Thin-Walled Structures 131 (2018) 192–204

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Full length article

Design of a continuous concrete filled steel tubular column in fire

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ARTICLE INFO

Keywords:
Fire resistance
Concrete-filled steel tubular columns
Design procedure
Fibre reinforced concrete

ABSTRACT

Concrete filled steel tubular (CFST) columns used in multi-storey buildings are generally designed as continuous members. The fire behaviour is predicted based on the results of experimental standard fire testing of CFST members where the same temperature is applied to the column over the full column height. Over the past 36 years, 238 experimental tests have been reported in the literature on CFST columns; different types of concrete infill have been considered: plain, steel fibre and bar reinforced concrete. In these tests, the columns were loaded axially under either concentric or eccentric load, and subjected to the standard ISO 834 fire or its equivalent in a furnace. This paper has focused on the in-depth analysis of behaviour of a continuous CFST columns in fire and provided a simple design procedure to calculate the axial capacity of the CFST columns at elevated temperature. The examples given in the later section gives a step by step design procedure for practicing engineers to calculate the axial capacity of both concentrically and eccentrically loaded CFST columns in fire.

1. Introduction

Engineers and building owners are becoming more aware of the benefits of using concrete filled steel tubular (CFST) columns, due to their combination of excellent stability during construction, high strength in service and clean lines for both appearance and durability. One of the most demanding loading conditions for multi-storey building design is the impact of severe fire. The columns play a critical role in ensuring the dependable behaviour of the building under severe fire attack. Design of these columns is based on the columns retaining their load carrying capacity for a specified time of exposure to Standard Fire conditions, known as a Fire Resistance Rating (FRR). During the design stage of the building, designers have to ensure column stability under compression or compression and bending for FRRs from 30 to 90 min typically, but up to a maximum of 300 min for firecells with very high fire load and limited ventilation, both of which generate high structural fire severity.

Design equations have been developed by various researchers [1–4] to calculate the design compression capacity of unprotected CFST columns in fire. However, some of these equations are too conservative for columns requiring FRR higher than 90 min, principally because they underestimate the contribution of the structural steel jacket at longer durations of fire exposure. The structural steel yield strength reduction factors given in AS/NZS 2327 [5] and Eurocode 4 Part 1–2 [6] were developed for bare structural steel sections; however, in the case of CFSTs, the structural steel and concrete act together to provide a composite resistance greater than that of the individual materials acting alone. The concrete core acts as a heat sink, keeping the steel jacket cooler than would be the case for a hollow bare steel section without concrete infill.

The proposed design equations presented in this paper and elaborated in a paper by the author [7,8] have been developed and validated against the results of 238 Standard Fire tests undertaken worldwide over the past 36 years (121 square, 104 circular and 13 rectangular). These also include laboratory tests conducted by the Authors [9,10]. Three different types of concrete infill have been used; plain, steel fibre and bar reinforced concrete.

These tests are for columns subjected to Standard Fire exposure. This means that, if a natural fire is being used for the design, the time equivalent must be determined from first principles for an insulated structural steel member to give the time of standard fire exposure required. The thickness of insulation used for this time equivalent determination should be such as to give the maximum temperature reached in the structural steel member in the natural fire at around 550–600 °C.

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https://doi.org/10.1016/j.tws.2018.07.001
Received 5 March 2018; Received in revised form 26 June 2018; Accepted 1 July 2018
0263-8231/

Notation

$A_{c,T}$	cross-sectional area of concrete
$A_{r,T}$	cross-sectional area of bar reinforcement
$A_{s,T}$	cross-sectional area of structural steel profile
A_m/V	section factor of structural member per unit length, calculated including the volume of the concrete core in determining V
$E_{c,sec,T}$	temperature dependent secant modulus of elasticity of concrete
$E_{r,T}$	temperature dependent modulus of elasticity of bar reinforcement
$E_{s,T}$	temperature dependent modulus of elasticity of structural steel
$E_{fi,Exp}$	design effect of actions in fire situation for laboratory experiment
$(EI)_{fi}$	effective flexural stiffness under fire conditions
$f_{c,T}$	compressive strength of concrete, at a temperature T
f_{sy}	characteristics yield strength of bar reinforcement
f_y	yield strength of the structural steel
$f_{sy,T}$	yield stress of bar reinforcement, at a temperature T
$f_{y,T}$	yield stress of structural steel, at a temperature T
$I_{c,T}$	temperature dependent second moment of area of concrete
$I_{r,T}$	temperature dependent second moment of area of bar reinforcement
$I_{s,T}$	temperature dependent second moment of area of structural steel
k_e	effective length factor
k_c	concrete strength reduction factor
k_{ec}	concrete strain corresponding to $f_{c,T}$
k_{esy}	bar reinforcement modulus of elasticity reduction factor
k_{ey}	structural steel modulus of elasticity reduction factor
k_{sy}	bar reinforcement reduction factor
k_y	structural steel reduction factor
$L_{e,T}$	buckling length of column in fire situation
$N_{c,fi,Rd}$	design section compression capacity at the fire limit state
$N_{fi,d}$	design value of the axial load under fire condition
$N_{fi,d,\varepsilon}$	design value of the eccentric axial load under fire condition
N_c	member capacity at ambient temperature
$N_{c,\varepsilon}$	member capacity for the eccentric axial load at ambient temperature
P_s	perimeter of structural steel section exposed to fire
R	Structural fire resistance
α_c	member slenderness reduction factor
T	Temperature
u_s	axis distance of bar reinforcement
λ_r	relative slenderness of column at room temperature
$\lambda_{r,T}$	relative slenderness of column in fire situation
η_{fi}	design load level in fire condition
φ_c, φ_s φ_r	design compression load in fire modification factor for concrete, structural steel and bar reinforcement
ϕ_c, ϕ_s and ϕ_r	capacity factor impacting a limit state for concrete, structural steel and bar reinforcement
e	distance of eccentricity

2. Experimental investigation

2.1. General

The fire tests were conducted in a furnace having dimensions of 2 m height × 1.5 m length × 1.5 m width, in accordance with EN 1364-1: 2012 [11]. The furnace temperature was controlled to match the ISO 834 [12] time-temperature curve. Fig. 1 shows the typical average furnace temperature to the ISO 834 fire curve for a typical test. Axial loads were applied for approximately 30 min before each fire test and were maintained throughout.

2.2. Test specimens

Table 1 gives a summary of test specimens; two different cross-sectional dimensions of square hollow section (SHS) were tested: 200 mm × 200 mm × 6 mm (P, F or R; 1–4) and 220 mm × 220 mm × 6 mm (P, F or R; 5–8).

The first letter for each specimen represents the type of concrete infill used (P = Plain, F = Steel Fibre and R = Rebar). All columns had a length of 3200 mm, with fire exposure only to the middle 2000 mm. It should be noted that the bottom 500 mm and the top 700 mm of the column were outside the furnace. Fig. 2 shows the column inside the furnace before the start of the experiment.

From Table 1 it can be seen that sixteen columns had fixed-pinned boundary conditions to allow for the load eccentricity to be applied in uni-axially. The pin-end boundary condition was provided through a ball (see Fig. 3). The fixed-fixed boundary was applied to the column by welding a boxed steel section having a depth of 200 mm to restrain the top of the column from rotational and translational movement; this was discussed in details by the authors [8,9,13].

Three types of concrete were used for the steel tube infill, namely: plain concrete; steel fibre reinforced concrete; and rebar reinforced concrete. Table 2 shows the mix used for the concrete infill. The specified compressive concrete cylindrical strength after 28 days was f_{ck} = 80 MPa. For the steel fibre reinforced concrete, Dramix hooked end steel fibre specification 5D 65/60BG were used. The length of the fibres were 60 mm, diameter was 0.9 mm and the dosage was 50 kg/m^3. For the rebar reinforced concrete infill, longitudinal reinforcement bars were tied using 6 mm diameter stirrups, shape code 51 according to BS 8666:2005 [14]. Fig. 4 show the arrangements of reinforcements inside the steel tubes before pouring of the concrete.

2.3. Experimental result

The structural fire resistance (R) of the columns are summarized in Table 3. The variation of axial deflection against time is shown in Fig. 2a–c for columns filled with plain concrete, rebar reinforced concrete and steel fibre reinforced concrete respectively; the structural fire resistance is the time from commencement of test until the load bearing capacity criterion specified in EN 1363-1 was achieved (corresponding to the limiting rate of vertical contraction). (Fig. 5)

As can be seen from Fig. 6, the initial longitudinal elongation of the

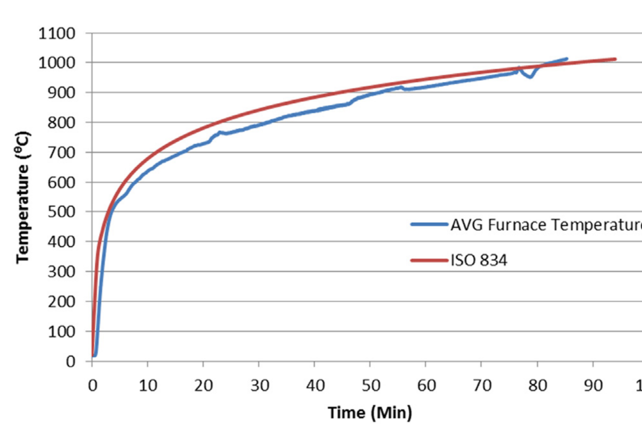

Fig. 1. Measured furnace temperature (ISO 834 shown for comparison).