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Unbonded Post Tensioned Concrete in Fire: A Review of Data from Furnace Tests and Real Fires

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ABSTRACT
The fire-safe design of concrete structures which incorporate post-tensioned prestressing tendons has recently been the subject of debate within the structural engineering community, particularly when unbonded post-tensioned (UPT) prestressing tendons are used. Despite several studies aimed at furthering our understanding of the response of UPT concrete structures in fire, many aspects of their response in real fires remain poorly understood. An exhaustive summary of available test data which have been used over the past five decades to generate fire design guidance for UPT concrete structures is given. Case studies showing the response of real UPT structures in severe building fires are also discussed. In both cases the intent is to highlight inadequacies in the current state of knowledge for UPT buildings in fire and to prioritize areas for future research.

Key words: Prestressing steel; Post-tensioned slabs; Unbonded construction; Fire endurance testing; Spalling; Concrete; Concrete cover; Case studies.

1. INTRODUCTION

Unbonded post-tensioned (UPT) concrete is an increasingly popular method of construction, since it allows for rapid erection of economical and sustainable buildings. While its use has been widespread in the United States since the 1960s, it has recently seen wider use in the UK, Europe, China, and the Middle East. UPT structural elements are more efficient than non-prestressed elements, making optimal use of the materials from which they are built [1]. However, the structural optimization and efficiency of UPT elements generate potential concerns associated with their performance during fire. Indeed, there has been some debate in recent years [2, 3] regarding the fire safety of post-tensioned concrete structures (flat slabs in particular). Some of the concerns specific to UPT beams and slabs in fire are openly acknowledged (and to a certain extent addressed) by available structural design codes. For instance, the fact that prestressing steel is more sensitive to temperature than mild steel reinforcement, and suffers proportionally greater losses in strength and stiffness as temperatures increase in a fire, is typically reflected by larger prescribed concrete covers to prestressed reinforcement. However, many potential, credible concerns have not received sufficient research attention, and available prescriptive rules for fire-safe structural design of UPT beams and slabs appear somewhat outdated on the basis of available information, particularly given current trends toward performance-based design for fire.

Available experimental data on the fire performance of UPT structures come from relatively few unrealistic and outdated large-scale furnace tests on isolated UPT beams and slabs [3-11]. It appears that no realistic fire tests of multiple bay continuous UPT elements have ever been performed. While available standard furnace test data are instructive, they are neither representative nor necessarily conservative with respect to the performance of real UPT buildings in real fires [12]. Indeed, available case studies of real fires in UPT buildings [13-18] suggest that when UPT beams or slabs fail in fire it is rarely for the reasons that would be expected on the basis of available testing [3-11] or design guidance.

This paper focuses on the structural fire performance of modern flat-plate UPT concrete slabs. A detailed summary of the available peer-reviewed furnace test data, which have been used over the past half century to generate current fire design guidance for UPT beams and slabs, is provided. Also presented are several case studies showing the response of real UPT buildings in real fires. The intent is to highlight the current state of knowledge of the fire performance of UPT buildings and to prioritize future research areas. A less exhaustive literature review on this topic has been presented previously by Lee and Bailey [19]; however this provided insufficient detail to clearly highlight the inadequacies of current knowledge.

2. MOTIVATION AND SIGNIFICANCE

It is acknowledged at the outset that the performance of concrete structures in real fires is generally very good. It seems clear, however, that current fire design requirements for UPT buildings are based on a fundamentally limited understanding of their response to fire; the absence of widespread evidence of failures should not be construed as evidence of satisfactory response. Available fire experiments on UPT elements have either been furnace tests on isolated beams or one-way slabs [9-11], with their numerous, well-documented shortcomings [20], or are so old as to be of limited relevance to contemporary concretes and construction methods [4-7]. Existing prescriptive design guidelines for fire-safe structural design of UPT buildings [e.g. 21-24] typically specify only minimum member dimensions and minimum concrete cover to the UPT reinforcement; this approach is insufficient to guarantee safety. Many other factors which have not previously been adequately considered may also be important for UPT structures in fire. These include:

- **Tendon continuity across multiple bays**: UPT buildings contain unbonded tendons which are continuous over multiple bays and which may exceed 70m in length [25, 26]. Unlike bonded PT buildings, in UPT buildings the tendons are free to move longitudinally within ducts within beams and/or slabs. Localized damage to a tendon, whether due to fire or other factors, thus has consequences across all bays of the structure. Furthermore, research has shown [27] that the longer the total length of an unbonded tendon between anchors, the greater the likelihood of tendon rupture due to localized heating. This rupture is due to a combination of accelerating creep strains and loss of tensile strength of the tendon. The consequences of premature tendon rupture during fire have received no research attention to date, despite the fact that tendon rupture during fire has led to progressive failure of several floors of a UPT building [17], and to demolition of several structures after real fires [15, 28].
- **Higher strength concrete**: A trend in the design and construction of modern UPT concrete structures is to use higher concrete compressive strengths than has traditionally been the case. Research has shown that modern, high strength concrete mixes are more likely to experience fire-induced explosive spalling than their low strength counterparts [29]. Thus, satisfactory performance of UPT elements in fire resistance tests conducted more than 25 years ago cannot necessarily be invoked as evidence of adequate fire resistance for modern UPT structures. The fact that higher concrete strengths are used in modern UPT elements also results in less reserve cross-sectional capacity being available should fire-induced spalling occur.
- **Pre-compression**: UPT slabs and beams are axially pre-compressed under service loads, and in general this means that a greater proportion of the soffit of a UPT element will be subjected to compressive stresses in service than would be the case for a non-prestressed member. Since compressive stress is a widely acknowledged risk factor for spalling in fire [30], it is reasonable to assume that that UPT beams and slabs are more likely to spall than their non-prestressed counterparts (all other factors being equal). This is not explicitly considered in available UPT design guidance.

- **Large span-to-depth ratios**: A primary advantage of UPT structures is that they enable larger span-to-depth ratios than non-prestressed flooring systems [31]. A UPT slab may therefore experience proportionally greater deflections during fire, due to thermal bowing and smaller lateral restraint forces, than would occur for a non-prestressed slab. There is therefore less chance of developing beneficial compression membrane action for larger UPT span-to-depth ratios. The beneficial effects of compression membrane action in preventing collapse of two-way reinforced concrete flat plate slabs in fire, even when the majority of the bottom steel reinforcement is lost due to heating and/or cover spalling, was clearly shown during the concrete frame fire test performed at Cardington in the 1990s [32]. Bailey's [32] post-fire assessment of the Cardington concrete frame noted that, "during the test on the concrete building, the slab's vertical displacement was small and the static load was supported due to compressive membrane action." However, Bailey also notes that, "if the slab's vertical displacements were greater... then it is difficult to see how the slab could have supported the static load." Inability to engage compressive membrane action is a credible concern for UPT slabs in fire, particularly since UPT tendons are reasonably likely to rupture during fire (as discussed below) and that some design guidelines allow zero mild steel reinforcement in sagging moment regions [23, 33, 34]. Furthermore, slender UPT slabs have larger buckling lengths and are pre-compressed. This increases the potential for global floor-plate buckling under the influence of lateral thermal restraint from surrounding bays. It can be argued that floor-plate buckling would send a UPT slab into tensile membrane action in fire, which could prevent collapse, although this can only be engaged if the UPT tendons are intact and/or if sufficient mild steel reinforcement is present. It is also worth noting that the span-to-depth ratios used in available furnace tests on UPT members have generally been unrealistically small due to limits in available furnace sizes.
- **Fire resistance based on concrete cover** – Available structural fire resistance ratings for UPT structures are based on minimum member dimensions and on providing sufficient concrete cover to the prestressed reinforcement [21-24]. The prescribed covers are based largely on results of standard furnace tests conducted on isolated UPT slabs and beams during the 1950s and 1960s [4-8], although some codes have been revised based on more recent furnace testing [9, 23]. As described below, in some cases these tests used concrete which had been pre-conditioned before fire testing, or used a light steel mesh within the concrete cover to prevent or arrest spalling during fire. Cover spalling was thus explicitly excluded from many of the tests, yet these important details are routinely omitted when invoking these test data to justify current prescriptive cover requirements. Clearly, use of light steel mesh in the cover and pre-drying of the concrete are non-representative of typical UPT construction. This, along with the other concerns noted above, casts doubt on the wisdom of relying on prescriptive concrete cover as the sole means of achieving fire resistance in UPT buildings.
- **Shear-critical designs** – Because UPT slabs allow shallow, flat floor-plates over large spans they are often shear-critical under ambient design loads [12, 31]. Such slabs have less reserve shear capacity than their non-prestressed counterparts, making them more susceptible to shear failure during fire. Furthermore, many international UPT design standards [e.g. 23, 33, 34] permit designers to consider the pre-compression from the UPT tendons when calculating the shear capacity of the concrete at ambient. Given that tendon rupture during fire, before the prescribed fire rating is achieved, is a credible concern in UPT structures, slab pre-compression may be lost during fire. Thus, there is a risk that the shear capacity of the already shear-critical structure may be compromised; this has potentially serious consequences for punching shear failure and progressive collapse during fire.
- **Inadequacies of standard furnace tests** – As already noted, existing requirements for fire safe design of UPT beams and slabs are based on a relatively small number of standard fire tests on isolated elements. It is widely recognized that standard fire exposures used in furnace tests [e.g. 35,36] are unrealistic, particularly for large compartments or open floor plates as found in many modern buildings [37, 38]. It is generally argued however that standard fires are conservative representations of credible worst-case fires for most types of construction. This rationale cannot be applied to UPT beams or slabs because UPT tendons are by definition continuous over multiple bays, and localized or travelling fires are more, rather than less, likely to result in premature tendon rupture [27].
- **Zero bonded mild steel reinforcement** – Despite recommendations to the contrary made by researchers [e.g. 9] many existing codes permit design of UPT slabs with zero mild steel reinforcement in sagging moment regions (depending on the in-service concrete stress levels) [23, 33, 34]. Thus, premature tendon rupture during fire could result in total loss of tensile reinforcement which would preclude engagement of tensile membrane action that may be needed to prevent collapse (as discussed previously).

The above discussion points to numerous concerns and inadequacies of available knowledge with respect to the fire performance of UPT beams and slabs in real buildings, all of which are worthy of investigation. To further highlight the apparent disconnect between existing knowledge, available guidance, and industry practice the following sections summarize and critically appraise available data from the 27 fire tests on UPT structural members that are currently available in the literature. Table 1 provides a detailed summary of the assembled test data.

3. LEGACY FURNACE TESTS (Before 1970)

Current fire design guidance for UPT beams and slabs is based predominantly on standard fire endurance tests performed prior to 1970. The Eurocode 2 [23] requirements for fire-safe design appear to have also been influenced by a series of furnace tests performed in 1983 [9]. While many dozens of fire tests on reinforced and prestressed concrete slabs and beams had been performed prior to 1970, only six tests had been reported on flexural elements incorporating

UPT reinforcement. None of these tests can be considered as representative of current construction materials or techniques [39].

3.1 Fire Prevention Research Institute (1958-1959)

Interestingly, the earliest tests ever conducted on UPT elements were among the most realistic yet undertaken. In 1958, the Fire Prevention Research Institute (FPRI), California, conducted the first openly reported fire test on a UPT assembly, which was a single standard furnace test of a two-way UPT beam and slab panel [4]. Figure 1a shows details of the tested assembly, which consisted of a 150 mm deep siliceous aggregate concrete slab spanning 3180 mm between two 4770 mm long UPT beams with different cross-sections. The beams were prestressed longitudinally with draped UPT cables. The slab was prestressed with draped UPT cables in the spanning direction and with straight cables at the slab's mid-depth in the orthogonal direction. The minimum clear concrete cover to the UPT cables, each of which consisted of four 6.3 mm diameter cold drawn, stress relieved wires with an ultimate tensile strength of 1760 MPa, prestressed to 1030 MPa, was 51 mm in the beams and 38 mm in the slab. There was no mild steel reinforcement within the tested spans. The assembly was pin-supported under the beams at its corners and was subjected to a uniformly distributed superimposed load corresponding to the full design live load. The assembly was exposed to the ASTM E119 [35] fire under a "restrained" condition in both spanning directions. Restraint was accomplished by fixing the test specimen within a steel restraining frame "to restrain it against thermal deformations and to simulate the conditions in a building." The gap between the restraining frame and the tested assembly was filled with grout prior to testing; this very good practice appears to have been largely abandoned in many contemporary structural fire testing laboratories. However, neither the rotational nor lateral stiffness of the restraining frame are stated. The relative humidity (RH) in the slab at the time of testing was 62% at a depth of 38 mm (≈1.9% moisture by mass) and the concrete compressive strength was 41 MPa. It is unknown whether the slab was preconditioned, although the relatively low moisture content suggests that this might have been the case.

Heat transmission failure criteria were exceeded at 231 minutes. No structural end-point was reached during the test, which lasted a total of 264 minutes. The maximum recorded tendon temperature was 506°C in the draped slab cables. No cable ruptures were observed. The beams experienced spalling, typically to a depth not greater than 25 mm but in some areas up to 64 mm deep, beginning at 14 minutes of fire and continuing for about one hour. The slab's soffit did not experience any spalling, although splitting cracks with a width of less than 1 mm were observed along the tendons in several locations. The unexposed surface developed large cracks as wide as 6 mm adjacent to the beams due to thermal/flexural deformations.

In 1959, FPRI conducted a second fire test of a similar two-way UPT slab panel (without beams). The original testing report is not readily accessible [5], although general descriptions of the test are available elsewhere [40-42]. Figure 1b shows the tested assembly, which consisted of a 152 mm deep siliceous aggregate concrete flat slab panel spanning 3910 mm by 4270 mm and supported on pins at its corners. The slab was prestressed with draped UPT cables in both directions. The minimum clear concrete cover to the UPT cables, each of which consisted of six 6.3 mm diameter wires, was 38 mm in the larger spanning direction at midspan. The tendons were stressed to 990 MPa. No mild steel reinforcement was provided within the spans. The slab was subjected to a uniformly distributed superimposed load corresponding to 1.0 times the design live load and exposed to the E119 [35] fire, again using laterally-restrained conditions in both spanning directions. The RH in the slab at the time of testing was 76% (≈2.3% by mass) at an unknown depth, and the concrete compressive strength was 30 MPa.

The test was halted at 190 minutes before any failures were observed, presumably because a three hour fire rating had been reached. The recorded temperatures suggest that a heat transmission failure would have occurred about five minutes after the test was stopped, if the heating had been maintained. The maximum recorded tendon temperature was 563°C. Neither cable ruptures, nor spalling or cracking are noted in the available references, although standard furnace testing reports do not generally require disclosure of such details provided that satisfactory ratings are achieved according to load-bearing, insulation, and integrity criteria.

3.2 Portland Cement Association (1964)

In 1964, as part of a larger study on the fire performance of various types of concrete beams, the Fire Research Laboratory of the Portland Cement Association (PCA), Illinois, conducted two standard furnace tests of UPT beams with T-shaped cross sections [8, 41, 42]. The beams' cross-sections are shown in Figures 1c and 1d; they were essentially rectangular, with a depth of 635 mm, a web width of 356 mm, and small flanges of 150 mm × 100 mm. One beam (Beam 80) was prestressed with four draped UPT high strength alloy bars, each with a diameter of 25 mm and an ultimate strength of 1168 MPa, prestressed to 770 MPa. The other beam (Beam 76) was prestressed with four draped UPT cables, each consisting of fourteen 6.4 mm diameter wires with an ultimate strength of 1760 MPa, prestressed to 1168 MPa. In both cases the minimum clear cover at midspan was 64 mm on both the soffit and the sides.

It is noteworthy that while details of the mild steel reinforcement are not shown in the summaries of these tests published in PCI's Post-Tensioning Manual [41], the original PCA research report [8] clearly shows that both of these beams included a light steel mesh (welded wire fabric of orthogonal 2 mm diameter wires, 51 mm on centre in both directions) placed at a depth of 25 mm (mid-depth of the concrete cover) over the entire fire-exposed perimeter of the beams. This is significant, since the presence of this mesh may have mitigated cover spalling [43] and would have arrested it if it occurred. On the basis of similar fire endurances achieved for two pair of beams which were identical but for the fact that two had mesh in the cover and the other two did not, the original PCA test report [8] states that, "the use of mesh within concrete cover is unnecessary"; this statement appears to have been embraced in the years since these tests. It is also significant that the specimens in this study were preconditioned in a humidity-controlled environment at 30-40% relative humidity for at least two years prior to testing. The RH in Beams 80 and 76 at unknown depths at the time of testing were 75% (≈2.3% by mass) and 72% (≈2.2% by mass), and their concrete compressive strengths were 39 MPa and 41 MPa, respectively. Both beams were of normal weight concrete fabricated using carbonate aggregates, which are known to be less prone to spalling than siliceous aggregates [44].

The beams were simply-supported over a span of 12200 mm; much larger than is typical for a standard furnace test, and were subjected to uniformly distributed superimposed loads sufficient to generate load ratios (ratios of total applied midspan moment to nominal midspan moment capacity) of approximately 0.52, during exposure to the E119 [35] fire. Restraint was prevented using hinged-roller supports.

Both beams reached structural end points during testing, Beam 80 at 302 minutes and Beam 76 at 184 minutes. In both cases failure was evidenced by gradually accelerating midspan deflections under sustained load. No spalling was observed for Beam 80, whereas Beam 76 experienced widespread spalling, to the depth of the welded wire fabric, beginning after 10 minutes. This was a key factor contributing to the comparatively low fire resistance of Beam 76. The increased spalling for Beam 76 was despite it having only 11% higher total prestress than Beam 80. Both beams had identical applied loading. Beam 80 developed a number of longitudinal hairline cracks along the tendons during the test, but these apparently remained small. The prestressing steel in Beam 80 reached maximum recorded temperatures of 277°C, 368°C, 429°C, and 503°C after 2, 3, 4, and 5 hours of exposure, respectively. The maximum prestressing temperature recorded at failure was about 541°C in Beam 80 after 5 hours of exposure. Beam 76 developed two "significant flexural cracks" in the midspan region at about 150 minutes, which likely contributed, along with widespread cover spalling noted above, to failure. The prestressing steel in Beam 76 reached maximum recorded temperatures of 316°C and 416°C after 2 and 3 hours of exposure, respectively. The maximum prestressing temperature recorded prior to failure was about 427°C in Beam 76 after 3 hours of exposure. No tendon ruptures were noted.

A key conclusion stated in the PCA report on these tests [8], which is reiterated in a later summary paper by Gustaferro [42] and which appears to have been embraced in the years since these tests, is that beams with UPT reinforcement have about the same fire endurance as their counterparts with bonded prestressed reinforcement. While this conclusion is unsurprising for isolated, simply supported, non-continuous beams with uniform heating, the wider applicability of this conclusion for real UPT flat plate structures is doubtful. Indeed, recent furnace testing by Bailey and Ellobody [3] (described below) showed that bonded PT slabs were capable of achieving their designed target fire resistance whereas otherwise identical UPT slabs had fire resistances that were lower than expected.

3.3 Underwriters' Laboratories (1965-1967)

In 1965, as part of a larger study involving various types of prestressed concrete members, the Underwriters' Laboratories (UL), Illinois, conducted a standard fire test of a UPT inverted T-beam [6, 41, 42]. Few details of this test have been reported in the open literature. The beam's cross-sectional details are shown in Figure 1e; it was fabricated from normal weight carbonate aggregate concrete of unknown strength, moisture content, and pre-test conditioning regime. It incorporated a single, draped unbonded tendon with a minimum clear concrete cover of 48 mm at midspan. No mild steel reinforcement appears to have been provided. The beam was tested under exposure to the E119 [35] fire over a span of 5310 mm with a superimposed load corresponding to 1.0 times the design live load. The beam ends were restrained by grout fill within a restraining frame, although details of the actual restraining system and stiffness are unavailable. Neither cracking nor spalling were noted during the test. The tendon temperature at midspan was 338°C at 180 minutes and 377°C at 240 minutes. The test was stopped at 255 minutes before a structural end point was reached.

In 1967, UL conducted a single standard fire test of a two-way UPT slab panel cast from lightweight concrete [7, 41, 42]. Figure 1f shows the tested assembly, which consisted of a 152 mm deep, expanded shale aggregate concrete slab panel spanning 4270 mm by 5380 mm supported on steel bearing plates at its corners. The slab was prestressed with draped UPT cables in both directions. The minimum clear concrete cover to the UPT cables, each of which consisted of five 6.3 mm diameter cold drawn, high tension stress-relieved wires, each having a minimum guaranteed ultimate tensile strength of 1655 MPa, was 25 mm in the larger spanning direction. The tendons were prestressed to achieve an in-service design stress of 992 MPa, once the superimposed load had been applied for the fire test. No mild steel reinforcement was provided within the spans. The assembly was subjected to a uniformly distributed superimposed load of 1.0 times the design live load, and was tested under an E119 [35] fire using a "fully restrained" condition in both directions (using similar methods as in the FPRI tests described previously). The concrete compressive strength at the time of testing is not known, although it was only 29 MPa at 28 days.

It is crucial to point out that this slab was pre-conditioned for approximately 7.5 months at an elevated temperature of 49°C and a relative humidity of approximately 20% before testing. This is not representative of a typical in-service environment. In combination with the use of expanded shale aggregates, the preconditioning makes the likelihood of cover spalling very low for this slab. The RH of the slab at the time of testing was extremely low, ranging between 42% and 47% (≈1.5% moisture by mass) at the deepest sections.

No spalling or cracking was observed on the exposed face during the fire, which lasted 225 minutes before it was stopped without a structural end point being reached. The maximum tendon temperatures recorded after 60, 120, and 180 minutes were 378°C, 516°C and 688°C, respectively. The maximum tendon temperature at the end of the test was very high, at 704°C after 3 hours. No cable ruptures are noted in the available references. At the end of fire test the slab was subjected to a hose stream for four minutes, and after cooling (overnight) to ambient temperature the slab was loaded to twice its ambient design live load "without signs of distress."

4. MODERN FURNACE TESTS (After 1980)

No additional peer-reviewed research on the fire behaviour of UPT structures was published until the early 1980s. By 1983, researchers apparently began to question the wisdom of relying on results from the pre-1970 tests to guarantee adequate fire resistance for UPT buildings. In particular, a 1983 study in Holland [9] (described below) recognized that both rotational restraint and tendon continuity into unheated bays could impact a UPT structure's response to fire. More recently, furnace tests have been performed in China [10] and the UK [11] although in both cases researchers have reverted to performing single element tests of simply-supported, one-way spanning members with small span-to-depth ratios and without accounting for continuity or rotational restraint.

4.1 Van Herberghen and Van Damme (1983)

Van Herberghen and Van Damme [9] report on an extensive series of non-standard structural fire tests on eight one-way continuous but unrestrained UPT slab strips. These tests are particularly interesting, since they appear to be the only fire tests to ever simulate rotational restraint through continuity at internal supports – however neglecting axial restraint, which, although unrealistic, is likely to be conservative. These are also the only available tests to rationally consider the possible influence of important parameters noted previously including: cover spalling, load ratio, and the presence and amount of "secondary" bonded mild steel reinforcement. Furthermore, they are the only tests which have ever included a UPT cantilever span exposed to fire. This is particularly interesting in light of the case study by Lukkunaprasit [14] (summarized below) in which a UPT cantilever span in a real UPT building collapsed during a real fire.

Details of a typical specimen tested by Van Herberghen and Van Damme [9] are shown in Figure 1g. Also shown is a schematic of the test setup, support and loading conditions, and fire exposure. The eight one-way slab strips were 180 mm thick by 1900 mm wide, and approximately 9000 mm in total length. Longitudinally they all had draped 12.7 mm nominal diameter longitudinal seven-wire prestressing tendons, with 1150 MPa initial prestress levels, and all were continuous over two internal supports with a central span of about 6000 mm and cantilever spans at each end. Only the central span and one cantilever were exposed to fire. Several parameters were varied amongst the specimens:

- **Concrete type**: Few details of the concrete mixes used in the fabrication of the slabs are available. Neither compressive strength nor moisture content is reported. Two of the slabs were fabricated with limestone aggregate concrete, whereas the other six slabs used "gravel" aggregate (assumed to mean siliceous aggregate, although this is not explicitly stated). The concrete age at the time of testing was between 98 and 273 days, making it difficult to draw meaningful correlations between concrete age, moisture content, and propensity for spalling during fire.
- **Transverse prestressing**: One of the slabs had transverse UPT tendons spaced at 950 mm along its length. The applied prestress is not given, although it seems likely that it was also 1150 MPa.
- **Casting technique**: Two of the slabs were cast integrally on top of precast reinforced concrete forming planks with a thickness of 50 mm to 60 mm and differing mild steel reinforcement details. These two slabs cannot be considered as typical UPT elements and are ignored in subsequent discussion.
- **Concrete cover and tendon profile**: The remaining six slabs (not cast integrally with precast planks) had minimum clear covers to the prestressed reinforcement, at the middle of the central span, varying between 20 mm and 40 mm. The cover at the support (measured from the top surface of the slab) varied between 20 mm and 32.5 mm. The source publication [9] should be consulted for complete details of cover combinations between slabs.
- **Passive (bonded) mild steel reinforcement**: The amount, location, and orientation of mild steel reinforcement in the slabs varied widely, as shown in Table 1. Two of the slabs had no passive reinforcement whatsoever, one had mild steel reinforcement in both directions only at the bottom face, and the remaining three had differing amounts of mild steel reinforcement at the top and bottom faces in both directions. The cover to the mild steel reinforcement ranged between 20 and 27.5 mm.

The slabs were loaded during testing using four hydraulic jacks (as shown in Figure 1g) to produce "a condition of zero rotation at the supports." Exactly how and why this loading condition was chosen is unclear. As a consequence of this rather unusual support condition, the applied load (and hence the bending moments at the critical sections) varied throughout the tests – it seems unlikely that this was any more representative of full-structure response in a real fire than a restrained standard fire test on an isolated structural element with constant vertical load. The initial load was chosen to simulate a comparatively high uniformly distributed superimposed live load of 4.9 kPa. In the paper describing these tests, Van Herberghen and Van Damme [9] comment that "the importance of the initial bending moments was of minor influence on the ultimate fire resistance." It is not clear exactly what is meant by this, but it appears from the data presented that the evolution of bending moments during the fire exposure was far more important in governing the slabs' collapse than the initial loading condition. This is of considerable interest, since the initial load ratio is universally assumed to be of central importance to the structural fire resistance of a flexural assembly in a standard furnace test – to the extent that larger fire endurances are assigned to assemblies with lower load ratios for some types of construction [42, 45]. This confirms the increasingly widespread notion that full-structure interactions in fire are likely to be more significant than the capacities of isolated members as demonstrated through standard furnace tests.

The results of these tests shed light on a number of concerns specific to UPT construction which have been observed in real fires in UPT buildings. Cover spalling was observed in all of Van Herberghen and Van Damme's tests. Slabs without mild steel reinforcement experienced widespread spalling, which is noted to have contributed to collapse after 40 minutes in one case (with 30 mm clear cover and a 60 minute prescriptive fire resistance rating based on Eurocode [23] cover requirements for a simply supported slab) and impending collapse after 59 minutes in another (with 20 mm clear cover and a 30 minute fire rating based on Eurocode requirements for simply supported slabs). Slabs with mild steel reinforcement also experienced spalling, but in these cases it was restricted to the depth of the mild steel reinforcement, confirming that a layer of mild steel reinforcement can, in some cases, arrest cover spalling and provide a measure of additional protection to the UPT tendons. In all cases spalling initiated in the most compressed region of the soffit close to the supports, confirming that pre-compression increases propensity for spalling in fire. This was particularly evident for the single slab which had both transverse and longitudinal prestressing (hence biaxial pre-compressive stresses) – this slab suffered severe spalling resulting in premature tendon rupture and impending structural failure in 56 minutes (despite a one hour prescriptive rating [23]).

Slabs without mild steel reinforcement experienced transverse cracking (both over the supports and at midspan) and longitudinal splitting cracking along the tendons. Van Herberghen and Van Damme noted that this was due to a combination of pre-compressive stress combined with lateral tensile stresses generated due to thermal effects during the fire; they state that transverse mild steel reinforcement is essential to address this. Slabs with increasing amounts of transverse mild steel reinforcement at the top and bottom faces displayed progressively fewer longitudinal splitting cracks. Subsequent researchers [3, 10] have also noted the importance of splitting cracks for the fire endurance of UPT structures, yet design guidelines continue to essentially ignore this issue.

Very significantly for the current discussion, premature tendon rupture (of two to four of the longitudinal tendons) was observed in all eight tests. The first tendon ruptures preceded overall structural collapse by between 5 and 25 minutes. Van Herberghen and Van Damme stated that these ruptures were due to localized heating of the strands resulting from a combination of cover spalling, splitting along the tendons, and transverse cracking at midspan. It is particularly interesting that premature tendon rupture was observed in this study, since the tendons in these experiments were long in comparison with all previous testing, they were subjected to fire only over a portion of their length, and they were locally heated as a consequence of cracking and spalling (as would occur in a real building fire). The authors of the current paper have previously shown [27], that the longer a UPT tendon and the shorter the length over which it is heated, the more likely is premature tendon rupture in fire. This is confirmed by Van Herberghen and Van Damme's tests and again shows that furnace tests on short UPT tendon lengths over single spans are, by definition, incapable of rationally simulating the structural conditions in a real UPT building; there is little doubt that furnace tests are unconservative for evaluating the risk of premature tendon rupture during fire.

Based on their results, Van Herberghen and Van Damme [9] suggested revised minimum concrete cover depths, minimum amounts of mild steel reinforcement in UPT flat plate slabs, and minimum amounts of mild steel reinforcement in support regions to prevent excessive flexural cracking. Current Eurocode [23] requirements appear to have been influenced by these recommendations (after adjusting from clear cover to axis distance). However, other code writing bodies [e.g. 21], have largely ignored the recommendations emerging from this study despite additional evidence from real fires (see below) that they are warranted.

4.2 Zheng and Hou (2006)

Zheng and Hou [10] report on nine standard furnace tests on small-scale (model), one-way spanning UPT slab strips prestressed with steel wires. Figure 1h provides details of a typical tested assembly. All specimens were fabricated from carbonate aggregate concrete with a width of 600 mm and spanning 3300 mm between simple supports. The prestressing consisted of between two and five prestressing wires, each with a diameter of 5 mm and an ultimate tensile strength of 1722 MPa. The wires were prestressed to between 655 MPa and 1022 MPa prior to testing. A clear concrete cover of 15 mm, 25 mm, or 30 mm to the prestressed reinforcement was provided for the 80, 90, and 95 mm thick slabs, respectively. Varying amounts of mild steel longitudinal reinforcement were provided within each of the slabs; the mild steel longitudinal reinforcement ratio varied between 0.20% and 0.45%. All slabs had mild steel transverse reinforcement with the same clear cover as the prestressed reinforcement at midspan, and with a reinforcement ratio of about 0.17% to 0.20% depending on their thickness.

The slabs were subjected to uniformly distributed superimposed loads using 20 kg dead weights and were tested under exposure to the ISO 834 [36] standard fire in an unrestrained condition. The load ratio varied between 0.41 and 0.72. The moisture content in the slabs at the time of testing varied between 1.8% and 4.0% by mass, although it is not clear how or where this was determined. The concrete compressive strength varied between 22 MPa and 57 MPa.

Unfortunately, all of Zheng and Hou's tests were halted before reaching structural failure, using a slope limit failure criterion. The resulting data are therefore not particularly useful for the current discussion. It is significant, however, that longitudinal cracking along the tendons was observed in at least two of the tests, and that spalling was observed in at least five. Spalling appears to have been most pronounced in the most compressed regions of the soffit (close to the supports), and to have been most severe for slabs with high initial total prestress levels, regardless of moisture content or concrete strength. No tendon ruptures are noted.

4.3 Bailey and Ellobody (2009)

Bailey and Ellobody [3, 11, 46] report the results of four large-scale furnace tests on one-way spanning UPT concrete floor slabs in which the lateral (longitudinal) restraint condition ("free" or "restrained") and the type of aggregate (limestone or Thames gravel) were varied. Different aggregates were apparently used to investigate the effect of different amounts of thermal expansion on the structural behaviour, and perhaps also to study spalling. Details of the specimens are given in Figure 1i. The slabs were 1600 mm wide and 160 mm deep, and spanned 4000 mm between simple supports. Each slab had three longitudinal seven-wire prestressing tendons, each with a nominal cross-sectional area of 150 mm^2 and an ultimate tensile strength of 1846 MPa. The tendons were prestressed to about 1120 MPa after losses. The minimum concrete cover to the centroid of the draped prestressing tendons was 42 mm at midspan. This corresponds to a clear cover of approximately 34 mm. No mild steel reinforcement was provided within the tested spans.

The concrete strength at the time of testing varied between 40 MPa and 48 MPa, and the moisture content varied between 1.7% and 2.5% by mass (again the method and location of moisture measurement are not known). The applied loading was selected as 50% of the capacity of an identical slab tested at ambient temperature, resulting in a fire test load ratio of about 0.65. The slabs were exposed to the BS EN 1991-1-2 [47] fire until collapse (first test) or until the test was halted (three subsequent tests).

Two of the four slabs were "restrained" longitudinally by two steel beams, although full details of the restraining mechanism are not available. Bailey and Ellobody [11] state that the restrained slabs were initially free to expand longitudinally during fire testing, for a total distance of 2 mm, "until (they) came in full contact with the restraining beams and until the bolts fixing the restraining beams became intact with the edges of the holes". The stiffness of the restraining frame, once engaged, is not known.

None of the slabs experienced major spalling during fire testing, although some localized spalling is visible in photos published in [11]. The unrestrained slab with limestone aggregate developed longitudinal splitting cracks on its unexposed surface after 20 minutes of fire exposure when the maximum recorded tendon temperature was only 108°C. This slab collapsed into the furnace after 178 minutes. The collapse was apparently due to tendon rupture at the location of a major transverse flexural crack near midspan. The maximum recorded tendon temperatures were about 200°C and 400°C after 60 and 120 minutes, respectively, and the maximum tendon temperature prior to failure was about 492°C. The restrained limestone aggregate slab performed similarly, with the exception that longitudinal cracking was observed on the unexposed face after only 15 minutes when the tendon temperature was 95°C. This test was halted after 83 minutes, when the tendon temperature reached 350°C, to avoid damaging the furnace.

The Thames gravel aggregate slabs performed similarly to the limestone aggregate slabs, with the exception that they experienced greater thermal expansion on heating. Both slabs experienced longitudinal splitting cracking on the unexposed face, at 18 minutes and a tendon temperature of 119°C or at 21 minutes and a tendon temperature of 115°C for the free and restrained cases, respectively. Both tests were halted when the tendon temperatures reached 350°C, which was at 72 minutes for the free case and 89 minutes in the restrained case.

Bailey and Ellobody [11] note evidence of arching (compressive membrane) action and horizontal shear cracking during their restrained tests. They reiterate Van Herberghen and Van Damme's [9] recommendation for transverse mild steel reinforcement to prevent longitudinal splitting cracking at the location of the tendons, which they view as being the "critical failure mode" for their UPT slabs in fire.

5. REAL FIRES IN UPT BUILDINGS

Case studies of real fires in UPT concrete buildings are rare. A few papers assessing bonded prestressed concrete structures are available [40, 48], although these are not particularly useful for evaluating the fire performance of UPT buildings for the reasons noted previously. The following is a description of six fires in real UPT buildings; these confirm that the behaviour of real UPT buildings in real fires merits additional study.

5.1 Los Angeles, California (1965)

The earliest report of a fire in a real UPT building is by Troxell [13], and describes a fire that occurred in a three year old UPT school building in Los Angeles, California. The two storey building had UPT flat plate lightweight aggregate concrete slabs with UPT tendons running longitudinally and transversely. The prestressing consisted of 6.4 mm diameter stress relieved prestressing wires grouped into cables of 4 to 14 wires. The slab thickness was between 240 mm and 250 mm and the minimum concrete cover was 25 mm. Details of the amount and location of any mild steel reinforcement in the slabs are not given. The fire lasted about 1.5 hours and was compartmentalized within two rooms of the six room south wing of the building. Both the ceiling and floor slabs experienced spalling. The room where the fire was assumed to have originated experienced spalling over 75% of its 80 m^2 ceiling area to a maximum depth of 44 mm and exposing several of the tendons. The floor experienced an average of 10 mm of spalling located centrally over a 900 mm diameter area. However, no tendon ruptures were observed and the structure did not collapse. The maximum temperature was estimated by noting fusion of copper wire on the ceiling, apparently indicating that the fire temperature may have peaked at about 1070°C. After the fire, tendons were tested in place using lift off tests. Troxell tested six UPT strands in this manner, one unexposed control which was assumed not to have been affected by the fire and five "fire affected" tendons. No obvious prestress relaxation was observed in the fire affected tendons in comparison with the control tendon. It is noteworthy that Troxell [13] states that severe spalling of the ceiling was due to thermal shock from a hose stream and that the tendons were not directly exposed to the peak temperatures of the fire.

5.2 Bangkok, Thailand (1987)
Partial collapse of a UPT slab exposed to fire for more than five hours occurred in an 18 storey building in Bangkok, Thailand in 1987 [14]. Each floor of the building had 4000 m^2 of two-way prestressed UPT flat plate construction, with interior bays of 80 m^2 each and 4 m long cantilevers located at the end of each floor. The concrete slab was 200 mm thick and the cover to the prestressing tendons was 20 to 25mm. Minimum mild steel reinforcement was provided according to the ACI [49] provisions at that time.

The fire started on the third floor and spread upward to the fifth floor. Widespread spalling occurred and exposed some of the tendons directly to the fire. Eventually, some of the tendons are thought to have ruptured and the cantilevers at the end of the fourth floor collapsed. This resulted in collapse of two supporting columns and caused progressive collapse of several interior bays. It was estimated that between 10 to 20 percent of the tendons in the floor plate had ruptured during the fire.

Significantly, while bays in the fire exposed region collapsed, adjacent bays did not fail even though the tendons were unbonded and continuous into the adjacent spans. Lukkunaprasit [14] believes that tension membrane action of the slabs occurred with the tendons anchored at the edges of the collapsed bays by “kinks” over the column lines. Large vertical displacements in the slabs are thought to have allowed a smaller tendon force to carry the slabs’ weight and the imposed loads by tensile membrane action, as is widely recognized [50] to occur in steel-concrete composite slabs in fire. On the basis of this fire, Lukkunaprasit suggested that engineers should supply unstressed bonded prestressing steel at the mid depth of UPT slabs so that the additional reinforcing can act as tensile membrane reinforcement and prevent collapse during a fire. Such measures have not, however, been adopted into modern UPT design codes, nor has any subsequent research seriously considered this idea.

5.3 Santa Ana, California (1988)
Tendon rupture during fire is highlighted in a report of post-fire controlled demolition of a UPT two-way slab structure in Santa Ana, California in 1988 [15]. In this case a four storey timber frame built on top of a flat plate podium UPT slab over a parking facility caught fire after completion of the slab but during construction of the timber frame. During the fire the timber frame collapsed and the podium slab was exposed to fire from above. The slab was 200 mm to 230 mm thick and 160 m × 120 m in plan. Spalling occurred at midspan at the top of the slab (i.e. in the compression zone) in locations where no passive reinforcement was present. The maximum depth of spalling was 76mm. Three percent of the prestressing tendons ruptured during the fire, and based on inspection by engineers the slab was demolished. The report [15] does not describe the severity of the fire, nor does it state the concrete cover to the prestressed reinforcement from above. However, this clearly illustrates the potential for tendon rupture during a real fire.

5.4 Portland, Oregon (1999)
A fire in a similar structure to that described in Section 5.3 occurred in Portland, Oregon [16], in 1999, when a timber frame that was built on top of a UPT flat plate concrete podium caught fire. In this case the UPT slab was 330 mm thick and had a minimum concrete cover of 48 mm. Again the timber frame caught fire and collapsed and the slab was exposed to fire from above. Spalling was limited to a depth of 12 mm to 19 mm and no tendon ruptures were observed. Tendons and mild steel reinforcement were manually exposed in six different locations after the fire to visually assess damage to the reinforcement. In one location the plastic sheathing of the UPT tendons exhibited signs of melting, although the grease appeared to be intact. No attempts were made to assess the post-fire prestress levels in the tendons. The slab was subsequently repaired and used during rebuilding of the timber frame structure.

5.5 Florida, USA (2000)
A large uncontrolled fire occurred during construction of a 12 storey UPT condominium building in Key Biscayne, Florida in 2000 [17]. The building contained UPT tendons that were continuous across seven interior bays. The construction of the building had progressed to the 12th floor when a localized fire broke out on the second floor in one interior bay next to an internal shear wall. The fire spread to an adjacent bay where a pour-strip was located (a pour strip is an area of the slab where tendons are anchored during construction and which is left void until the building is nearing completion). The fire then spread vertically, ultimately causing visible fire damage up to the seventh floor across two interior bays. The engineers who examined the structure after the fire stated that “heat caused the tendons at the pour strips to release tension.” They further noted that release of tension “triggered progressive failure of the post-tensioned slab well beyond the zone of visible damage.” The result of this was that “almost half the slabs on levels three to six, and possibly seven, lost integrity” [17]. This represents loss of structural integrity across a total of 48 bays of the structure. The risk of progressive collapse was sufficiently high that no contractor could be found who would re-shore the floor slabs after the fire, and the entire building was demolished. This case study illustrates the potential consequences of localized tendon heating during fire and prestress loss across multiple bays.

Brannigan and Corbett [28] have also presented a case study of a UPT building which was subjected to a formwork fire during construction and which led to collapse by a progressive mechanism, although this is less worrying given the unusual nature of the fire.

5.6 Tel Aviv, Israel (2000)

A short case study presented by Stern [18] in 2002 describes a UPT slab with spans up to 16 m exposed to a severe fire in Tel Aviv, Israel. Stern indicates that spalling of the concrete cover to the tendons occurred over a 300 m^2 area. Some of the tendons apparently ruptured and the mild steel was exposed during the fire. In this case the UPT slab was reinstated by re-connecting and re-tensioning the ruptured tendons. The wisdom of this approach is questionable unless considerable testing was performed to check for deterioration in post-heating residual mechanical properties of the prestressing steel [51].

6. DISCUSSION

The above sections raise a number of issues relevant to the fire-safe design of UPT structures. UPT members appear to perform reasonably well in standard furnace tests, and in these scenarios they are typically able to comply with prescribed fire resistance ratings up to or exceeding three hours. However, given the large number of UPT structures in service and the relatively small number of real fires which have been reported (only six fires globally), this should not be construed as clear evidence of the fire safety of UPT structures. The following are considered the key issues identified in the available literature.

6.1 Summary of Available Test Data

- No realistic tests have ever been performed on multiple span continuous UPT (or bonded PT) structures incorporating both rotational and axial restraint.
- Most of the tests which have been used to demonstrate fire safety of UPT slabs were performed prior to 1970 using construction materials which are now outdated and specimens which in some cases were pre-conditioned prior to testing (thus explicitly reducing the likelihood of cover spalling).
- Spalling was observed in at least 16 of the 27 available tests (59%). Useful correlations cannot be drawn between propensity for spalling and relevant specimen parameters such as concrete strength, pre-compression levels, aggregate type, amount and location of mild steel reinforcement, load ratio, concrete moisture content, etc. However, limiting the moisture content to less than 3.0% by mass, as is suggested in the Eurocodes [23], is clearly not sufficient to prevent spalling during fire. The presence of non-prestressed mild steel reinforcement within the concrete cover does not prevent spalling, although in some cases it can arrest cover spalling to the depth of the mild steel reinforcement and thus provide some protection to the UPT tendons.
- In unrestrained standard furnace tests with realistic load ratios, structural collapse is imminent when the tendon temperature exceeds about 400-450°C. In restrained tests, even without mild steel reinforcement, tendon temperatures as high as 700°C have been observed without collapse. Current prescriptive guidance is therefore defensible for predicting standard furnace tests; but not necessarily for predicting the response of real UPT buildings.
- Longitudinal splitting cracking along the tendons was observed in at least 16 of the 27 tests (59%). Current design guidance does not address this issue even though researchers have suggested that this is the governing mode of failure for UPT structures in fire (as early as 1983).
- Transverse (flexural) cracks, in some cases as wide as 6 mm, were observed in at least 18 of the 27 tests (67%). Localized heating of tendons at these cracks (even in the absence of cover spalling) could induce localized heating and premature tendon rupture during fire. UPT members develop fewer, wider cracks than equivalent bonded PT members, both under ambient conditions and, more importantly, during fire.
- Premature tendon rupture during heating was observed in at least 9 of the 27 tests (33%), and was particularly evident in tests with multiple spans and localized heating. Tendon rupture is therefore more, rather than less, likely in a real UPT building than in a furnace test of an isolated structural element or assembly.

6.2 Summary of Case Studies

- Some degree of spalling occurred in all cases. It therefore seems that without preventative measures taken to avoid spalling localized spalling should be considered as likely, rather than unlikely, in all UPT buildings exposed to severe fires. It is therefore difficult to justify design for fire safety of UPT systems purely on the basis of minimum concrete cover to the tendons, with an inherent assumption that the concrete cover will remain in place during fire.
- Tendon rupture or release of prestress occurred in two thirds of the case studies. This confirms that tendon rupture (or prestress loss) is likely to occur in a UPT building in a real fire. Premature tendon rupture during fire has led to both partial and progressive failure of real UPT buildings.
- UPT structures exposed to real fires are almost certain to experience non-uniform and localized heating, which will expose the tendons to heating over only a portion of their total length.

7. CONCLUSIONS

While it is widely acknowledged that standard furnace testing is unrealistic for most real structures [20], whether this raises serious concerns for structural fire safety remains under debate. However, it has been shown herein that an awareness of the lack of realism which is inherent in standard furnace testing is particularly important for the specific case of UPT concrete beams and slabs. Standard furnace testing is fundamentally incapable of rationally simulating several important (and interrelated) behaviours that can be expected (and that have been directly observed) in real UPT buildings during real fires and that can lead to premature tendon rupture and progressive failure. Localized heating of UPT tendons may occur due to a combination of:

1. single bay, localized, or travelling fires in multi-bay structures;
2. draped tendons with variable concrete cover;
3. spalling of the concrete cover; and
4. longitudinal and/or transverse cracking.

Localized heating of a UPT tendon is likely to lead rapidly to tendon rupture, as has been seen in many tests and real fires. The risk and consequences of localized heating can be mitigated through the use of bonded mild steel reinforcement, although this issue is still neglected in some design standards. Bonded mild steel reinforcement can limit the depth of spalling, promote a finer and more evenly distributed cracking pattern, and permit alternative load carrying mechanisms. Research is needed to define the appropriate minimum amount and placement of bonded reinforcement to provide the required safety against collapse during fire. The consequences of localized heating for the capacity of a UPT floor plate in fire remain largely unknown and require additional investigation to ensure that unexpected failures of UPT structures do not occur, both during the code-prescribed fire resistance time but also during the cooling and post-fire periods.

Because full-scale fire tests on actual or model UPT buildings, however badly needed, are unlikely to occur in the foreseeable future, research is currently restricted largely to using computational analysis tools [52, 53] to study their response to fire. In general, these tools have not been validated against real fires in UPT concrete structures and their ability to accurately model various aspects of UPT concrete beams and slabs at elevated temperature is doubtful. A detailed experimental and computational examination of the potential consequences of localized heating on UPT tendons is therefore needed with a view to eventually developing the ability to defensibly model real UPT buildings in real fires.

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Table 1: Selected details of furnace test specimens available from the literature (Part 1)

#	Source	Year[a]	General Description	Concrete precompression due to prestress (Longitudinal, MPa/ Transverse, MPa)[b]	Restraint conditions (lateral/ rotational)[c]	Specimen pre-conditioning[d]	Bonded steel reinf.[e] (%)	f_c' at testing (MPa)
1	[4]	1958	2-way beam-slab assembly	0.94/1.13	Y/P	--	0/0/0/0	41
2	[5]	1959	2-way slab panel	1.64/1.61	Y/P	--	0/0/0/0	30
3	[8]	1964	T-beam	6.85/0	N/N	Y	Mesh in cover	39
4				6.08/0				41
5	[6]	1965	Inverted T-beam	3.29/0	Y/P	--	0/0/0/0	--
6	[7]	1967	2-way slab panel	2.39/2.07	Y/P	Y	0/0/0/0	29[f]
7	[9]	1983	Continuous 1-way slab strip	0.78/0	N/Y	--	0/0/0/0	--
8				0.78/0			0/0/0/0	
9				0.78/0			0/0/0.12/0.12	
10				0.78/1.65			0.26/0.26/0.1/0.1[h]	
11				0.78/0			0.15/0.15/0.08/0.08	
12				0.78/0			0.2/0.2/0.1/0.1	
13				0.78/0			0.2/0.2/0.1/0.1	
14				0.78/0			0.2/0.2/0.2/0.2	
15	[10]	2006	1-way slab strip	0.77/0	N/N	--	0/0.45/0/0.21	57
16				0.98/0			0/0.31/0/0.21	48
17				2.09/0			0/0.31/0/0.21	57
18				0.88/0			0/0.46/0/0.18	52
19				1.09/0			0/0.31/0/0.18	52
20				1.19/0			0/0.21/0/0.18	52
21				0.7/0			0/0.25/0/0.17	52
22				0.96/0			0/0.29/0/0.17	47
23				1.44/0			0/0.20/0/0.17	23
24	[3, 11]	2008	1-way slab strip	1.97/0	N/N	N	0/0/0/0	48
25				1.97/0	Y/P			41
26				1.97/0	N/N			40
27				1.97/0	Y/P			40

[a] refers to the year the tests were conducted (estimated on the basis of publication date in some cases)
[b] The precompression is the total tensioning force divided by the cross sectional area of the slab normal to that force [26]
[c] Y = yes, N = no, P = partial (assumed by current authors)
[d] "--"means that this information was not disclosed
[e] top longitudinal/top transverse/bottom longitudinal/bottom transverse
[f] 28-day compressive strength (strength at time of testing not known)

Table 1: Selected details of furnace test specimens available from the literature (Part 2)

#	Moisture content at testing	Aggregate type	Load ratio[g]	Max. tendon temp. (°C)	Span/ depth ratio	Longitudinal cracking?	Transverse cracking?	Spalling?	Tendon rupture?	End-point
1	62 % RH	Siliceous	1.0 × LL	506	21	Y	Y	Y	N	Transmission
2	76 % RH	Siliceous	1.0 × LL	563	28	--	--	--	--	None
3	75 % RH	Carbonate	0.516	541	--	Y	N	N	N	Collapse
4	72% RH	Carbonate	0.520	427	--	N	Y	Y	N	Collapse
5	--	Carbonate	1.0 × LL	377	--	N	N	N	N	None
6	44 % RH	Expanded shale	1.0 × LL	704	23	N	Y	N	N	None
7	--	“Gravel”	Variable	--	33	Y	Y	Y	Y	Collapse or imminent collapse
8		“Gravel”								
9		“Gravel”								
10		“Gravel”								
11		Limestone								
12		Limestone								
13		“Gravel”								
14		“Gravel”								
15	4.0 % by wt	Carbonate	0.41	--	41	N	Y	N	N	Slope limit
16	3.5 % by wt		0.54		41	Y	Y	Y		
17	4.0 % by wt		0.70		41	N	Y	Y		
18	2.4 % by wt		0.42		37	N	N	N		
19	3.3 % by wt		0.56		37	Y	N	N		
20	2.4 % by wt		0.68		37	N	N	N		
21	3.3 % by wt		0.42		35	N	Y	Y		
22	3.5 % by wt		0.56		35	N	Y	Y		
23	1.8 % by wt		0.72		35	N	Y	Y		
24	2.5 % by wt	Limestone	0.65	492	23	Y	N	N	Y	Collapse
25	2.2% by wt			350[i]			Y	Y	N	None
26	2.3 % by wt	Thames gravel					N	N	N	None
27	1.7% by wt						N	N	N	None

[g] in some cases the Load Ratio is not given, in which case the loading approach is given instead
[h] this slab was also prestressed transversely
[i] these tests were explicitly stopped when the tendon temperature reached 350°C in order to avoid collapse

Figure 1: Selected details of furnace test specimens available from the literature