Modelling of bonded post-tensioned concrete slabs in fire

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This paper presents a finite element model highlighting the behaviour of bonded post-tensioned one-way spanning concrete slabs in fire conditions. The model was verified against ten fire tests on bonded post-tensioned concrete slabs at ambient and elevated temperatures. The slabs were simply supported and post-tensioned with 15.7 mm nominal diameter seven-wire mono-strand tendons. The mechanical and thermal material non-linearities of the entire slab's components, consisting of the concrete, plastic and galvanised steel ducts, prestressing tendon and the anchorages, have been carefully inserted into the model. The interface between the tendon and surrounding grout was also considered, allowing the bond behaviour to be modelled and the tendon to retain its profile shape during the deformation of the slab. The temperature distributions throughout the slab, together with the slab's development of displacement and stress, as it was heated, were predicted by the model and verified against test data. A parametric study was conducted to investigate the effects on the global structural behaviour owing to the change in the aggregate type, duct type, load ratio, boundary conditions and different fire scenarios. The study has shown that the bonded post-tensioned concrete slabs investigated in this study are capable of achieving the designed 90 min fire resistance. It is also shown that the fire resistance given by BS 8110-2 and BS EN 1992-1-2 are acceptable for the design of bonded post-tensioned one-way spanning concrete slabs under fire conditions.

NOTATION

- CS: cold surface
- e: concrete emissivity
- f: stress
- f_c: compressive stress of concrete
- f_k: proportional limit stress of concrete
- f_m: compressive cube strength of concrete
- f_t: tensile stress of concrete
- G_I: fracture energy
- HS: hot surface
- h: slab height
- LC: long cool fire curve
- LS: limestone
- LT: left tendon
- M: metallic duct
- MS: middle surface
- MT: middle tendon
- PL: plastic duct
- RT: right tendon
- SH: short hot fire curve
- ST: standard fire curve
- T: tendon level
- TG: Thames Gravel
- α_c: convective coefficient
- ε: strain
- ε_u: ultimate strain of concrete in compression
- ε_I: ultimate strain of concrete in tension

1. INTRODUCTION

Post-tensioned floor slabs can be bonded or unbonded. For bonded post-tensioned floor slabs the transfer of the force from the tendon to the concrete is by way of the end anchors, the bond between the tendons and concrete which is maintained by grouting and the curvature of the tendons. For unbonded systems the transfer of force from the tendon to the concrete is only by way of the end anchors and the tendon's curvature. To date, there has been no detailed information regarding the structural behaviour of bonded post-tensioned concrete slabs under fire conditions with only a few fire tests, recently conducted by Bailey and Ellobody1,2 reported on this form of construction. Current design rules specified in BS 8110-23 and BS EN 1992-1-24 provide guidance on the fire resistance of post-tensioned floor slabs, but these have been derived based on tests carried out on pre-tensioned and non-tensioned concrete members. In terms of post-tensioned construction the limited previous fire tests were only conducted on unbonded post-tensioned slabs as reported in references [5–7]. An extensive review of these tests, together with evidence of fires in actual buildings constructed using post-tensioned floor slabs, was provided by Lee and Bailey8 who concluded that further investigation into the behaviour of post-tensioned floor slabs in fire is required. This conclusion, of the need for further investigation, is also shared by the latest Fédération Internationale de la Précontrainte (FIB) design guide on post-tensioned slabs.9 The existing limited test data on unbonded slabs were augmented by Bailey and Ellobody10 where the effect of different aggregate type and restraint conditions was highlighted.

Recently, Bailey and Ellobody1 have conducted ten tests on bonded post-tensioned one-way concrete slabs at ambient and
elevated temperatures. Eight slabs were subjected to a fire under a static load representing a load ratio of 0-6, where the load ratio is defined as the applied load divided by the design capacity. In addition to the fire tests two additional tests were conducted in the cold condition to define the capacity of the slabs. The tests highlighted the different structural response between using limestone and Thames Gravel aggregates, different duct material types (plastic and metallic), and different longitudinal restraint conditions. The temperature distribution through the slabs, the strains in the tendons, and the horizontal and vertical displacements were measured in each test. The fire resistance of the bonded post-tensioned concrete slabs were compared with specified values in BS 8110-2 and BS EN 1992-1-2, which were shown to be acceptable for design purposes. Although it is known that the use of Thames Gravel aggregate and restraint to thermal expansion can contribute to spalling, there was no observation of any significant spalling in the tests, as presented in Refs 1 and 2, primarily owing to the moisture content being below 2-5% by weight.

Finite element models of post-tensioned concrete slabs under fire conditions have previously been used to investigate the behaviour of unbonded slabs as presented by Lee and Bailey and Elllobody and Bailey. Compared with the modelling of unbonded slabs, the behaviour of bonded post-tensioned concrete slabs is more complicated owing to the presence of ducts, the bond between the tendons and grout, the bond between the grout and ducts, and the bond between the ducts and concrete, as well as the fact that the bond between any two components may not be maintained at elevated temperatures. The current paper highlights the behaviour of post-tensioned bonded one-way spanning concrete slabs in fire conditions through non-linear numerical modelling. A detailed thermo-mechanical three-dimensional (3D) finite element model was developed using Abaqus and verified against the tests conducted by Bailey and Elllobody. A parametric study has been conducted to study the effects of the change in the aggregate type, duct type, load ratio, fire scenarios and boundary conditions on the behaviour of bonded post-tensioned one-way concrete slabs at elevated temperatures. The results of the numerical investigation have been compared with the design values specified in BS 8110-2 and BS EN 1992-1-2, with detailed discussions and conclusions presented.

2. SUMMARY OF EXPERIMENTAL INVESTIGATION

As shown in Table 1, ten tests—two at ambient temperature (TB1 and TB2) and eight at elevated temperature (TB3–TB10)—were carried out on bonded post-tensioned one-way spanning concrete slabs. The main variable parameters in the tests were the boundary conditions (free and restrained longitudinal expansion), the aggregate type (limestone and Thames Gravel) and the duct type used within the bonded slabs (plastic and steel). The slabs were designed according to BS 8110-1 and the Concrete Society technical report 43. The slabs were 4-3 m long, 1.6 m wide and 160 mm deep. Each slab had three longitudinal parabolic tendons with a nominal diameter of 15.7 mm and an area of 150 mm². The tendon was a monowire with seven high-strength steel wires, with an average measured tensile strength of 1846 MPa. Plastic ducts, 23 mm internal diameter, were used with slabs TB3, TB5 and TB6. Galvanised steel ducts with 40 mm internal diameter were used with slabs TB7, TB8, TB9 and TB10. The ducts were positioned in a second degree parabolic shape before casting the concrete. Steel chairs were used to create the parabolic shape for the ducts to ensure that the distance between the bottom surface of the slab to the centre of the tendon was 42 mm. Each slab had three longitudinal ducts, where one duct was positioned in the middle of the slab with the other two positioned either side, at a spacing of 530 mm, as shown in Fig. 1. The duct ends were positioned exactly at the mid-height of the slab, where the dead and live anchorages were located. One tendon was threaded through each duct before casting the concrete. Bursting reinforcement was designed according to BS 8110-1 to resist tensile bursting forces around individual anchorages. No other non-tensioned reinforcement was included.

The concrete cube strength at 4, 14 and 28 days after casting, as well as at the time of testing, is shown in Table 2 together with the measured moisture content for the slabs which were tested.
under fire conditions. Grouting followed BS EN 447\textsuperscript{18} using a mixing ratio of ordinary Portland cement to water of 2:5:1. Grouting was conducted immediately after the post-tensioning of the tendons. The measured grout strength was 47 MPa at 28 days. The full applied design prestressing force to the slabs was 195 kN, with the average measured force in the three tendons being 169 kN, equating to 13% losses. Further details of the post-tensioning process, and measurements obtained during this process, are presented in Ellobody and Bailey.\textsuperscript{19}

The slabs were tested over a span of 4-0 m. The test set-up, loading positions and instrumentation are shown in Fig. 2. In the ambient tests, the slabs were loaded to failure, while in the fire tests, the slabs were subjected to a static load equal to 78-3 kN representing a load ratio of 0.6. The slabs were loaded at four locations using spreader plates 1600 \times 350 \times 40 \text{mm}, as shown in Fig. 3. The applied load, the strains in the tendons, and the vertical deflections were measured in all tests. In the fire tests, temperatures through the slab were also measured together with longitudinal displacements in the unstrained slabs, at the locations shown in Fig. 2, with the middle 3:2 m of the slab heated. The strains in the tendons were measured near the ends of the tendons (Fig. 2), ensuring that gauges remained at ambient temperature. As the tendons were bonded these gauges should give a good indication of the strains at their location near the dead ends. The slabs were heated in the fire tests using two burners following the standard time–temperature curve specified in BS EN 1991-1-2.\textsuperscript{20} Further details of the tests and results obtained are given in Ref. 1.

In the restrained fire tests, two restraining beams were designed and positioned against both ends of the slab to prevent horizontal displacement and end rotation of the slab. The beams were bolted to the loading frame, which allowed an initial expansion (approximately 4-5 mm) to occur until the slab became in full contact with the restraining beams and until the bolts fixing the restrained beams became intact with the edges of the holes. Further details for the test set-up of the restrained fire tests is given in Ref. 1.

3. FINITE ELEMENT MODEL

The bonded post-tensioned concrete slab components were modelled using a combination of 3D solid elements (C3D6 and C3D6) available within the Abaqus\textsuperscript{15} element library. The elements have three degrees of freedom per node. Owing to symmetry, only one quarter of the bonded slab was modelled (Fig. 4), with 6894 elements, including interface elements, used. The boundary conditions and load application were identical to that used in the tests. The measured post-tensioning stress in the tendons (169 kN) was initially applied in a separate step. The dead load representing the weight of the slab was applied as a static uniformly distributed load on the top surface of the slab and the dead load representing the weight of the loading tree (Fig. 3) was applied as a static distributed load over the area of the spreader plates. For the ambient tests the jack load was applied in increments as a static distributed load over the area of the spreader plates. For the fire tests the applied load remained constant and the temperature within the slab increased.

For the modelling of the fire tests a thermal analysis was conducted, using the heat transfer option available within Abaqus,\textsuperscript{15} to calculate the temperature distribution throughout the slabs, based on the measured furnace temperature from the test. A constant convective coefficient (\(\alpha_c\)) of 25 W/m\textsuperscript{2}K was assumed for the exposed surface and 9 W/m\textsuperscript{2}K was assumed for the unexposed surface. The radiative heat flux was calculated using a concrete emissivity \((\varepsilon)\) value of 0.7.

Concrete was modelled using the damaged plasticity model implemented within Abaqus.\textsuperscript{15} Under uniaxial compression the response is linear until the value of the proportional limit strain \((\varepsilon_p)\) is reached, which is assumed to be equal to 0-33 times the compressive strength \((f_c)\), as shown in Fig. 5(a). Under uniaxial tension the stress–strain response follows a linear elastic relationship until the value of the failure stress. The tensile failure stress is taken as 0-1 times the compressive strength of concrete \(f_c\), which is assumed to be equal to 0-67 times the measured concrete cube strength \((f_{cm})\). The softening stress–strain response, past the maximum tensile stress, was represented by a linear line defined by the fracture energy and crack band width. The fracture energy \(G_f\) (energy required to open a unit area of crack) was taken as 0-217 N/mm.\textsuperscript{19} The fracture energy divided by the crack band width was used to define the area under the softening branch of the tension part of the stress–strain curve, as shown in Fig. 5(b). The crack band width was assumed as the cubic root of the volume between integration points for a solid element, as recommended by Comité Européen du Béton (CEB).\textsuperscript{21}

The measured stress–strain curve of the tendon at ambient temperature, as shown in Fig. 6, was used in modelling the tendons. Based on experimental observations,\textsuperscript{3} there was no slip at the duct–grout interface as well as at the duct–concrete interface. Hence only the contact between the tendon and grout was modelled, utilising interface elements (using the contact pair option) available within the Abaqus\textsuperscript{15} element library. The fracture energy and crack band width are functions of the maximum tensile stress, and the measured stress–strain curve of the tendon. The fracture energy was calculated using a concrete emissivity \(\varepsilon\) value of 0.7.

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<table>
<thead>
<tr>
<th>Slab</th>
<th>Aggregate type</th>
<th>Measured concrete strength at different days after casting: MPa</th>
<th>Moisture content: %</th>
</tr>
</thead>
<tbody>
<tr>
<td>TB1</td>
<td>Limestone</td>
<td>32.3 39.2 40.0 41.2</td>
<td>—</td>
</tr>
<tr>
<td>TB2</td>
<td>Limestone</td>
<td>22.1 30.3 31.7 30.3</td>
<td>—</td>
</tr>
<tr>
<td>TB3</td>
<td>Limestone</td>
<td>25.2 36.4 39.4 36.6</td>
<td>1-19</td>
</tr>
<tr>
<td>TB4</td>
<td>Limestone</td>
<td>32.9 41.5 43.9 40.9</td>
<td>1-93</td>
</tr>
<tr>
<td>TB5</td>
<td>Thames Gravel</td>
<td>24.0 35.0 35.3 35.5</td>
<td>1-07</td>
</tr>
<tr>
<td>TB6</td>
<td>Thames Gravel</td>
<td>30.8 37.2 39.3 38.6</td>
<td>2-50</td>
</tr>
<tr>
<td>TB7</td>
<td>Limestone</td>
<td>29.2 36.0 39.1 40.4</td>
<td>2-43</td>
</tr>
<tr>
<td>TB8</td>
<td>Limestone</td>
<td>29.0 37.4 40.0 42.3</td>
<td>1-84</td>
</tr>
<tr>
<td>TB9</td>
<td>Thames Gravel</td>
<td>26.5 35.3 38.9 36.9</td>
<td>2-27</td>
</tr>
<tr>
<td>TB10</td>
<td>Thames Gravel</td>
<td>32.0 35.1 38.6 39.3</td>
<td>2-18</td>
</tr>
</tbody>
</table>

Table 2. Measured concrete cube strength and moisture content
library. The interface elements consisted of two matching contact faces from the tendon elements and surrounding grout elements. As observed experimentally, and discussed later in the paper, there was a significant decrease in the strains of the tendons in most of the fire tests. The decrease initiated at a tendon temperature of 100°C owing to water migration from the grout, suggesting a possible loss of bond at this temperature. Within the model a friction coefficient of 0.25 was therefore considered between the two faces until the
tendon temperature reached 100°C and was then taken as zero after this temperature. The interface element allowed the surfaces to displace relative to each other, based on the friction coefficients, but ensured that the contact elements could not penetrate each other when there was no friction.

The stress–strain–temperature curves for concrete in compression and tension are shown in Fig. 5. The curves were based on the reduction factors given in BS EN 1992-1-2\(^4\) for calcareous aggregate, which were adopted for the limestone aggregate used in this study, and siliceous aggregate, which were adopted for Thames Gravel. The specific heat and thermal conductivity were also calculated according to BS EN 1992-1-2\(^7\) with the measured moisture content (see Table 2) considered in the calculation of the specific heat of concrete. The measured stress–strain curve for the tendon at ambient temperature was used to calculate the stress–strain curves at elevated temperatures following the reduction factors given in BS EN 1992-1-2\(^4\) as shown in Fig. 6. Similar curves were used for the anchorages, with material properties at ambient temperature conforming to BS 4447\(^2\). The stress–strain curves of the plastic and steel ducts were predicted based on data provided by the manufacturers.

Recent finite element modelling presented in Ref. 14 recommended the use of measured values of thermal expansion coefficients for concrete with limestone and Thames Gravel aggregates available in the literature\(^23,24\) instead of the values given in the code\(^4\). A linear thermal expansion coefficient of \(8.1 \times 10^{-6}/°C\) was used for the concrete with limestone aggregate based on measurements by Ndon and Bergeson\(^23\). For the concrete with Thames Gravel a linear thermal expansion coefficient of \(13.2 \times 10^{-6}/°C\) was used based on measured values given by the Building Research Board\(^24\). The same values were used to model the thermal expansion of the bonded concrete slabs with limestone and Thames Gravel investigated in this study.

For the modelling of the restrained fire tests, detailed in Ref. 1, interface elements, similar to that used between the tendons and grout, were used between the slab ends and the restraining beams. The interface elements allowed the initial thermal expansion and rotation of the slab observed in the tests to occur until the slab was fully in contact with the restraining beams. After this stage, the longitudinal expansion was prevented. The faces of the interface elements were the slab end (the slave surface) and a rigid plate representing the flange of the restraining beam (the master surface). The initial clearance between the two faces was taken as 4.5 mm based on the average initial expansions observed in the tests\(^1\). Only horizontal restraint to expansion of the slab was provided. The slab was free to contract, corresponding to the test conditions.

### 4. VERIFICATION OF THE FINITE ELEMENT MODEL

#### 4.1. Verification of the ambient tests

The results obtained from the finite element model in terms of the ultimate loads, load-central deflection curves, failure modes and stresses in the tendons were compared against the test results. The load–central deflection relationship obtained from slab TB1, in comparison with that obtained from the model, is shown in Fig. 7. Good agreement was achieved between numerical and experimental results. The failure load observed experimentally was 215.4 kN, at a central deflection of 112 mm, compared with 212.5 kN and 105 mm obtained from the model. The failure load predicted using the model was 1.3% lower than that observed from the test. The deformed shape at failure for slab TB1, obtained from the model, is shown in Fig. 8, which is in good agreement with that observed in the test TB1\(^5\). Fig. 8 also shows the stress distribution in the concrete.
elements at failure. It can be seen that the concrete element stresses in the compression zone of the maximum bending moment region under the middle spreader plates approached the maximum compressive stresses. The mode of failure of concrete crushing in the model therefore corresponded to the mode of failure observed experimentally and reported in Refs 1 and 2. The load–strain curves of the tendons obtained experimentally and numerically are presented in Fig. 9. The strains were recorded by the three strain gauges fitted on the left tendon (LT), middle tendon (MT) and right tendon (RT) of the bonded slab TB1, as shown in Fig. 9. It should be noted that the strains observed experimentally remained constant from the start of loading until the end of the test, while that predicted numerically had an increase of 560 microstrain from a load 183.1 to 212.5 kN. The strains recorded from the finite element model were averaged over the full cross-sectional area of the bar whereas in the test the strains in the individual wires were measured. The absolute average value used in the model was close to the maximum stressed wire, as shown in Fig. 9. At higher loads, the strains recorded in the tendon from the finite element model slightly exceeded the elastic strains. Similar conclusions could be drawn from the comparison of load-strain curves of the tendons for the bonded slab TB2, as shown in Fig. 10.

The load–central deflection curves obtained experimentally and numerically for slab TB2 are compared in Fig. 11. Once again the experimental and numerical curves are in good agreement. The failure load observed experimentally was 187.9 kN at a central deflection of 107 mm compared with 203.9 kN and 108 mm obtained from the model. The failure
The difference in the failure loads is attributed to the unexpected lower concrete strength of slab TB2 (30.3 MPa) compared to slab TB1 (41.2 MPa) as reported in Refs 1 and 2. Interestingly, as the post-tensioning of the slabs took place before grouting, the axis of the tendon might have shifted upwards away from the centre of the duct, especially at the centre span of the slab. The effect of the movement of the tendon is particularly prominent in slab TB2 owing to the greater diameter (40 mm) of the metallic duct used. This was highlighted using the model by moving the tendon inside the metallic duct by a nominal 2 mm upwards at the centre span of slab TB2, which reduced the failure load to 176.6 kN at a central deflection of 94 mm, as shown in Fig. 11. As the movement of the tendon inside the duct cannot be controlled, and is dependent on the diameter of duct used, the designed tendon axis distance (i.e. at the centre of the duct) will be used in the finite element analysis for all the slabs investigated in this study. The deflected shape, stress contours and failure modes predicted for slab TB2 were similar to that of slab TB1 shown in Fig. 8.

4.2. Verification of the fire tests

To indicate the accuracy of the thermal modelling, the temperature distribution throughout the bonded post-tensioned concrete slabs was plotted and compared against the tests. As an example, the predicted and recorded test temperatures at the hot surface (HS), the tendon (T), the mid-surface (MS) and the cold surface (CS) at the centre of slab TB3 is shown in Fig. 12. Similar accuracy was obtained when the model was compared against the other fire tests. The time–central deflection curves obtained from the unrestrained tests and the finite element analysis were compared and good agreement was achieved. As an example, Fig. 13 shows the central deflection–time relationships plotted for the unrestrained slabs TB3 (having plastic ducts and limestone aggregate) and TB5 (having plastic ducts and Thames Gravel aggregate). The central deflections observed experimentally for slabs TB3 and TB5 were 59 mm at 94.5 min and 82 mm at 88 min, respectively, compared with 66 mm at 94.5 min and 95 mm at 86 min numerically. The axial movements for slabs TB3 and TB5 were also compared experimentally and numerically, as shown in Fig. 14. The maximum axial movements were 4.2 and 7.2 mm at the mid-height of the slab, recorded in the slabs TB3 and TB5 respectively, compared with 4.1 and 7.5 mm from the model. Considering Figs 13 and 14, it can be seen that both the test and finite element results show that the slab TB5, which had Thames Gravel, had significant higher vertical and horizontal
displacements compared with slab TB3 having limestone aggregate owing to the higher thermal expansion of the Thames Gravel.

Figures 15 and 16 show the central deflection–time curves plotted for the restrained fire slabs TB8 (having metallic ducts and limestone aggregate) and TB10 (having metallic ducts and Thames Gravel aggregate), respectively. For slab TB8, the maximum central deflection observed experimentally was 35 mm at 55.5 min from the start of heating, compared with 37 mm at 48.4 min from the model. For slab TB10, the maximum central deflection observed experimentally was 39 mm, recorded at 39 min from the start of heating, compared with 40 mm predicted at 43 min from the model. In both tests it can be seen (Figs 15 and 16) that there is an excellent agreement between test results and the model. The finite element model was able to accurately follow the test from the start of heating, where the slab is not fully restrained, until it became in full contact with the restraining beam.

The stresses in the tendons were predicted from the finite element analysis for all the bonded slabs. The measured strains recorded by the strain gauges fitted on the middle tendon (MT1, MT2 and MT3), left tendon (LT1, LT2 and LT3) and right tendon (RT1 and RT2) were used to predict the stresses in the tendons adopting the reduction factors given in BS EN 1992-1-2. As an example, the stresses predicted numerically were compared with those obtained experimentally for slab TB3, as shown in Fig. 17. Generally good agreement was achieved between experimental and numerical results. It can be seen that the prestress force in the tendon started to decrease significantly after 35 min as observed experimentally and confirmed numerically. As explained previously, the strains recorded from the finite element model were averaged over the full cross-sectional area of the bar whereas in the test the strains in the individual wires were measured. This caused the prestress force predicted numerically to decrease gradually as shown in Fig. 17. Similar behaviour was observed for the other bonded fire tests.

Taking the restrained slab TB4 as an example, the stresses at the maximum temperature are shown in Fig. 18. As slab TB4 was restrained, the strains in the concrete elements at the restraint position approached the maximum compressive strains. Hence local concrete crushing was predicted using the finite element model, which compares well with the behaviour observed experimentally in the restrained tests. Fig. 19 shows the cracks and spalling of the restrained slabs TB4 and TB6 observed in the tests. Fig. 18 also shows that tensile stresses are concentrated at the tendon position and extend towards the top surface in the longitudinal direction of the slab.

Similar to the fire tests conducted on unbonded post-tensioned concrete slabs, longitudinal cracks directly above and in line with the tendons were observed in all the bonded slabs. The cracks appeared on the unexposed surface of the bonded slabs between 15 and 19 min, corresponding to a tendon temperature of approximately 110°C. The cracks are caused by tensile stresses being induced perpendicular to the trajectory of the tendons owing to lateral thermal expansion at elevated temperatures. Detailed explanation of the cause of the cracks, when investigating unbonded slabs, was previously provided using finite element modelling conducted by the authors.

5. PARAMETRIC STUDIES AND DISCUSSIONS

The verified finite element model was used to investigate the effects of the change in aggregate type, duct type, load ratio, boundary conditions and different fire scenarios, on the global structural behaviour of bonded post-tensioned one-way concrete slabs in fire. The different parameters considered are summarised in Table 3.
Group G1 included three slabs S1–S3 that had plastic ducts (PL), limestone aggregate (LS) and load ratios of 0.3, 0.5 and 0.7 respectively for slabs S1, S2 and S3. Group G2 (slabs S4–S6) was identical with G1 except the type of aggregate used was changed to Thames Gravel (TG). Groups G3 (slabs S7–S9) and G4 (slabs S10–S12) were identical with G1 and G2 respectively, except the slabs had metallic ducts (M). All the four groups (G1–G4) were allowed free longitudinal expansion. Groups G5 (slabs S13–S15), G6 (slabs S17–S18), G7 (slabs S19–S21) and G8 (slabs S22–S24) were identical with G1, G2, G3 and G4 respectively, except the slabs were restrained against longitudinal thermal expansion. All the slabs in the eight groups (G1–G8) were subjected to the standard fire curve, as shown in Fig. 20. Group G9 (slabs S25–S27) had limestone aggregate, plastic ducts, a load ratio of 0.7, were allowed free longitudinal expansion, and were subjected to different fire curves. The fire curves used represent the ‘standard’ (ST), ‘short hot’ (SH) and ‘long cool’ (LC) fires shown in Fig. 20 for slabs S25, S26 and S27 respectively. More details regarding the ‘short hot’ and ‘long cool’ fires can be found in Lamont et al. The fire resistance and maximum central deflections for all the slabs investigated in the parametric study are summarised in Table 3. The time–central deflection relationships for all the slabs are plotted in Figs 21–26.

Similar to the tested slabs, the tendon had a cover of 30 mm at the central span of the slab, which according to BS 8110-23 should achieve 90 min fire resistance. BS EN 1992-1-2 specifies fire resistance in terms of the distance from the bottom of the slab to the centre of the tendon (denoted axis distance), which in this case is 42 mm. For 90 min fire resistance BS EN 1992-1-2 specifies an axis distance of 45 mm and for 60 min an axis distance of 35 mm. The slab should therefore have at least 60 min fire resistance (as shown in Table 3). It can be seen that all the bonded post-tensioned one-way spanning concrete slabs investigated in the parametric study had a fire resistance of approximately 105 min with slabs S26 and S27 surviving the full duration of the natural fire. This indicates that the codified values in BS 8110-2 and BS EN 1992-1-2 are conservative, however the fire resistance specified in BS 8110 were more closely aligned with the finite element analysis predictions.

The effect of the load ratio, defined as the applied load divided by the design capacity, was investigated by modelling the three slabs in each group, except group G9, for different load ratios of 0.3, 0.5 and 0.7 (Table 3 and Figs 21–24). It can be seen that the greater the load ratio the greater the vertical deflection of the slab for a given time. For the restrained slabs, however, the final
deflections were limited by the restraining force created by the restraining beams. Of interest is the fact that all the slabs failed at the same time (105 min) irrespective of the load ratio. Previous experience has shown that the increase in the load ratio generally results in a decrease in the fire resistance. For the bonded post-tensioned concrete slabs investigated in this study, however, failure in the model was initiated owing to a crack occurring directly in line and parallel to the tendons. Similar observations were previously reported in Ref. 14. When the temperature of the tendons increases the tendon starts to expand longitudinally causing partial relief of the compressive stresses, which then allows the lateral expansion to cause splitting at the weaker sections of the slab. The same failure mode has also been observed in an independent study conducted by Herberghen and Damme.7 From the study presented here it can be concluded that the splitting mode of failure is not affected by the applied static load. As shown in Table 3, all the slabs failed at 105 min, which is higher than the 90 min specified in BS 8110-23 and the 60 min specified in BS EN1992-1-2.4

As shown in Table 3 and Figs 21–24, the slabs with Thames Gravel aggregate have a significantly higher vertical deflection compared with the equivalent slab with limestone, owing to the greater thermal expansion of Thames Gravel. For the restrained slabs, the final deflections were limited by the restraining force created by the restraining beams. Owing to the variation of

<table>
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Table 3. Summary of parametric study results

Fig. 20. Standard, short hot and long cool fire curves

Fig. 21. Effect of load ratio and different aggregate types (LS = limestone, TG = Thames Gravel) on free bonded tests with plastic ducts (groups G1 and G2)
Looking at the restrained slabs in Table 3 (and the predicted response shown in Figs 22 and 24), it can be seen that owing to the longitudinal restraint to thermal expansion, as the slab tries to expand, a restraining moment at the ends was created which together with arching action reduces the central vertical displacement. The failure mode was attributable to tensile splitting cracks at the tendon locations as well as localised failure in the proximity of the restraint at the bottom of the slab. It can also be seen that the maximum deflections were observed generally between 40 and 60 min, after which the deflections started to decrease or remained approximately constant until the failure was observed at approximately 105 min for all the slabs. Once again, the fire resistances predicted were higher than that specified in BS 8110-2 and BS EN1992-1-2.

The temperatures throughout the bonded post-tensioned slabs S25, S26 and S27 analysed for different fire scenarios are shown in Fig. 25. The thermal analysis was conducted for slabs S26 and S27 which were heated using the ‘short hot’ and ‘long cool’ fires, respectively, during the heating and cooling stages of the fires (a total period of 540 min). Slab S25, which was heated using the standard fire curve, was also analysed for the same period in the thermal analysis. Slab S25 failed in the thermal-mechanical analysis at 106 min, however, owing to the tensile splitting along the tendons, as shown in Fig. 26.

temperature through the depth of the slab, thermal curvature contributes to the vertical deflection (with the bottom part of the slab expanding greater than the top part), together will loss of strength of the tendon and concrete, with the Thames Gravel slabs having the greater displacement.
Looking at Fig. 25, it can be seen that the tendon temperature predicted using the ‘short hot’ fire was greater than that predicted using the ‘standard’ and ‘long cool’ fires in the first 50 min, from the start of heating. After that the tendon temperature was much higher in the slab using the ‘standard’ fire curve compared with the other two curves. Fig. 26 shows the time–central deflection curves plotted for slabs S25, S26 and S27. The maximum deflections predicted from the finite element model were 106 mm at 105 min, 52 mm at 55 min and 118 mm at 214 min, respectively as shown in Fig. 26.

The maximum central deflections of the slabs with plastic and metallic ducts (summarised in Table 3 and plotted in Figs 21–24), show that the type of duct has a nominal effect on the time–central deflection behaviour owing to the fact that the ducts are subjected to compressive stresses created by the post-tensioning.

6. CONCLUSIONS
A detailed thermo–mechanical non-linear finite element model, for the analysis of bonded post-tensioned concrete slabs at elevated temperatures, has been developed and presented. The interface between the tendon and surrounding grout was modelled, allowing the tendon to retain its profile shape during the deformation of the slab and representing the bond behaviour between the tendon and grout. The temperature distribution throughout the slab, the time–deflection behaviour, time–longitudinal expansion, time–stress behaviour of the tendon, and the failure modes have been predicted by the model and verified against experimental results. The comparison between the experimental and numerical results has shown that the model can accurately predict the behaviour of bonded post-tensioned one-way concrete slabs under fire conditions. It is also shown that the model provides excellent representation for the varying boundary conditions at elevated temperatures.

The developed model was used to investigate the effects on the global structural behaviour owing to the change in the aggregate type, duct type, load ratio, boundary conditions and different fire scenarios. It is also shown that the central deflection is higher with an increase of the load ratio, but the fire resistance time remains the same owing to the mode of failure, comprising cracking (splitting) above the tendons, not being influenced by the magnitude of load. Similarly, using different aggregates influenced the displacement–time response but did not significantly affect the failure fire resistance time. The numerical results were compared with values calculated using current design codes. The comparison has shown that the bonded post-tensioned concrete slabs investigated in this study are capable of achieving the designed 90 min fire resistance. It is also shown that the fire resistance given by BS 8110 and BS EN 1992–1–2 are conservative for bonded post-tensioned one-way spanning concrete slabs under fire conditions. It was shown that BS 8110 predictions were more aligned with the finite element analysis predictions.

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