Influence of Reinforcement Properties on the Failure of Composite Slabs in Fire

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ABSTRACT

This paper examines the influence of the properties of steel reinforcement on the response of composite floor slabs under idealised fire conditions, with emphasis on ultimate failure considerations. An experimental investigation into the effect of elevated temperatures on the mechanical characteristics of steel reinforcement is firstly described. The study includes tests carried out at ambient temperature as well as under steady state and transient elevated temperature conditions. Apart from the evaluation of stress-strain response and degradation of stiffness and strength properties, particular emphasis is given to assessing the influence of elevated temperature on enhancing the ductility of steel reinforcement. The implications of the specific reinforcement properties on the ultimate behaviour of composite floor elements and assemblages in fire are then discussed. For this purpose, novel analytical models are used to assess the ultimate behaviour of members incorporating different types of reinforcement.

1 INTRODUCTION

The structural response of buildings to fire conditions has been the focus of intensive research activity in recent years. For composite steel/concrete buildings, this has been largely motivated by the desire to achieve more cost-effective designs which are based on the actual structural performance rather than typical prescriptive methods which are based on unrealistic idealisations of isolated elements. This is particularly relevant at elevated temperature, when the interactions between various structural components may have a direct influence on the response and are potentially advantageous to the overall building performance. However, before the potential benefits can be incorporated in to design methods, it is necessary to gain a detailed understanding of the underlying behavioural mechanisms in fire conditions.

Towards this end, significant insights into the actual structural response of buildings in fire were provided through the large-scale tests at Cardington [1, 2]. The findings of these tests identified the important role played by the composite floor slab in carrying the gravity loading within the fire compartment after the loss of strength in the supporting secondary steel beams due to elevated temperature. It was shown that the floor slab continues to support load through membrane action even after the loss of the deck and steel beams, thereby enabling alternative load paths to develop after conventional strength limits have been reached.

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Before the above-mentioned secondary load-carrying mechanisms can be relied upon in design, it is necessary to understand the limiting failure criteria. Apart from compressive mechanisms that may occur in the slab, a key failure condition is related to fracture of the steel reinforcement in tension. In this respect, fundamental analytical approaches have recently been proposed which are capable of predicting the level of deformation and load corresponding to failure by reinforcement fracture at elevated temperature [3-6]. The reliability of these methods, however, is directly dependent on the availability of information pertaining to the key material characteristics at elevated temperature.

In this context, this paper describes the observations from an experimental investigation into the effect of elevated temperature on reinforcing bars tested to fracture. A primary objective of the study is to gain an insight into the effect of elevated temperature on the ductility properties of steel reinforcement. Following this, the findings of the tests are employed, together with the analytical models, to investigate the behaviour of idealised reinforced concrete members under simulated fire conditions.

2 TEMPERATURE-DEPENDENT PROPERTIES

Elevated temperature has the effect of reducing the strength and stiffness of steel reinforcement [6, 7]; the reduction is directly related to the manufacturing process of the bars. For example, in Eurocode 2 [7] an idealised stress-strain relationship is assumed as depicted in Figure 1. A linear relationship is initially considered followed by an elliptical representation until the maximum stress is achieved at a strain of $\varepsilon_{sy,\theta}$, after which a constant strength is assumed between $\varepsilon_{sy,\theta}$ and $\varepsilon_{st,\theta}$. The main parameters related to stiffness and strength (i.e. $E_{s,\theta}$, $f_{sp,\theta}$ and $f_{sy,\theta}$) are assigned reduction factors for increasing temperatures. These reduction factors are discussed in subsequent parts of the paper.

More importantly, in terms of ductility, the Eurocode approach considers $\varepsilon_{sy,\theta}$, $\varepsilon_{st,\theta}$ and $\varepsilon_{su,\theta}$ as constant values irrespective of the temperature; these are stipulated as 0.02, 0.15 and 0.2, respectively (for Class B and C reinforcement) and 0.02, 0.05 and 0.1, respectively (for Class A reinforcement). Accordingly, it is assumed that the ductility of reinforcement is unaffected by the level of temperature, an assumption which is examined in more detail in the experimental investigation described in this paper.

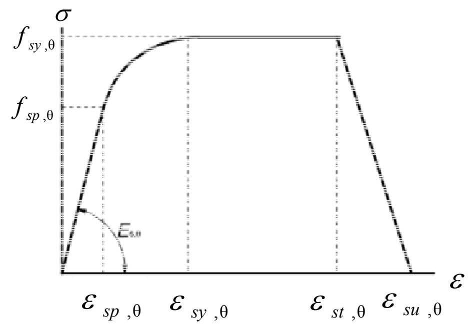

Figure 1. Stress-strain relationship for reinforcement at elevated temperature [13]

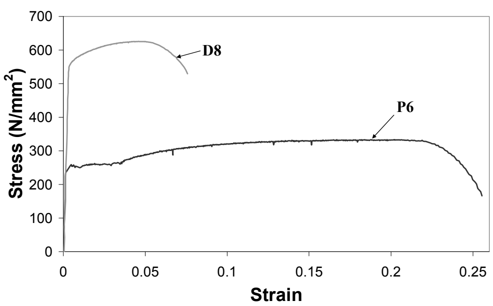

Figure 2. Typical stress-strain relationships at ambient temperature

3 EXPERIMENTAL STUDY

The main objective of the material tests was to examine the variation in key properties of steel reinforcement with temperature. Particular emphasis is given to the influence of temperature on ductility, in terms of ultimate strain at fracture, which is critical for the reliable assessment of the performance of structural members under fire conditions. The full test program included (i) steady-state elevated temperature tests; (ii) transient elevated temperature tests at a constant load; and (iii) steady-state tests for assessing residual properties. For brevity, this paper focuses on the results of (i) only; the other tests are described elsewhere [8].

3.1 Experimental Response at Ambient Temperature

In order to assess the behaviour of steel reinforcement of different characteristics, two bar types are considered, namely 6mm plain hot-rolled bars (P6) and 6mm ribbed cold-worked bars (D6). Other bars were also included in the full test programme but are not included in this paper for brevity (see [8]). The results from these two bars are indicative of the general observations during the full test programme. Ambient tensile tests were conducted for each bar type to ascertain the mechanical characteristics; typical stress-strain relationships obtained for each of the reinforcement configurations are presented in Figure 2. In addition, the key mechanical characteristics are summarised in Table 1 where f_{sy} and f_{su} are the yield and ultimate strengths at ambient, respectively, and ε_{su} is the corresponding ultimate strain, measured through an extensometer. The values given in the table are the average obtained from at least three specimens for each bar-type.

Table I: AMBIENT STEEL REINFORCEMENT PROPERTIES

	f_{sy}	f_{su}	ε_{su}
P6	251	328	0.20
D6	551	592	0.04

3.2 Steady-State Elevated Temperature Tests

The test arrangement is shown in Figure 3. A hydraulic testing machine was utilised and the temperature was applied using an electric furnace. Each specimen had a full length of 1000mm with a heated segment of 325mm. As well as overall load and displacement readings, the extension in the heated part of the bar was measured using the arrangement shown in Figure 4. In addition, the reinforcement was marked at 30 mm intervals prior to testing to facilitate the measurement of ultimate strain after cooling. Once the specimen and furnace were in position, the temperature was increased to the required level at a rate of 10ºC/minute. This temperature was then maintained for 30 minutes before tensile loading was applied, through displacement control, at a rate of 4mm/minute until fracture occurred. In this paper, the stress-strain relationship at a given temperature, θ, is defined by four key parameters: (i) the slope in the linear-elastic range ($E_{s,\theta}$); (ii) the proportional limit ($f_{sp,\theta}$) after which non-linear behaviour is exhibited; (iii) the ultimate stress ($f_{su,\theta}$) corresponding to the maximum capacity of the bars; and (iv) the ultimate mechanical strain at fracture ($\varepsilon_{su,\theta}$).

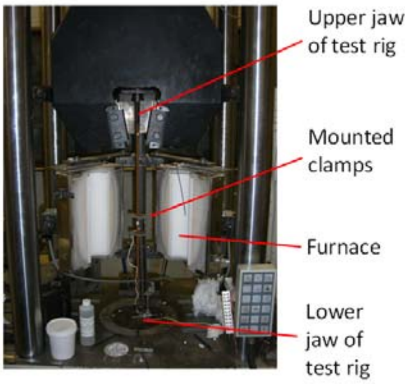

Figure 3. Elevated temperature testing arrangement

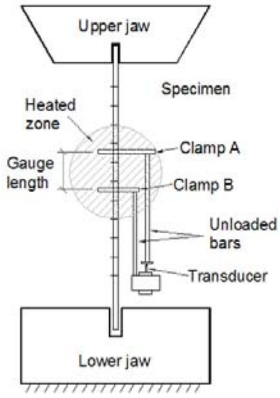

Figure 4. Arrangement for measuring bar extension

3.2.1 EVALUATION OF STRENGTH AND STIFFNESS

The experimental response curves obtained for P6 and D6 are shown in Figures 5 and 6 respectively, presented in terms of stress against extension. In addition, the degradation of reinforcement properties with elevated temperature are presented in Figures 7 and 8 for P6 and D6 respectively, where the reduction factors are normalised by their corresponding values at ambient conditions, and plotted against the temperature (θ). For comparison purposes, the plots also include the reduction factors suggested in Eurocode 2 [13] for hot-rolled and cold-worked bars.

With reference to the overall shape of the stress-strain response depicted in Figures 5 and 6, it is evident that the clear yield-plateau, demonstrated by the hot-rolled bars (P6) at ambient temperature, disappeared at temperatures above 200°C and the behaviour became more continuous. Furthermore, strain-hardening diminished for both bar-types from around 400-500°C. This conflicts with the Eurocode which assumes that strain hardening is negligible at all temperatures and hence the maximum stress level is essentially treated as an 'effective yield strength', refereed to as $f_{sy,\theta}$. The test results shown in this paper suggest that strain hardening becomes insignificant only when temperatures above 400°C are reached. Characterisation of a representative effective yield strength at elevated temperature from the experimental results is not possible without either: (i) defining a limiting strain criteria, which is difficult due to the variable $E_{s,\theta}$, or (ii) ignoring the presence of strain hardening characteristics as assumed in EC2.

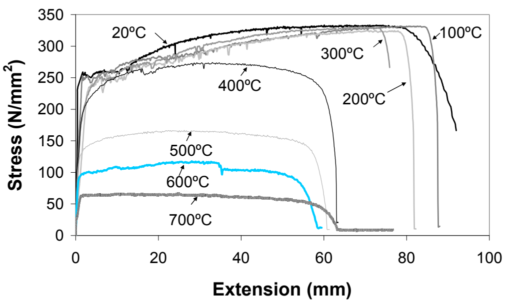

Figure 5. Stress versus extension response for P6 bars at various temperatures

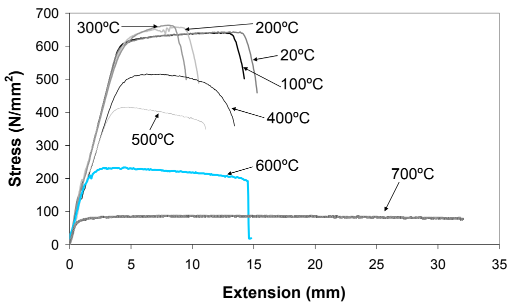

Figure 6. Stress versus extension response for D6 bars at various temperatures

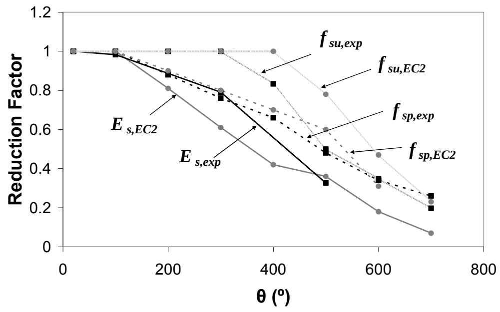

Figure 7. Effect of elevated temperature on properties of P6 reinforcement

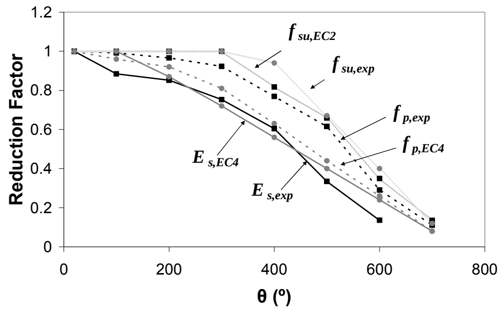

Figure 8. Effect of elevated temperature on properties of D6 reinforcement

In terms of $E_{s,\theta}$, $f_{sp,\theta}$, and $f_{su,\theta}$, Figures 7 and 8 indicate that each of these properties decrease gradually with temperature. $E_{s,\theta}$ and $f_{sp,\theta}$ reduce at a relatively constant rate at temperatures above 100-200°C and are largely in agreement with the corresponding Eurocode values. On the other hand, $f_{su,\theta}$ does not reduce until around 300-400°C after which it degrades at a similar rate for both bar-types. The 'effective yield strength' of cold-formed bars typically reduces more than that of hot-rolled reinforcement at elevated temperature. However, for the bars in this study, this is counterbalanced by the greater strain-hardening capacity of the P6 bars at ambient temperature. Consequently, both types display similar trends of ultimate strength when the normalised values are assessed. It is noteworthy that the test results for P6 (hot-rolled) bars show that the corresponding Eurocode values appear to be un-conservative.

3.2.2 REINFORCEMENT DUCTILITY

Figure 9 illustrates the effect of elevated temperature on the ultimate strain for both P6 and D6. It is shown that the behaviour of both the hot-rolled and cold-formed bars is similar until around 500°C, with the ultimate strain reaching around twice the corresponding ambient value. At higher temperatures, the enhancement increased significantly for the D6 bars, reaching values of between 7 to 9 times the ambient value at 700°C whereas the hot-rolled bars only increased by a factor or 2 or 3 in the same range. Clearly, when the cold-working effect is alleviated at temperatures exceeding around 500°C, the ductility increased significantly in comparison with the characteristically low values exhibited at ambient temperature for this type of reinforcement.

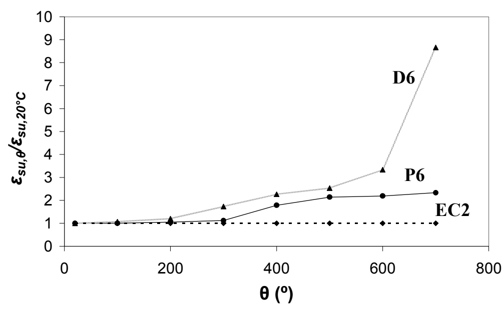

Figure 9. Effect of elevated temperature on $\varepsilon_{su,\theta}$

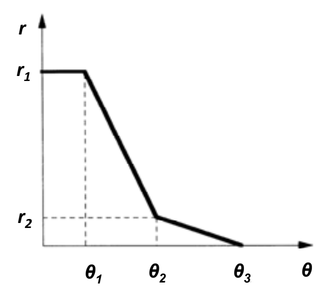

Figure 10. Variation of material reduction factors with temperature

The findings of this experimental programme are critical for the reliable assessment of the performance of structural members in fire. The subsequent section employs some of the key results to describe a brief analytical investigation into the response of reinforced concrete slab members under idealised fire conditions.

4 MEMBER RESPONSE

As previously discussed, simplified analytical models have been developed which can predict the ultimate behaviour of one- and two-way spanning slab components at ambient and elevated temperature. The models enable a fundamental assessment of the large-displacement behaviour of reinforced concrete members in fire conditions, including the failure conditions. This section provides an analysis of the behaviour of one-way spanning slab strips at elevated temperature. For compactness, the simplified analytical model (hereafter referred to as the SAM) is only briefly discussed herein; more detailed discussions can be found elsewhere [3, 4, 9]. However, the model is utilised, together with the material data presented in this paper, to investigate the influence of several parameters on the failure behaviour of slabs under realistic fire conditions.

The SAM accounts for the effects of elevated temperature including the variation in material properties as well as thermal expansion and thermal curvature. It also considers the influence of complex relationships such as bond-slip. Extensive validation of the predicted load deflection response has been carried out elsewhere [4]. In this section, focus is given to examining the influence of restraint conditions and the degradation of material properties on the ultimate response, at elevated temperature. In order to investigate the ensuing phenomena, and to illustrate important behavioural aspects, the properties of a reference configuration adopted to facilitate the interpretation of the results are presented in Table 2. The table gives details of the half-length (L), width (b), depth (h), depth of the reinforcement from the compressive face (d_s), area of steel (A_s) and reinforcement ratio (ρ). It also indicates the effective bond strength (σ_b) as well as the compressive strength of concrete (f_c'). The steel characteristics adopted replicate those of P6 reinforcement.

Table II: DETAILS OF CONTROL MODEL

Member configuration		Ambient material properties	
L	1500mm	E_s	2.1 x 10^5N/mm^2
h	60mm	f_{sy}	252N/mm^2
b	600mm	f_{su}	330N/mm^2
d_s	30mm	ε_{su}	0.2
ρ	0.23%	f_c'	40N/mm^2
A_s	85mm^2	σ_b	0.9N/mm^2

The degradation of material properties with elevated temperature are represented in the analysis through tri-linear reduction curves (Figure 10), as summarised in Table 3. The temperature-dependent material properties related to bond and concrete are taken from available information [7, 10]. In subsequent analysis, the temperature distribution is assumed to be linear within the cross-section and constant along the length.

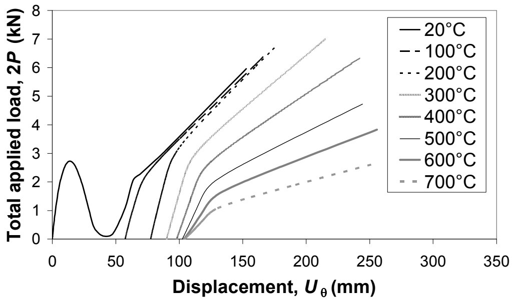

Figure 11. Response of unrestrained control model at various temperatures

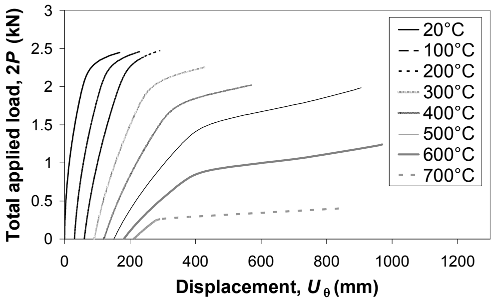

Figure 12. Response of restrained control model at various temperatures

Figure 11 presents the relationship between the total applied load (*2P*) versus the temperature-dependant deflection (U_θ) for various steel reinforcement temperatures θ. The last point on each curve corresponds to the attainment of ultimate strain in the steel and hence indicates failure. Evidently, the member capacity reduces with increasing temperature owing to material degradation whereas the failure displacement increases significantly (until about 600°C) as a result of the improved material ductility. It is noteworthy that the increase in temperature causes the member to deform even before gravity loading has been applied, resulting in a significant initial displacement. Rupture of the reinforcement depends directly on the combination of the thermal expansion characteristics together with the variation in the relevant material properties. It is seen that for the particular material properties employed in these analyses, as the temperature approaches 700°C, the failure deflection reduces again.

The effect of boundary conditions on the ultimate behaviour, at various levels of elevated temperature, is investigated by examining the response of the control model using the axially-restrained SAM (Figure 12). As before, it is evident that the response is significantly influenced by the increase in temperature, in terms of the initial and ultimate deflection, failure load and the overall response history. Most notably, the compressive arching effect is lost at elevated temperatures. The initial deformation due to the increase in temperature is sufficient to take the member beyond the range within which compressive arching can develop. Furthermore, at relatively low levels of elevated temperature, the initial deformations are more significant in this case than in the previous unrestrained model as the restraint against thermal expansion causes the member to buckle at a relatively early stage.

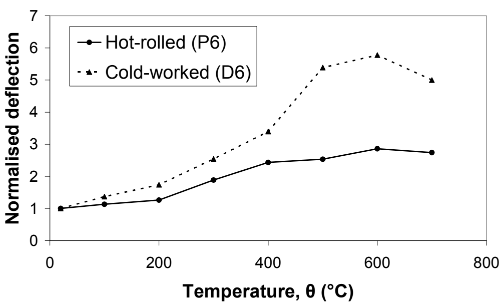

Figure 13. Effect of bar type on failure displacement at elevated temperature

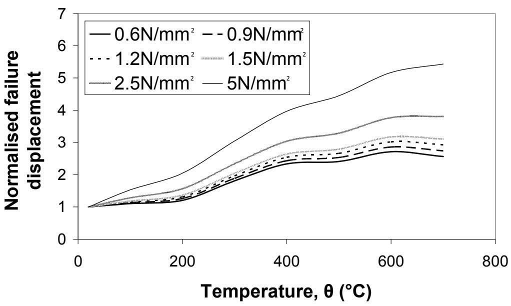

Figure 14. Effect of bond strength on failure displacement at elevated temperature

In order to investigate the effect of reinforcement type, the bars employed in the control model are varied by considering a cold-worked material with properties similar to D6. All of the other properties are retained from the previous analysis. The effect is illustrated in Figure 13 in which the failure displacements have been normalised to their corresponding values at ambient temperature and plotted against θ. Clearly, elevated temperature has a greater influence on the failure of strips with cold-worked bars than those with hot-rolled reinforcement.

It has been shown elsewhere [4, 9] that bond strength has only a marginal influence on the load-deflection response whereas, more importantly, it has a pronounced influence on the failure level. The influence of bond on the failure displacement at various levels of elevated temperature is presented in Figure 14. The ambient bond strength is varied between 0.5 and 5N/mm^2. As the bar type is unchanged, the degradation of bond with elevated temperature is assumed to be identical in each case. Evidently, the normalised failure deflection increases proportionally to the level of bond strength. In other words, elevated temperature has a greater influence on failure when the bond strength is relatively higher.

5 CONCLUDING REMARKS

This paper has provided an insight into the effects of elevated temperature on the characteristic properties of steel reinforcement and the consequent effect on member behaviour. Generally, the stiffness and strength of steel reduce progressively with increasing temperature whereas the ductility increases. The study also a brief analytical investigation into the effect of elevated temperature on the response of one-way spanning reinforced concrete members under idealised fire conditions.

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