TESTS OF CONTINOUS POST-TENSIONED CONCRETE SLABS UNDER LOCALISED FIRES

John Gales*, Luke Bisby, Tim Stratford, & Michal Krajcovic
BRE Centre for Fire Safety Engineering University of Edinburgh, UK

ABSTRACT

Modern concrete buildings increasingly rely on post-tensioned (PT) concrete for flat plate flooring (slab) systems. Post-tensioned concrete uses high strength prestressing steel (PS) tendons which, when tensioned after casting, pre-compress the concrete slab and result in excellent control of in-service structural deflections. However there are a number of load carrying mechanisms and failure modes that may be important for PT concrete structures in fire which have not yet been identified or adequately considered in the available literature. PS tendon continuity (particularly when unbonded) and thermal restraint can be expected to play significant roles in real PT structures. Because PS tendons can be unbonded and run continuously across multiple bays, tests on isolated, simply-supported members with short tendon lengths cannot be considered as representative. This paper presents the results of novel heated tests on three one way three span continuous and restrained monostrand PT concrete slabs (two of unbonded construction and one bonded). The slabs were tested under localised heating in their central span. Slabs were built with a realistic span to depth ratio as would be encountered in real construction. The slabs were rigidly supported on four steel columns of representative stiffness for a PT building; the result was three spans on four effectively fixed connections. The supporting columns were instrumented with strain gauges, making it possible to monitor the restraint forces generated during exposure to high temperature. Sustained loading was applied during testing using lead weights, and heating was applied using four radiant panels placed beneath the slabs. Thermocouples were placed at various locations throughout the slabs to provide thermal data, and a thermal imaging camera was used to generate heating profiles over the slab's soffit during testing. The slabs were monitored during both heating and cooling. The thermal and structural responses of these novel slabs during heating is presented and discussed.

INTRODUCTION

Post-tensioned (PT) concrete is increasingly used in modern building construction for flat plate slab flooring systems in concrete buildings. Post-tensioned concrete uses high strength cold-drawn prestressing steel (PS) tendons which, when tensioned inside ducts in the concrete after casting, compress the concrete slab and result in excellent control of in-service deflections compared to conventional (non-prestressed) reinforced concrete slabs. This prestressing results in secondary support reactions, thereby balancing the applied loads on the slab; this reduces use of building materials and enables large span-to-depth ratios which create additional space and open plan compartments. Post-tensioned concrete exists in two forms: unbonded (in which the PS tendon is greased and sheathed within the concrete, preventing bond to the surrounding concrete) and bonded (in which the PS tendon is grouted within a steel or plastic duct and thus continuously bonded to the concrete after tensioning). Post-tensioned concrete structures are increasingly popular as they help to meet stringent sustainability and aesthetic objectives in modern, optimized buildings. Post-tensioning has been used since the 1960s. In modern times, however, aided by higher strength concretes and computer aided structural optimization, PT construction has become increasingly complex – buildings of 60 storeys common – as compared with their historical counterparts.

The continual optimization of PT concrete buildings has resulted in structures requiring more engineering attention to guarantee safety in the case of fire. Prescriptive building code requirements, particularly in the United States, have failed to keep up with innovation. Optimization of PT concrete buildings was a key reason for the re-development of the ACI 318 code to be published in 2014[1]. As noted by Randall Poston, chair of ACI Committee 318:

"Today's flat-slab PT buildings… with columns spaced (12 m) on center and span-depth ratios of 40 are more complex and require more engineering attention than typical flat-slab buildings of 40 years ago, with columns spaced at (6 m) on center and span-depth ratios of 20."

The above statement presumably refers to the ambient temperature structural design of PT concrete buildings; however it can just as easily extend to issues around structural design for fire.

A key additional benefit for designers of PT slabs (whether explicitly acknowledged or not) is their widely claimed 'inherent' fire endurance. Confidence in PT concrete slabs' ability to resist fire was 'demonstrated' as early as the 1950s through a series of experiments conducted under standard fire conditions in large scale testing furnaces. However, the applicability of standardized fire tests to real buildings with real fires is debatable on various grounds[2]. Standard furnace tests fail to capture many of the complexities of real structural response in real fires. For example, when unbonded PS tendons run continuously within a slab across several bays of a building, local damage to the tendon will affect the response and structural capacity of adjacent bays; for evidence of this the reader is referred to the demolition of a fire damaged unbonded PT concrete frame discussed by Post and Korman[3]. In the event that no conventional bonded reinforcement is provided (as is permitted by some design codes[4, 5]) the PT concrete slab could thus lose tensile reinforcement across multiple bays, even in areas not directly exposed to the fire. A standard fire test of a simple span is thus incapable of capturing the real structural response. In addition, since most fire tests of PT concrete slabs are now more than 50 years old, the materials and construction techniques used in available tests are not representative of those used in modern PT concrete construction. To date, no fire tests have demonstrated the degree to which restraining or membrane actions in continuous PT concrete structures of realistic span-to-depth ratio can be created and sustained during a fire to prevent collapse. The performance of PT concrete slabs in fire has recently been the subject of debate within the UK structural engineering community[6, 7].

Because PS tendons are more sensitive to high temperature damage than mild steel reinforcement, PS requires larger concrete cover protection from fire, however this cover can be lost in a fire due to spalling, thereby directly exposing the PS tendon to severe heating. It is clear that a more complete understanding of the response of PT concrete slabs to fire is needed. This paper investigates some of the above issues for realistic, modern PT concrete structures in (and after) fire so as to steer future research towards the development of defensible, performance-based, fire-safe designs. A series of non-standard structural fire experiments on continuous, axially restrained PT concrete slabs are presented with a view to better understanding the response of these structures to localised high temperature exposure. Test observations are presented and their significance discussed.

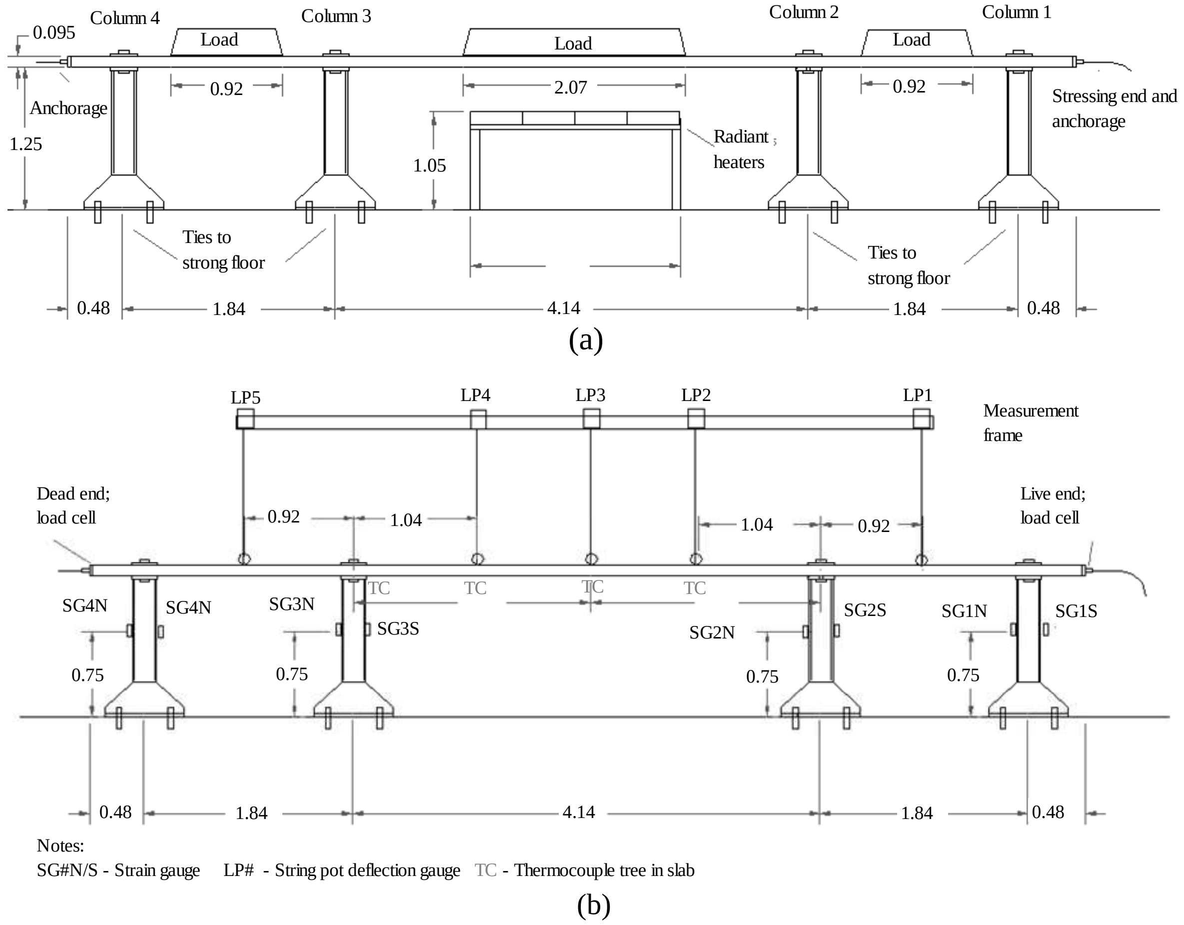
MOTIVATION

Prior research by the authors has studied the effects of high temperature prestress relaxation and possible heating induced rupture of unbonded PS tendons from different sized (i.e., localised or generalised) fires[8]. This previous work indicated that localised heating, as would be experienced in a real fire (travelling, ceiling jet, etc), is in fact the more critical exposure with respect to stress relaxation and rupture of unbonded PS tendons in fire. However, the high temperature response of a continuous, restrained unbonded PT concrete slab system has not yet been duly considered. To date little experimental data exist which shed light on the factors influencing the fire performance of realistic, modern PT concrete slabs. The experimental program is to study the structural and thermal response of simplified continuous PT concrete slabs, which incorporate as many of the relevant structural behaviours as possible, to severe localised heating. This is a critical step in the development of a rational understanding of the fire performance of PT concrete structures.

METHODOLOGY

Three realistically constructed one way continuous, restrained, monostrand PT concrete slabs were tested under localised heating. The slabs were designed and constructed specifically to investigate the influence of various potentially important parameters on the response of PT slabs during fire. Figure 1a gives the experimental set up, along with selected parameters and dimensions.

Figure 1
Schematic showing the (a) test setup and selected geometry and (b) instrumentation



* all dimensions in m

Slabs

The slabs were placed on four semi-rigid steel columns attached to a strong floor during testing. The principal goal was to design a one-way spanning restrained and continuous PT concrete slab. The slab spans needed to be restrained, vertically, axially, and rotationally; this would be representative of conditions in a real building. It was decided to use three bays with two short cantilevers. The minimum slab width-to-depth ratio was chosen as five, in accordance with CL 5.3.1 (4) of BS EN1992-1-2 so as to be classified as a slab rather than a beam[11]. The thickness and width of the slab were chosen accordingly, which resulted in a slab width of 475mm and a depth of 95mm.

A minimum span-to-depth ratio of about 40 was necessary for the centre span to be representative of modern PT construction. The length of the centre span was thus taken as 4140mm. The total length of the slab (8770mm) was restricted by the size of the strong floor in the Large Structures Test Hall at the University of Edinburgh. Cantilevers outside the end supports accommodated bursting reinforcement in the anchorage zones. Concrete cover (axis distance) was chosen to give a nominal one-hour fire rating, with bonded mild rebar at 25mm and tendon axis distance at 35mm at midspan (and 35mm to the top surface at supports). The specified drape of the monostrand seven-wire 12.5mm diameter Grade 1860 prestressing tendon followed a parabolic profile to give load balancing of the applied service load (as would be the case in design of a real PT building). Each of the three slabs was precast with eight connection points and four lift hooks (the location of which is not important for the current discussion).

The above parameters were identical for all three slabs; the slabs differed, so as to investigate the influence of overall tendon length and bonded versus unbonded PT construction, as follows:

- **Slab A:** The PS tendon was installed as unbonded (i.e. greased and sheathed).
- **Slab B:** To investigate the influence of bonded versus unbonded construction the PS tendon was installed as bonded (i.e. grouted within a plastic post-tensioning duct).

- **Slab C:** This slab was identical to Slab A with the exception that an innovative stacked disc-spring anchorage was used at the dead end. This anchorage was constructed using multiple disc-springs stacked in series and enclosed within a steel sheathing. This was connected to one fixed bearing plate which rested against the end of the slab. A second 'free' plate acted on the top of the spring inside the sheath. A hole through the spring anchorage allowed the tendon to pass through the middle of the spring anchorage. When the PS tendon was tensioned, this caused the 'free' bearing plate to compress the spring stack against the slab. The PS tendon also elongated at the live end of the slab. This compression of the spring stack is representative of a decreased effective stiffness of the unbonded PS tendon, thereby simulating the structural effects of a longer unbonded tendon length. A calibration exercise on the disc-spring anchorage confirmed the added simulated PS tendon length; an additional 8500 mm of PS tendon length in the configuration used.

To guarantee that Slab B would have the same concrete cover (axis distance) after tensioning as slabs A and C, the plastic PT duct was positioned to compensate for alignment changes during tensioning. To allow the slab to be precast before tensioning the PS tendons, sufficient positive moment reinforcement, running the length of the slab, was required to satisfy expected lifting stresses and to prevent excessive slab cracking prior to lifting, stressing, and testing. All mild steel rebar was 8mm diameter Grade 500 deformed reinforcement. Negative moment reinforcement was provided over all supports. The final slab configuration was verified for service stresses, flexural, shear, and final expected camber (both short term and long term). The concrete was ready-mix C40/50 with 10mm aggregate. Slabs were cured for more than six months.

Supporting Columns

The four supporting columns shown in Figures 1a and 1b were fabricated from steel sections and were designed on the basis of their axial and flexural stiffness at ambient temperature. A representative concrete column in a real PT building might have dimensions 450 × 450mm, an internal steel reinforcement ratio of 0.05%, 50MPa specified concrete compressive strength, and a storey height of 2.6m[13]. An appropriate commercially available steel section was selected to nominally match these properties (a 203 × 203 × 60 I-section made from Grade 275 steel). The column height was specified as 1.25m. The supporting columns were re-used for each test as they were never taken beyond their elastic range.

All previous testing available in the literature has failed to realistically represent restraining conditions that would act in a real PT structure due to lack of slab continuity and the use of either roller connections or flat plate frictional support conditions[4, 9, 10]. The columns used in the current study enabled restraint forces to be monitored by measuring elastic strains at predefined locations on the supporting columns themselves. Connections between the columns and the slab were via precast bolt-holes formed with steel ducting. The slab was placed on a grout bed on top of each column, and two Grade 10.9 bolts were installed to create rigid bond between the slab, column, and bolt. Rigid steel top plates were installed and the bolts were then tightened, resulting in a semi-rigid connection. After the slabs were connected to the supports, their PS tendons were tensioned to representative stress levels of 1165 ± 13 MPa (after short term losses).

The strain gauges installed on the supporting columns were calibrated by applying known loads with hydraulic jacks and measuring the strains at various locations. This allowed determination of the columns' true loading response; such that strains measured during fire testing could be correlated to restraining loads developed both vertically and longitudinally.

Loading

The load applied to the slabs during testing was calculated to represent the applied load (Dead + Imposed) divided by the theoretical capacity of the slab at ambient temperature. Applied distributed loading for of each span was provided using 70 individually weighed lead weights. The loading was applied over half of each span as shown in Figure 1a. This load was maintained constant throughout testing and gave a load ratio of approximately 0.5 with respect to the theoretical flexural capacity of Slab A at ambient temperature. Loading occurred after stabilization of short term tensioning losses.

Heating

Heating was applied using an array of four propane-fired radiant panels placed 20 cm beneath the soffit of the slab at midspan. The radiant panels were mounted in a horizontal steel frame and the entire heating array was surrounded with ceramic fiber boards during testing to prevent direct radiative and convective heating of adjacent portions of the slab and the supporting columns. Air was supplied by four high-speed electric blowers, and a digital mass flow controller was used to regulate the gas flow upstream of the gas manifold. The flow of propane was set and maintained at 1.4g/s for all tests to ensure reproducible heating profiles between respective tests. The burners were lit by an automatic sparking source. The radiant panel array had a total length of 2070mm and width of 450mm, resulting in the slabs being subjected to uniform heating over approximately 50% of their central spans; or about 23% of their total tendon length. It is noteworthy that Slab C, with additional spring-simulated tendon length, had a simulated heated length ratio of about 13%.

Slabs A and B were heated until it was assured that the 'critical temperature' of the PS tendon was reached according to BS EN 1992-1-2 (i.e. 350ºC was recorded at the tendon location)[11]. This resulted in a total heating time of approximately 200 minutes and 215 minutes for slabs A and B, respectively. Slab C was heated until the higher critical temperature used in North American guidance was reached (i.e. 427ºC at the location of the tendon)[12]. This resulted in a total heating time of 270 minutes for Slab C. After these criteria were achieved the propane supply was turned off and the panels were removed from beneath the slabs to allow them to cool to ambient temperature. Each Slab was continuously monitored for 24 hours during both the heating and cooling phases.

INSTRUMENTATION

Previous testing reported in the available literature[4] has shown that standard fire testing on UPT elements has used materials and/or techniques that are either too old to be representative of current practice, dimensions which cannot be considered typical of modern construction, and/or support conditions which are unrealistic. These factors may lead to structural behaviour that has yet to be considered for fire-safe design of these systems. The PT concrete slabs described in this paper begin to explore some of the key issues. Four main types of measurements (see Figure 1b) were of interest in the experiments reported herein:

(1) **Slab temperatures** are crucial for predicting unbonded PS tendon stress relaxation and the thermal/structural response of the slabs. Numerous embedded Type K thermocouples were cast into each slab to provide thermal data during testing. Thermocouples were placed at midspan, with three thermocouples installed across the soffit of the slab, two thermocouples within the concrete at the axis distance of the PS tendon, one thermocouple at the depth of the flexural reinforcing steel, and two thermocouples along the unexposed (top) surface. One supporting column adjacent to the heated central span was monitored for temperature changes, with thermocouples placed along the head of the column. Additional thermocouples were installed at the quarter points of the central span. A thermal imaging camera (FLIR A320A) was also used to record the spatial and temporal distribution of slab soffit temperatures during heating.

(2) **Tendon load (stress) level** was measured using through-hole load cells installed at both ends of the slabs to give an indication of the PS tendon's stress level throughout the tests.

(3) **Column strain** was measured using two calibrated electrical resistance foil strain gauges on each column face. Because the columns were calibrated for testing, these gave an indication of the restraining forces exerted by/on the slabs during testing.

(4) **Slab deflections** were monitored using five string-pot displacement transducers installed at midspan of each span and at the quarter points of the centre span. These were attached to an independent instrumentation frame which spanned the full length of the slab. The instrumentation frame was protected against heating using 100mm thick Ceraboard™. The deflections are crucial to investigate continuity effects and behavioural trends during the fire tests, and for future validation studies on finite element modeling capability.

A Vishay Measurements Group™ System 7000 data acquisition system was used to record all sensor data from the instrumentation during testing. A digital image correlation technique was also used to measure slab and column displacements, deflections, strains, etc, however the image correlation data are not presented herein due to space limitations.

RESULTS AND DISCUSSION

Slab temperatures

Thermal data were used to demonstrate the uniformity of heating on the slabs' soffits, to monitor the similarity of temperatures experienced between tests, and to indicate the magnitudes of thermal gradients in the slabs. The reinforcement temperatures were measured to inform the deformation (i.e. prestress relaxation) and strength reductions of the PS and bonded reinforcing steel during testing, based on prior tests and modelling by the authors[8].

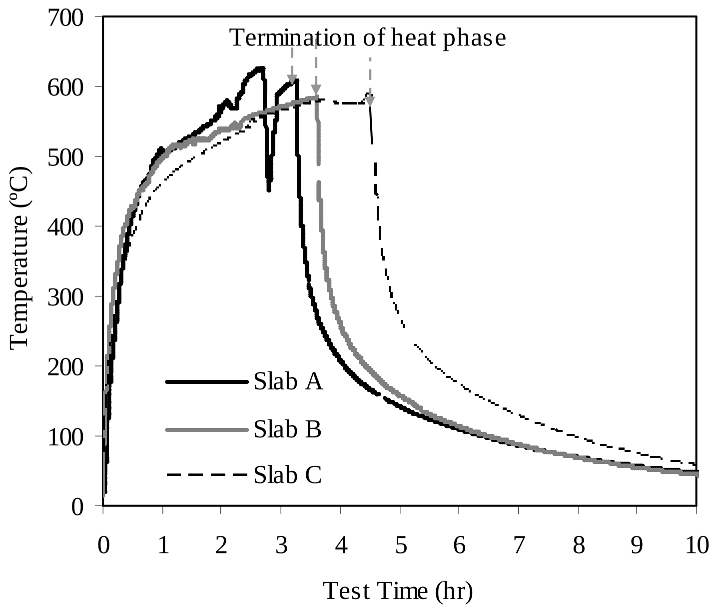
The repeatability of thermal exposure for the three tests is crucial for making comparisons of slab response, and this is influenced primarily by the performance of the radiant panel assembly (both spatial and temporal) between tests. A comparison of recorded soffit temperatures from all three tests is given in Table 1, which presents temperatures at midspan during heating. The data present an average of the three thermocouples placed at midspan. Slab C was heated for a longer total duration, so Table 1 includes data only up to three hours of heating. In general there was reasonable consistency of heating between all three tests, confirming good repeatability using the radiant panel assembly. The complete heating cycles measured at the centre most soffit thermocouple at midspan are shown in Figure 2 for all three slabs. A dip in the measured temperature for Slab A was caused by the radiant panels being turned off temporarily when a 'blow-back' occurred and the panels had to be re-lit.

Table 1. Comparison of soffit thermocouples at midspan for first heating of slabs A-C

Test time (minutes)	Slab A (°C)		Slab B[a] (°C)		Slab C (°C)		Averaged results (°C)
	Average	Deviation	Average	Deviation	Average	Deviation	
30	416	9	427	42	383	13	402+/-21
60	508	33	496	45	461	19	488+/-32
120	562	34	538	27	520	25	540+/-28
180	593[b]	-[b]	571	17	564	48	576+/-32

[a] Data for Slab B is based on 2 thermocouples due to an instrument malfunction
[b] Data is based on only 1 thermocouple so no deviation could be calculated

Figure 2
Soffit temperatures of slab recorded by centre most thermocouple



Thermal imaging was used to explore spatial distributions of temperature along the slabs' soffits during heating. The cooling phase data could not be recorded by the thermal camera as the radiant panels were moved from beneath the slab, thus blocking the camera's view of the slab's soffit.

Given that four radiant panels were used it was expected that there might be some non-uniformity in heating. However the maximum deviation observed along the length of the PS tendon according to the thermal data was never greater than 40°C during these tests, as compared against the temperatures measured using thermocouples. It can thus be concluded that there was reasonably good uniformity of heating over the entire heated region. Temperatures on the unexposed surface of the slabs at midspan are plotted in Figure 3 (average of two thermocouples). With reference to Figures 2 and 3 the total heating through thickness gradient in the slab tests eventually exceeds 450°C; thus considerable thermal bowing would be expected.

Figure 3
Unexposed surface temperatures on slabs, averaged of three midspan thermocouples

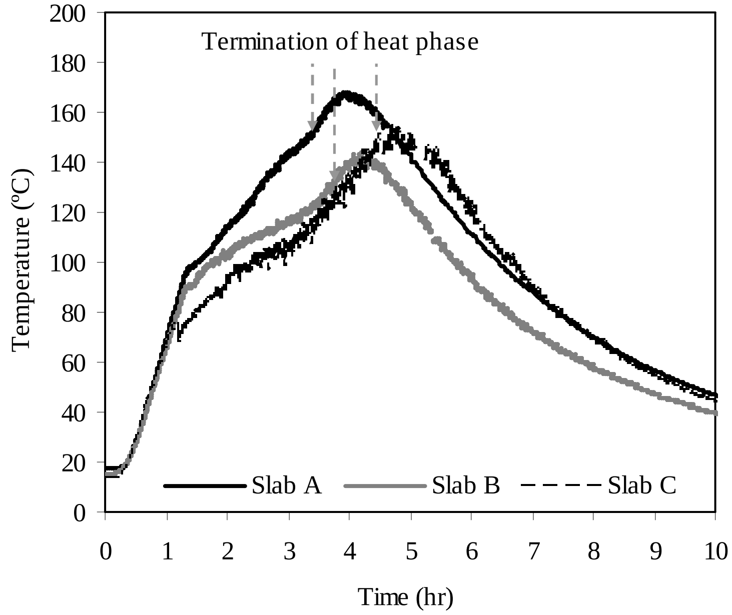


Figure 4
Averaged PS tendon temperatures recorded at the tendon axis distance

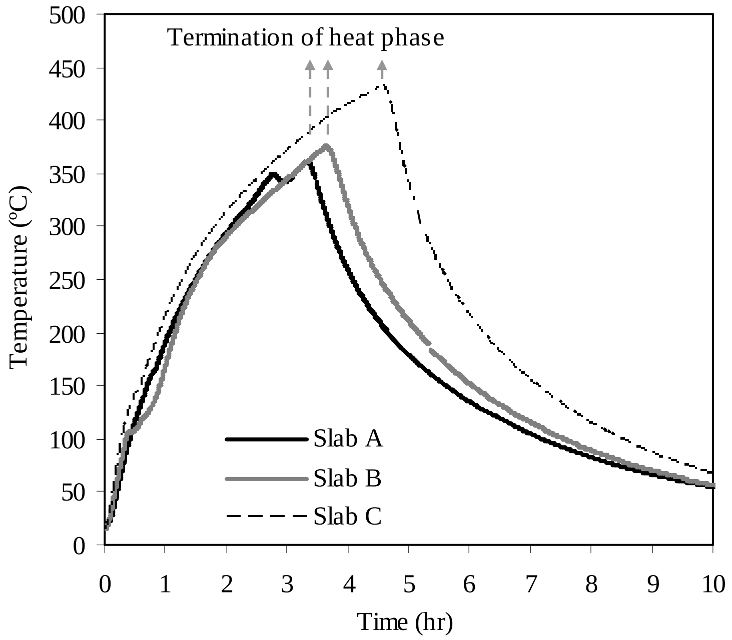


Figure 4 shows the average of the two PS tendon temperatures taken from thermocouples on the tendon at midspan for all three slabs; it is assumed that this would be the highest tendon temperature recorded anywhere, since this is where maximum tendon drape occurred and was the most heated location in terms of both radiative view factor and direct convective heating from the radiant panels. The deviation between the two tendon thermocouples in each slab never exceeded ±2.2°C (the accuracy of a Type K thermocouple). Slab B demonstrated a reduction in heating rate in the temperature region near 100°C, which is thought to be due to the higher (measured) moisture content of the cementitious grout which was used to create a bonded PT situation within the plastic post-tensioning duct at the time of testing; this was 6% moisture content (by mass) as compared with the surrounding concrete in which is was about 4% (also measured at the time of testing).

Tendon stress

The relationship between PS tendon stress and temperature is critical for PT structures, and under certain combinations of the two tendons may experience tensile failure due to heating alone[8]. The variation of tendon stress with tendon temperature measured at midspan is plotted in Figure 5 for Slabs A and C. Since Slab B had a grouted and bonded PS tendon, this slab showed no measureable stress relaxation during heating, as observed from the load cells installed at the tendon anchorages.

Figure 5 shows that a considerable amount of stress relaxation was observed for both slabs with unbonded tendons. This is due to a combination of thermal and structural actions including thermal elongation of the tendon, thermal expansion of the concrete, thermal bowing of the concrete, reductions in elastic modulus of the tendon at elevated temperature, and creep of the tendon (which is a time, load and temperature dependent plastic deformation that accelerates at elevated temperature). Prior work by the authors has discussed these aspects in considerable detail [8]. Reductions in tendon stress level increase and accelerate with increasing temperature, and even continue once the heating is stopped due to the thermal inertia of the concrete.

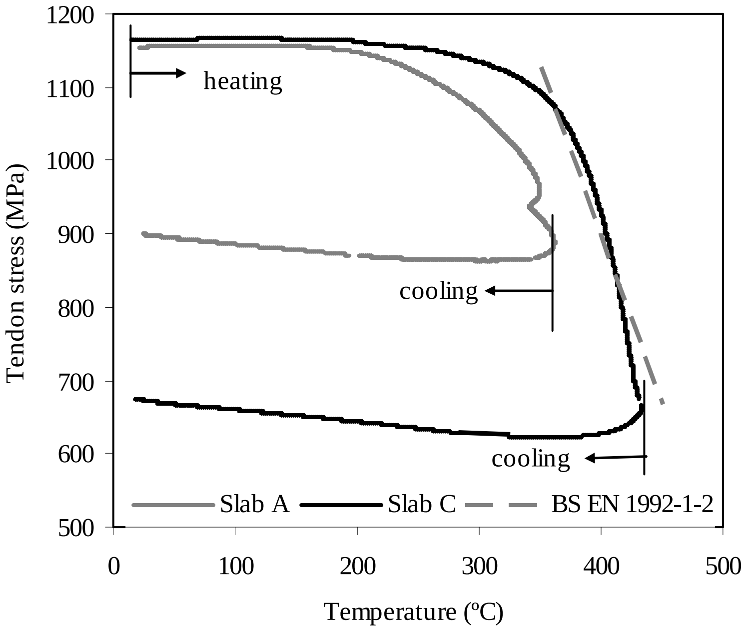
A minimal amount of stress recovery was observed during the cooling phase, indicating considerable irrecoverable creep of the tendons. The potential for irrecoverable creep in unbonded prestressing tendons in fire has been postulated previously[4,5], and the data in Figure 5 confirm that this is a reality in real structures. The observed stress relaxation (or lack of stress relaxation) of all tests will have an impact on a structure's ability to balance applied loading and other inter-related consequences on the PT slabs. Relative restraining forces and deflection of the slab (cracking mechanisms aside) would be directly impacted from stress relaxation. This is because the PS tendon's resulting pre-compression on the slab will diminish on heating, as well as its ability to balance applied loading. Whether such structural actions could be defensibly modelled using the best available finite element codes, even for this simple case with well defined thermal profile, remains an open question.

As already stated, one issue of concern in UPT slabs during fire is that localised heating of the prestressing tendon could result in tensile rupture. Strength reduction relations with temperature are available for cold-drawn PS wire in literature, for example in BS EN 1992-1-2, and this is also plotted as the dashed line in Figure 5[11]. Slab A did not seriously risk tensile failure of the tendon at any point during testing. Slab C included the aforementioned disk-spring anchorage which simulates a longer tendon length and thus a shorter heated length ratio (13% instead of 23% as in Slab A). Plotting the stress relaxation-temperature results for Slab C in comparison of Slab A confirms that for shorter heated length ratios less irrecoverable relaxation will occur. This was previously numerically predicted by the authors[5].

Slab C indicates an interception with BS EN 1992-1-2 strength reduction curve for PS steel at high temperature. Post fire evaluation of the slabs confirmed a discrete location of necking on the PS tendon representing 10% area reduction and indicating that the tendon was indeed on the verge of failure during the test. This gives some credibility to the BS EN 1992-1-2 reduction factors as a conservative means by which to predict PS tendon failure under tensile stress at high temperature. However, it must be noted that heated length ratios below 13% represent the more likely fire scenarios in real buildings, and these were not tested. This would be akin to a multi bay structure of 40m total length heated in a single bay (or compartment). For larger structural configurations (>40m), smaller heated length ratios could be expected in practice and therefore premature tendon rupture prior to code prediction is a legitimate concern. In the case of Slab C, had the spring simulated and even greater length of unbonded tendon, such as to simulate just one additional structural bay, the tendon would most certainly would have ruptured. The above discussion has not considered the possible effects of more rapid heating, which would result in less time-dependent prestress losses and an earlier stress-strength interception. The implications of PS tendon rupture and possible consequential load shedding mechanisms must be explored in future research if North American code guidance is to be justified as conservative.

Figure 5

Prestress relaxation unbonded PS tendons relative to tendon temperature during heating and cooling



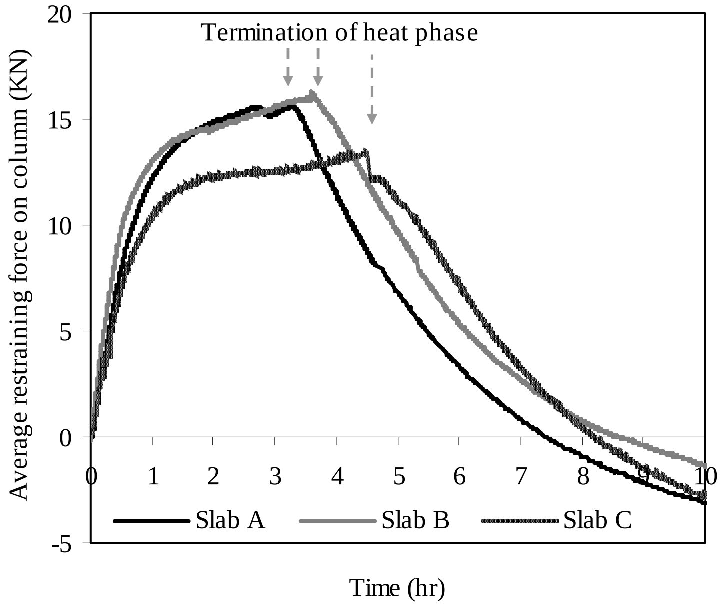
Restraining forces

It is well known that restraining forces and mechanisms may benefit the performance of structural systems during fire. However there are cases where high restraining forces can influence the formation of perimeter cracking, column shear loading (and even failure), and/or slab spalling due to induced compressive stresses. Restraining forces are generated based on the boundary conditions at the supports and the stiffness of the surrounding structure. Each column had two identically placed and calibrated strain gauges which allowed the columns to be used effectively as retraining load cells. Figure 6 gives the average longitudinal thrust experienced in each of the slabs during heating (relative to zero thermal thrust the start of the tests). Also shown is the thermal contraction induced *tension* which is developed during cooling for all tests. All columns generated similar push pull responses, never deviating from their average by more than 3kN.

A similar push-pull response was observed for each test. Each slab's 'push' restraining force during heating follows the soffit heating trend (see Figure 2). For instance in Slab A, when the panels briefly fail during heating the restraint force changes. It is important to note that the observed restraint force results apply for one boundary (heat and support) condition only. The forces are relatively small during the current tests; however they are representative of a thin strip of slab as tested. When considered for a real slab that is two way and placed on a 4m × 4m grid (while still heated within a centre bay over half its area), a restraining force of over 100kN or more might be induced on each column head.

In addition, had more of the slabs' soffit been heated or had the slabs been heated to a higher temperature it is likely that larger thermal thrusts would have been induced. This raises important questions about the shear capacity of supporting concrete columns and their ability to resist lateral and axial forces to prevent progressive collapse mechanisms. For rational design in a performance based environment, the likely fire scenario and likely end/intermediate support conditions must therefore be known. Knowledge of these boundary conditions is essential, otherwise any structural analysis will have little rational basis.

Figure 6
Average longitudinal restraining thrust forces generated on the supporting steel columns adjacent to the central span during heating and cooling



Slab deflections
To date no experiments have carefully described the deflection behavior of a restrained and continuous (one-way) PT slab during localised heating. The deflection response is crucial to identify and understand various interacting mechanisms occurring in the structural system during fire; and as such it is also crucial for validating numerical models of PT slabs during fire.

Figure 7 shows the respective vertical deflections at the midpoints Slabs A, B, and C during testing, with deflections zeroed at the onset of heating. In all cases quarter point deflections (not shown) were approximately one half the midspan values. End spans (unheated) displayed negligible deflections. All tests displayed four distinct deflection trends during heating:

(1) **Thermal bowing** – high thermal gradients developed during initial heating promoted thermal bowing early in the tests and caused a rapid downward deflection in all cases. This reached a peak of 9-10mm within about 30 minutes of heating in all cases.
(2) **Concrete stiffness degradation** – When the rate of heating on the exposed surface of the concrete began to slow and the soffit temperature of the concrete exceeded 450ºC, the slabs' deflection reversed in direction and the slabs cambered upward. This response was likely due to transient (time and temperature) creep damage and stiffness reduction of the concrete close to the heated face. A reduction in slab stiffness and sufficiently high tendon prestress (>1000MPa at this time of testing, see Figure 8) promoted cambering of 5-7mm within the heated region only. This increasing in camber persisted until approximately two hours into the tests.
(3) **PS tendon creep** – After the PS tendons reached about 300ºC, prestress relaxation of the tendons accelerated due to creep (and thermal elongation) of the PS tendon. This reduction in tendon stress caused global increases in deflection, thus canceling out the cambering response of the locally heated region.
(4) **Cooling** – Tests were halted when the tendon had reached its critical temperatures, so that the cooling-phase response could also be observed. The deflection reversed on cooling due to thermal contraction and reductions of thermal bowing. Slabs A and C experienced nearly the same recovery of deflection as occurred due to thermal bowing. Interestingly all slabs exhibited a final upward (camber) after cooling. This is indicative of permanent plastic deformation of the concrete. Slabs A and C exhibited more deflection as compared with Slab B, this likely due to prestress relaxation in the tendons. The bonded slab maintained prestress and thus had the least overall deflection throughout.

Figure 7
Slab vertical deflections recorded at midspan during heating and initial cooling phases

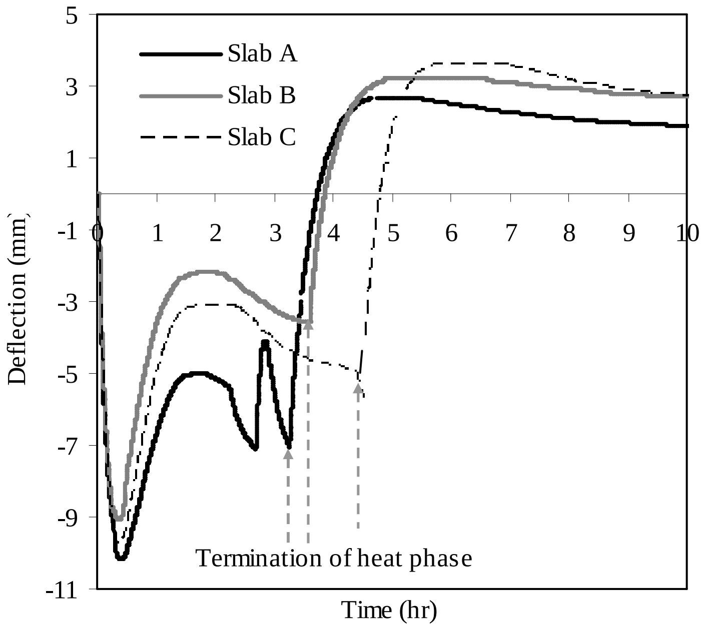
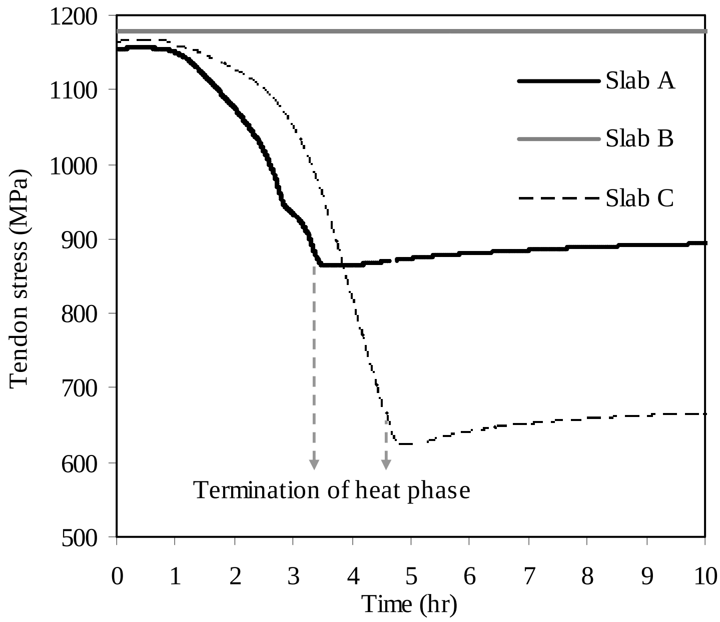


Figure 8
Tendon prestress relaxation with time during heating and initial cooling phases



CONCLUSIONS

The main objective of the experiments described in this paper was to investigate the thermal and structural response of multiple span continuous, restrained one-way PT concrete slabs exposed to localised heating. Despite the comparatively simple layout of these tests and the well defined uniform heating applied to the slabs' soffits, the resulting global response is highly complex and demonstrates numerous thermal and mechanical actions which would need to be carefully considered to rationally model even these extremely simple PT concrete structures. However these tests enable initial efforts to build rational models which account for the requisite complexities; this work is currently underway by the authors. In the meantime, several important conclusions can be formulated on the basis of the test results presented herein:

- Considerable time dependent tendon prestress losses occurred during heating. The magnitude of this prestress relaxation was primarily influenced by the heated length ratio, as well as time, temperature, and loading conditions.
- Premature tendon rupture is a realistic concern for unbonded PT slabs in fire, and the likely scenario for smaller heated length ratios has been proven, thus validating previous experimentation and modeling by the authors. While the BS-EN 1992-1-2 tendon rupture predictions (proposed strength versus temperature reductions) appear defensible on the basis of the experiments presented; North American guidance with a higher critical temperature appears not to be defensible for realistic heated length ratios. Therefore the effect of fire-induced tendon rupture on the global structural response of a PT building should be further investigated.
- Considerable restraining forces will be generated when a PT slab is exposed to localised heating; these will be of sufficient magnitude to cause distress to supporting columns.
- The continuous and restrained, one-way spanning PT slabs tested herein exhibited four distinct phases of deflection, as described previously. These trends are influenced by a complex interplay between stiffness degradation of both concrete and PS tendons, plastic and creep deformation, and thermal expansion/contraction. Any attempt to numerically model such systems in fire must defensibly account for this complexity to be considered credible.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the support of Ove Arup and Partners, the Ove Arup Foundation, the Royal Academy of Engineering, the University of Edinburgh, and Natural Sciences and Engineering Research Council of Canada.

REFERENCES

[1] Poston, R., Dolan, C. 2008, Re-organizing ACI 318, Concrete International, July 2008, p 43-37

[2] Bisby, L., Gales, J., and Maluk, C. 2013. A Contemporary Review of Large-scale Non Standard Structural Fire Testing (crica 1980-present). Accepted. Fire Science Reviews.

[3] Post, N., Korman, R., Implosion Spares Foundations, Engineering News Record, 2000, p 12-13.

[4] Gales, J., Bisby, L., Gille, M. 2011, Unbonded Post-Tensioned Concrete in Fire: A Review of Data from Furnace Tests and Real Fires, Fire Safety Journal, 46, p 151-163

[5] Gales, J., Bisby, L., Gille, M. 2011, Unbonded Post-Tensioned Concrete Slabs in Fire-Part 2-Modelling Tendon Response and Consequences of Localized Heating, Journal of Structural Fire Engineering, 2(3), p 154-171

[6] Kelly, F and Purkiss, J. 2008, Reinforced Concrete Structures in Fire: A Review of Current Rules, The Structural Engineer, 86(19), p 33-39

[7] Bailey, CG., Ellobody, E. 2009. Whole-Building Behaviour of Bonded Post-Tensioned Concrete Floor Plates Exposed to Fire, Engineering Structures, 31, p 1800-1810

[8] Gales, J., Bisby, L., and Stratford, T. 2012, New Parameters to Describe High Temperature Deformation of Prestressing Steel Determined using Digital Image Correlation, Structural Engineering International, 22(4), p 476-486

[9] Zheng, W., Hou, X and Xu, M. 2007. Experiment and Analysis on Fire Resistance of Two-span Unbonded Prestressed Concrete Continuous slabs, Journal of Building Structures, 28(5), pp 13 (in Chinese)

[10] Aimin, Y., Yuli, D., Dai, H,. Li, Y. 2008, Behaviour of Unbonded Prestressed Continuous Concrete Slabs with the Middle Span and Edge Span Subjected to Fire In Order. Journal of Harbin Institute of Technology 40(10), pp. 6 (in Chinese)

[11] BS-ENV-1992-1-2. 2004, Eurocode 2: Design of concrete structures, Parts 1–2: General rules-structural fire design. European Committee for standardization, Brussels

[12] IBC. 2009. International Building Code, International Code Council, USA

[13] CPCI. 2007. Design manual: precast and prestressed concrete. Canadian Prestressed Concrete Institute. Ottawa Canada.