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Residual capacity of fire-exposed concrete-filled steel hollow section columns

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ABSTRACT

Concrete filled steel hollow structural (CFS) sections are an increasingly popular means to support large compressive loads in buildings. Whilst the response of unprotected CFS sections during a fire is reasonably well researched, their post-fire residual structural performance is less well established. A better understanding of the response of fire-damaged CFS columns is needed to enable better performance-based structural fire engineering of buildings incorporating CFS sections. This paper presents post-fire residual compression tests on unprotected and protected CFS columns along with control tests on six unheated sections. The tests confirm that as the maximum exposed temperature within the cross-section increases, the residual strength capacity, ductility and axial–flexural stiffness decrease. The data are subsequently used to assess the ability to predict the residual capacity of CFS columns after fires, using available post-fire material models and in-fire and ambient structural models.

1. Introduction

Concrete filled steel hollow structural sections (CFS) are hollow steel sections (circular, prismatic, or ovular) that are in-filled with plain or reinforced concrete to provide superior load carrying capacity, and enhanced structural fire resistance, as compared with unfilled steel tubes. They are an attractive, efficient, and relatively environmentally sustainable means (as compared to plain steel or reinforced concrete members) by which to support large compressive loads in multi-storey buildings. The concrete infill and the steel tube work together to share load through composite action at ambient temperatures, but also during a fire and after a fire. The concrete infill enhances the steel tube's resistance to local buckling, and the steel tube provides confinement to the concrete core, thus increasing its load bearing capacity. The steel tube also acts as stay-in-place formwork during construction, reducing forming and stripping costs, and provides a smooth, rugged, architectural surface finish.

A large amount of design guidance is available [e.g. 1,2] to predict the fire resistance for CFS columns during standard fires. However after a fire, when a building may have not experienced any obvious structural failure, a question arises as to the level of structural damage that may have been sustained, and whether (or how) the building can be safely repaired and put back into use. Such questions are becoming more important, as structural fire engineers, insurers, building developers and tenants begin to factor other performance criteria in addition to life safety, such as property protection, environmental impacts, and business continuity considerations, when making structural fire engineering design decisions. Only limited work is available on the post-fire residual strength of fire-exposed CFS columns [3–5].

This paper presents tests on the post-fire residual compressive load bearing and lateral deformation capacity of 19 CFS columns after being exposed to standard fires and cooled to ambient temperature prior to structural testing to failure. Tests on six unheated control columns are also presented. Parameters varied between tests include the severity and duration of heating, the concrete infill type, the cross-section shape, the steel wall thickness, and the amount of supplemental fire protection. The data are then used to assess the ability of available post-fire structural and material models [1,5,6] to predict the residual capacity of CFS columns after fires.

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2. Background

Only limited research is available on the residual capacity of fire-exposed CFS columns. Han et al. [3,5] have presented tests and analysis of more than 40 CFS columns after exposure to fire, and have also suggested complex post-fire material models for use in predicting CFS columns' residual capacity [5]. Han's work includes post-fire residual tests [3,5] on both protected and unprotected, scaled CFS columns. This work has considered only the standard ISO 834 fire curve [5], or exposure to constant temperatures ranging between 20 °C and 900 °C [4], with tests on both square and circular columns ranging in length between 380 and 1200 mm and with maximum cross-sectional dimensions between 80 and 133 mm. Wall thicknesses between 2.9 and 4.8 mm have been considered. The steel tubes were filled with plain concrete ranging in strength from 35 to 72 MPa. For tests exposed to transient thermal regimes, unprotected specimens were subjected to 90 min of exposure to the standard fire, whilst protected specimens were heated for 180 min. The majority of the columns were tested under concentric axial load, however a small number had initial load eccentricities of 15–18 mm and (as expected) failed at lower loads than otherwise identical concentrically loaded columns.

Taken together, Han et al.'s published work in this area [3–7] has demonstrated that the residual mechanical behaviour of the fire exposed CFS columns under axial load remains ductile (as for ambient unheated tests), and that composite enhancement (i.e. confinement) of the concrete core remains present after heating [3]. The post-heated columns failed in either global buckling of the columns or local buckling of the steel tubes, with accompanying crushing of the concrete core. The fire duration, column section size, and slenderness ratio were observed to have significant effects on the residual strength of the columns, whereas other parameters (steel ratio, concrete strength, and steel strength) had only minor effects. Unsurprisingly, loss of strength was considerably less for protected sections [4]. Interestingly, it was noted that load eccentricity appeared to be important for the *residual strength index* (RSI) of the columns. The RSI is defined (also herein) as the ratio of the tested strength (N_{test}) to the strength of an identical unheated column ($N_{ambient}$); i.e. RSI = $N_{test}/N_{ambient}$.

Han et al. [5,6] have devoted considerable effort to developing post-fire residual material models and predictive equations for the RSI of both unprotected and protected (with a specific cementitious protection material) CFS columns after exposure to the standard fire. The material models for concrete and steel make two noteworthy assumptions, namely that: (1) the residual mechanical properties of both steel and concrete depend only on the peak exposure temperature, and are not influenced by the cooling rate, the time since heating, or the relative humidity of the cooling environment (which is known to influence the residual properties of concrete, in particular [8]); and (2) steel tubes in CFS columns provide confinement to the core concrete both before and after fire, and enhance its compressive strength and deformability (more effectively for circular columns). Given the complexity of Han et al.'s formulations, full details of their equations are avoided here and are presented in full in [5,6]. However, for the purposes of illustration, Fig. 1 shows Han et al.'s predicted residual stress versus strain curves for the concrete used in the current study (described in detail in the following sections), when confined within either circular or square CFS sections with the dimensions tested herein, with 5 mm steel wall thickness. These predictions were made for 70 MPa compressive strength concrete. It is noteworthy that Han et al.'s proposed stress–strain relations suggest the same confined compressive strength for concrete in both circular and square columns (which is known not to be the case [9]), however circular columns demonstrate a lower level of post-peak softening as a consequence of the superior lateral confinement in circular versus square CFS sections. Also shown in Fig. 1(c) are the Han et al.'s proposed bi-linear post-fire residual stress strain curves for steel.

Han and Huo [5] also propose equations for the RSI of unprotected CFS columns exposed various durations of the standard fire, based on a series of numerical parametric studies performed using a plane-sections equilibrium analysis similar to that suggested by Lie and Celikkol [10]. Again, the full details of the equations are avoided here but are given in the source publication [5]. It should be noted, however, that Han and Huo's equations are explicitly limited to section sizes greater than 200 mm, with no explanation of the rationale for this limitation. This is seems peculiar since Han and Huo's own testing, upon which their models are based, involved circular 108 mm Ø and square 100 × 100 mm sections which were reasonably predicted using their RSI equations. For the purposes of illustration, Fig. 2 shows the predicted RSIs for the CFS column geometries tested in the current study (described in the following section). Han and Huo's [5] predictive equations account for column size, shape, and slenderness, but take no account of steel wall thickness, concrete infill type or strength, steel yield strength, or non-standard heating regimes. They are also unable to treat the case of protected columns, making their utility marginal for post-fire assessment (where heating will have been

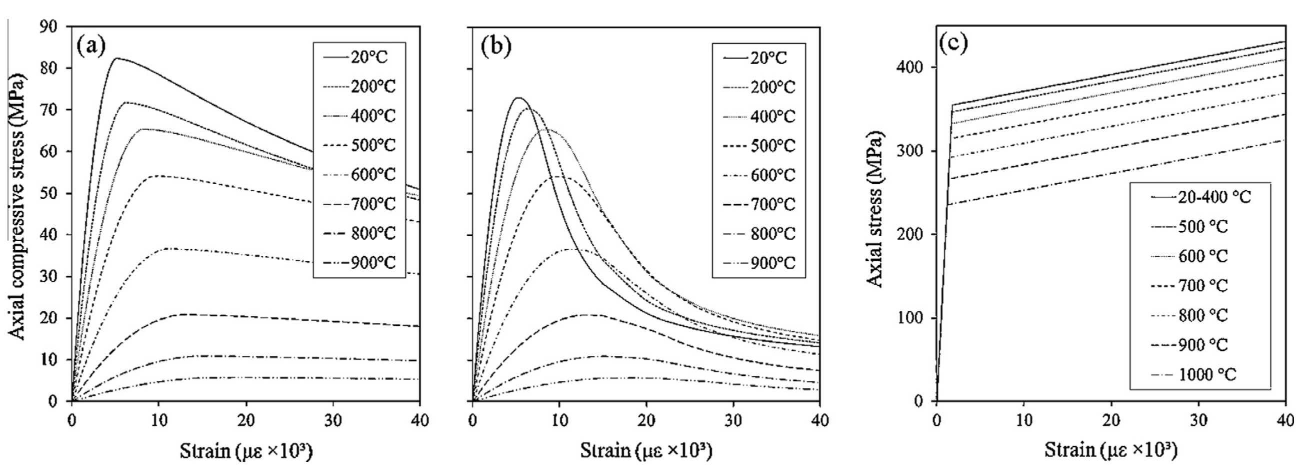

Fig. 1. Predicted [5,6] residual stress versus strain curves for the concrete used in the current study when confined in (a) circular or (b) square steel tubes; and (c) predicted stress versus strain response for steel.