical point of steam beyond which pressures increase, as does the risk of spalling. Spalling at midspan is the most damaging location in terms of tendon rupture since the concrete cover is smallest at this location. Again, the analysis assumes that only the central bay is exposed to fire. The initial concrete cover axis distance (C_m) for the central bay was taken as 28mm (19mm clear cover). The depth (10mm to 28mm), time (5mins to 20mins), length (100mm to 9140mm), and location (centred on midspan or adjacent to the left hand column) of assumed spalling were varied and the time to tendon rupture predicted.

The results are summarized in Table 3. For spalling at midspan, using the EN 1992-1-2 [11] tendon strength reduction factors tendon rupture is predicted within 14 minutes of the imposed spalling event. If, however, the Abrams and Cruz [12] tendon strength reductions are used tendon rupture is predicted not to occur in several cases (because the prestress is predicted to relax faster than the tendon's strength reduces). In the case that spalling removes the entire clear cover, tendon rupture occurs essentially instantaneously using either the Abrams and Cruz [12] or Eurocode [11] tendon strength reduction factors. Spalling adjacent to the left hand column was found not to cause tendon rupture providing the spalling length was less than 2m and the depth was less than 20mm. Clearly, this is because larger covers are provided near supports. This analysis suggests that any amount of cover spalling in sagging moment regions of UPT structures is likely to lead rapidly to tensile rupture of any UPT tendons in the region of the spall; this is consistent with observations from furnace tests and real fires [5].

ACI/ASCE Committee 423 [34] recommends a fully developed system of non-prestressed bonded reinforcing in all one way slabs and beams such that total loss of prestress will not cause collapse under full dead load plus 25% of the specified live load. This criterion is not applied to two way slabs since they are thought to possess increased redundancy [21], mainly as demonstrated through the University of Texas tests mentioned previously [30]. However, the Texas test is not representative of the loading or structural conditions that would be found in a building during a fire. Van Damme and Van Herberghen [26] have suggested that bonded reinforcement should be provided over the entire soffit of UPT slabs for increased fire resistance. A steel reinforcing percentage of 0.2% in both directions was deemed as sufficient, and the research presented in this paper supports this assertion. Additional research is needed to more accurately define the minimum bonded reinforcement levels that should be provided in all UPT slabs.

5. CONCLUSIONS AND RECOMENDATIONS

The modelling presented in this paper has clearly illustrated the time-temperature-stress-strength interdependencies affecting UPT tendons subjected to transient localized heating, which was also shown experimentally in Part I of this paper. Localized heating of UPT tendons in real flat plate concrete slabs is likely to induce tendon rupture during a fire, as shown by the modelling exercises presented herein. Current prescriptive code procedures based on concrete cover (or axis distance) alone are insufficient to prevent this.

As far as the prescriptive axis distance requirements stated by Eurocode 2 are concerned, the assumed critical temperature of 350ºC appears to be defensible for simply supported slabs, although for shorter heated length ratios it appears to be insufficient to prevent tendon rupture before the fire resistance period is achieved. Increasing the Eurocode's prescribed covers by about 5mm would adequately address the issue of tendon rupture under localized heating for the simply supported case.

For the IBC, the assumed critical temperature of 426°C seems hard to justify and should be considered for revision. In all cases the IBC [10] covers are considerably less than required to prevent tendon rupture before the prescribed fire resistance time is achieved. For example, for 120mins fire resistance an additional 18mm of concrete cover is required to prevent tendon rupture for smaller heated length ratios for the simply supported case.

In both Eurocode 2 and the IBC the required concrete covers (or axis distances) for restrained slabs are further reduced even from the unrestrained case, since collapse prevention during fire is assumed to be aided by restraint and continuity (redistribution of moments). While this approach may be defensible on the basis of available standard furnace test data, it virtually guarantees that tendon rupture will occur in restrained slabs well before the prescribed fire resistance time is met. Additional research is needed to determine whether this unreasonably increases the risk of structural collapse of UPT structures during fire.

Based on the demonstrated risk of premature tendon rupture described in the current paper, particularly for restrained/continuous UPT concrete slabs, the following recommendations are made:

1. *Minimum bonded reinforcement*: As suggested previously by others [26,34,35], but not yet adopted in most design codes for UPT slabs, it is recommended that a fully developed system of non-prestressed bonded reinforcement be provided such that total loss of prestress will not cause collapse under full dead load plus a credible proportion (depending on local building code requirements) of the specified live load. Bonded reinforcement should be provided over the entire soffit of UPT slabs. Van Herberghen and Van Damme [26] have previously proposed a steel reinforcing percentage of 0.2% in both directions. Additional research is needed to more accurately define the minimum bonded reinforcement levels that should be provided in all UPT slabs, particularly for multiple span continuous flat plates.
2. *Spalling mitigation*: Measures should be taken to mitigate spalling in UPT slabs, since spalling, particularly in sagging moment regions, virtually guarantees premature tendon rupture. The most practical means by which to mitigate spalling is to use at least 2kg/m^3 of polypropylene microfibres within the concrete mix [11], although for very high strength concretes this fibre dosage may need to be further increased.
3. *Definition of concrete cover*: Research should be performed to better understand how to properly define the temperature of UPT tendons during fire. The use of axis distance, versus clear concrete cover, versus the axis distance to the most heated wire needs to be confirmed.

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Table 1: Overview of computational modelling results for transient, localized, high temperature relaxation tests performed by Gales [9] and MacLean [36]

		Test setup				Prestress levels			
						Experiment		Model	
Test #[a]	Heated length ratio (%)	Initial prestress [b] (MPa)	Target soak temp[c] (°C)	Soak time (min)	Ramp rate (°C/min)	End of soak (MPa)	Residual stress (MPa)	End of soak (MPa)	Residual stress (MPa)
Gales' Tests									
1	3	974	200	90	10	961	972	959	972
2	3	971	300	90	10	941	953	935	948
3	3	973	400	90	10	808	831	669	688
4	3	1009	400	90	10	807	831	664	686
5	3	599	400	90	10	549	569	533	556
6	3	997	400	5	10	882	897	808	829
7	3	1015	400	45	10	815	824	702	713
8	3	1007	400	90	2	805	814	645	668
9	3	1015	400	90	30	769	786	631	657
10	3	997	500	2	10	--[d]	--[d]	--[d]	--[d]
11	3	983	700	--	10	--[d]	--[d]	--[d]	--[d]
MacLean's Tests									
12	11	1002	200	90	10	947	993	939	993
13	11	1006	300	90	10	896	972	882	975
14	11	1001	400	90	10	648	762	576	680
15	11	1014	400	90	10	663	775	567	670
16	11	1022	400	45	10	697	812	622	754
17	11	1036	400	5	10	771	875	727	855
18	11	1003	500	90	10	245	388	175	313
19	11	975	700	90	10	3	140	32	136

[a] For Tests 12-19, details can be found in MacLean et al. [36]
[b] The target initial prestress was 1000MPa with exception of Test 5 in which it was 600MPa
[c] The maximum overshoot for tests 1-11 was 1.5 ± 2.0°C and tests 12-19 was 5.6 ± 3.6°C
[d] Tendon rupture

Table 2: Summary of analysis results for prediction of tendon rupture times with varying prescribed axis distances (concrete covers) assuming carbonate aggregate concrete

Fire resistance rating	Depth of 350ºC contour (mm)[a]	Required fire resistance axis distance (mm)						
		For stress > strength at 36% heated length[b]	For stress > strength at 13% heated length[b]	Prescribed by EN 1992-1-2 [11]			Prescribed by the IBC [10]	
				For simply supported slabs	For continuous slabs	For flat plate slabs[c]	For simply supported slabs[d]	For continuous slabs[d]
30 minutes	20	21	22	25	25	25	--[f]	--[f]
60 minutes	34	36	39	35	25	30	--[f]	--[f]
90 minutes	45	46	51	45	30	40	--[f]	28
120 minutes	54	55	61	55	35	50	47	28
180 minutes	69	--[e]	75	70	45	60	59	34
240 minutes	81	--[e]	87	80	55	65	--[f]	41

[a] Based on HTA using [18] which compares well with those performed by EN 1992-1-2 [11] and in experimentation [11,37]
[b] Based computational model stress reduction compared to Eurocode strength reduction taken from Section 5.2 [11]. This time may be higher or lower depending on possible modelling improvements suggested within the paper
[c] 15mm additional axis distance was added [22] to tabulated data from [11]
[d] Clear cover adjusted to axis distance, by adding 3mm sheathing and ½ bar diameter of 12.7mm
[e] In this simulation an axis distance greater than 58mm does not predict failure by tendon rupture
[f] The IBC [10] does not tabulate values for these fire resistances

Table 3: Predicted time to tendon rupture for different assumed spalling configurations

Simulation #	1	2	3	4	5	6	7	7	8	9	10	11
Time of spalling (min)	n/a	10			5	20	10					
Spalling depth (mm)	0	10	20	28	10					20		
Spalling length (mm)	0	1000					9140	100	1000	2000	3000	4000
Location	Midspan								Adjacent to support			
EN 1992-1-2 strength	42	22	10	10	19	28	23	21	42	42	18	10
Abrams and Cruz	--	--	11	10	--	--	--	--	--	--	36	11
Time to reach 350ºC	47	26	10	10	24	32	26	26	47	47	21	10
Time to reach 426ºC	67	37	11	11	34	42	37	37	67	67	28	11

* Note: Default configuration (Simulation 2) of sample slab assumes spalling centred at midspan over 1000mm to a depth of 10mm, with an initial concrete axis distance of 28mm. Tendon strength reduction is based on Eurocode Section 5.2 [11]. Note that "--" denotes no tendon rupture calculated by direct stress exceeding strength

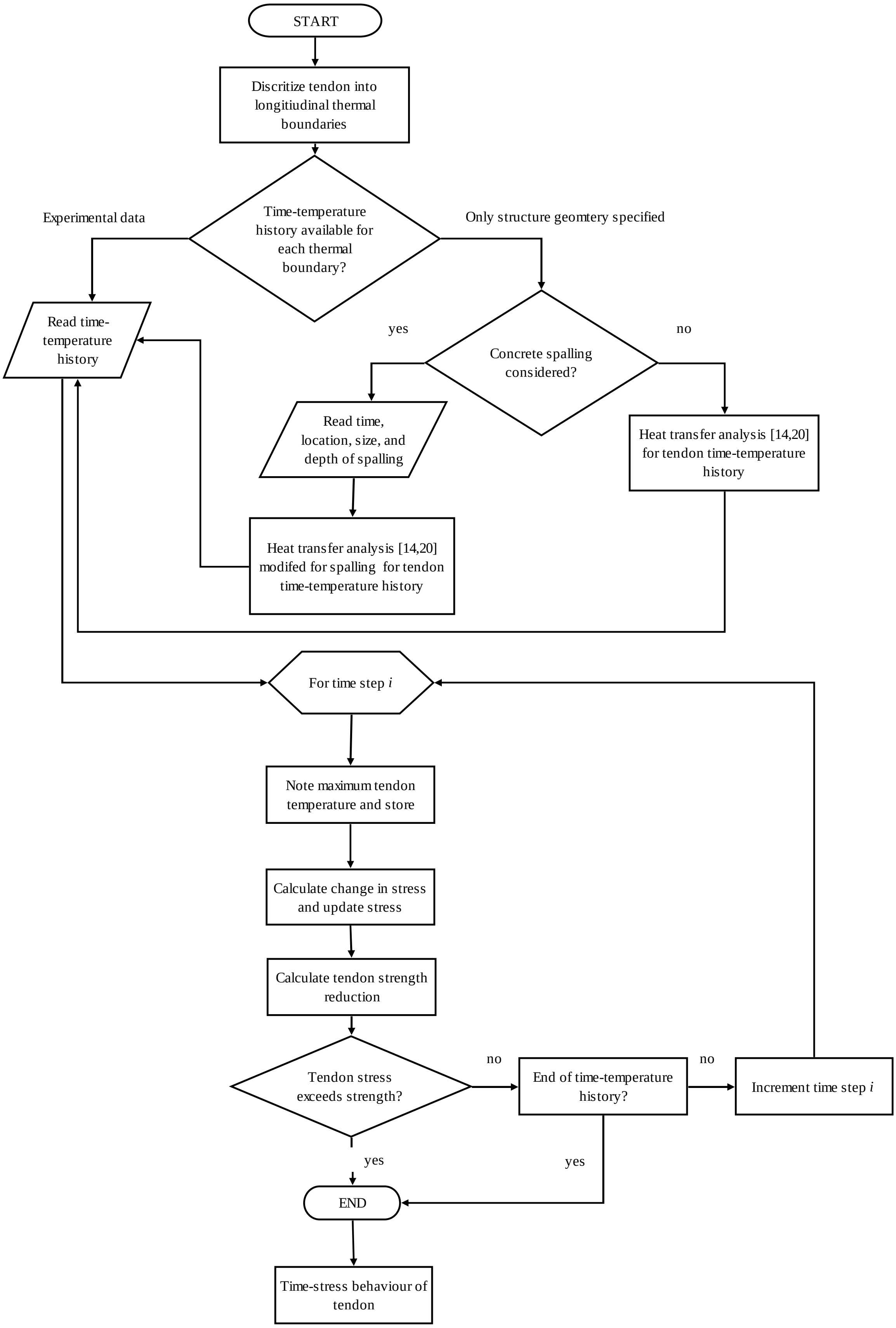

Figure 1: Flowchart of main computational model for transient high temperature prestress relaxation

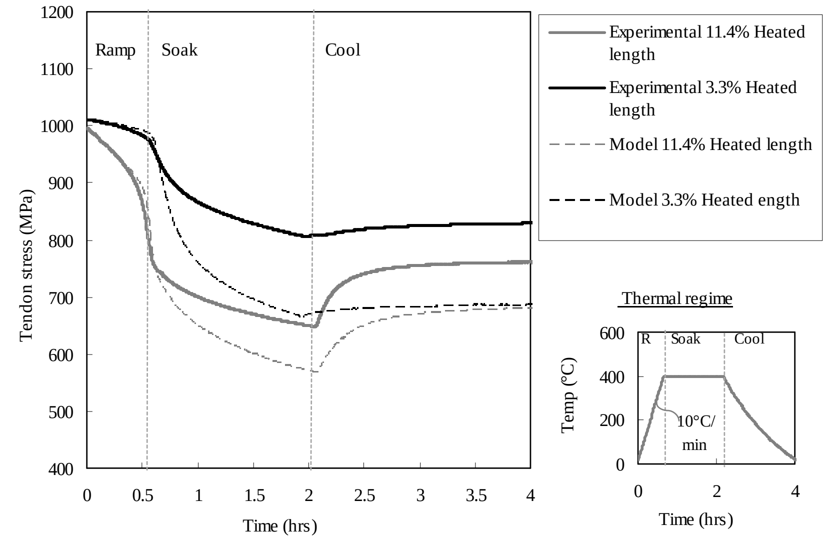

Figure 2: Predicted variation of tendons stress for tendons with different heated length ratios heated to 400ºC for 90 minutes and subsequently cooled to room temperature

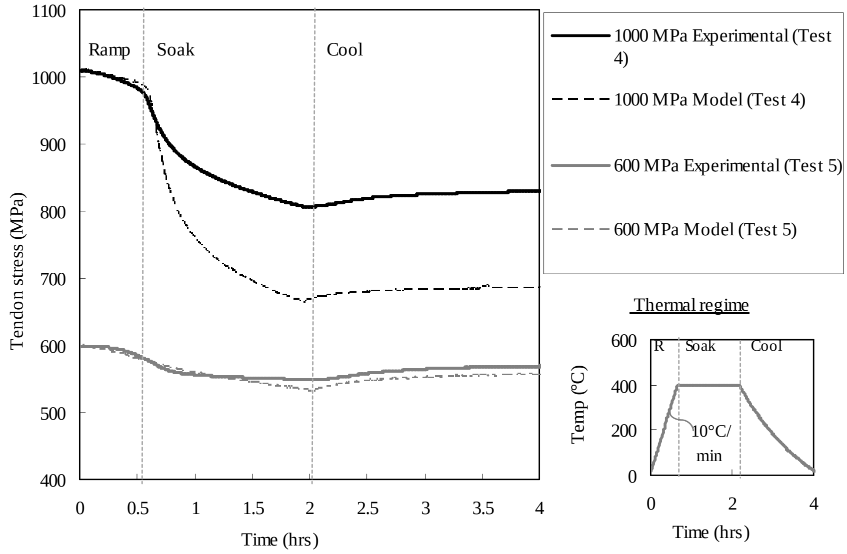

Figure 3: Stress versus time for varying initial prestess levels

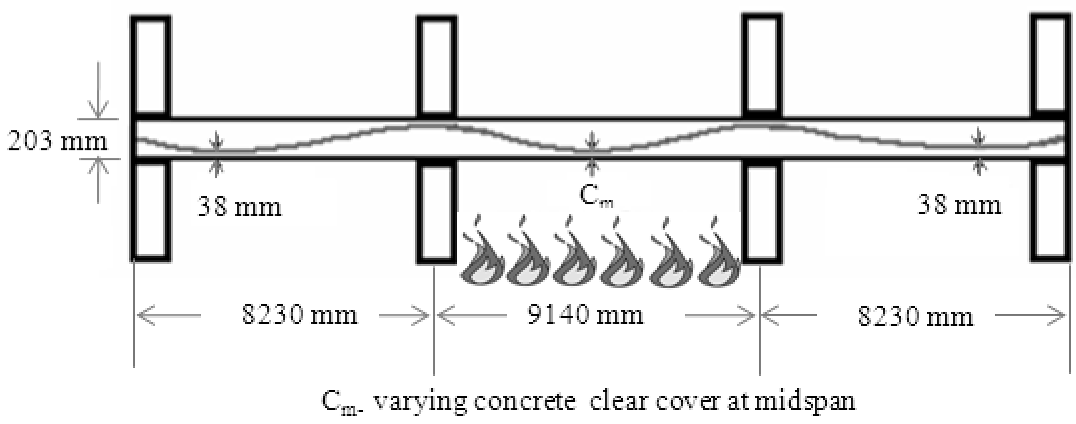

Figure 4: Schematic of the default example UPT slab configuration used in the current paper for computational studies (based on design example slab taken from reference [19])

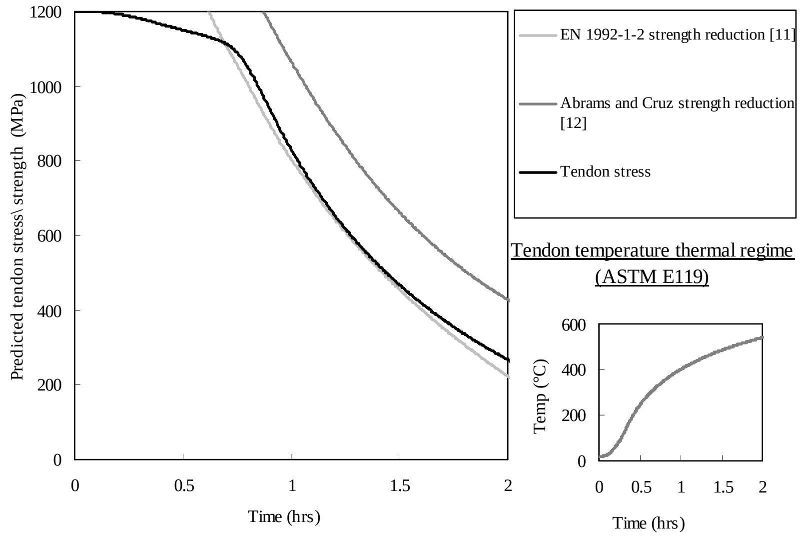

Figure 5: Predicted tendon stress variation with time for the default analysis of the 2 hour fire-rated example slab with 19mm concrete clear cover (28mm axis distance) in the central bay (refer to Figure 4)

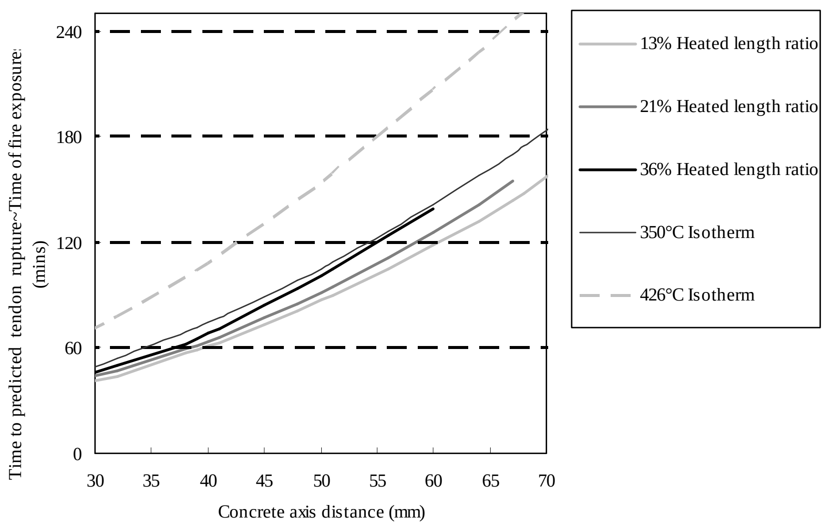

Figure 6: Calculated axis distance to prevent tendon rupture versus fire resistance time