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Simple design procedure for concrete filled steel tubular columns in fire

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ABSTRACT

Over the past 36 years, 238 experimental tests have been reported in the literature on concrete filled steel tubular (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. In this paper, the experimental tests reported in the literature have been used to propose simple design equations to determine the axial capacity of CFST columns. The proposed equations are compared with that presented in DR AS/NZS 2327 and Albero et al.; it is shown that the proposed equations are more accurate. The proposed design procedure has been used for the Auckland International Airport Phase 4 Pier B Extension Project's structural fire design, brief details of which are presented.

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 120 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 DR 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 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. Three different types of concrete infill have been used; plain, steel fibre and bar reinforced concrete. A comparison between the proposed equations, the published draft standard AS/NZS 2327 [5] and Albero et al. [2] equations is presented.

The proposed design equations are applicable to columns within the tested range, as given in the scope in Section 3. 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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Nomenclature

$A_{c,T}$	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
$\varphi_c,\varphi_s\ \varphi_r$	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. Existing design equations

Lie [7] developed a mathematical model to calculate the deformation, temperature and fire resistance of a concrete filled steel tubular column. The measured values from laboratory experiments were compared to the calculated values using an analytical approach; it was observed that the analytical approach could predict with acceptable accuracy the fire resistance of circular hollow steel columns filled with bar-reinforced concrete. For the evaluation of columns using the following parameters within stated ranges; column sizes, column lengths, applied load ratios and percentage of bar reinforcement, this analytical approach was also sufficient. Using this approach, the cross section of the column is divided into various sections (see Fig. 1).

Lie and Irwin [8] conducted further investigation to develop a mathematical model to predict the temperature, deformation and fire resistance ratings of a square CFST column filled with bar reinforced concrete. The model was validated against experimental test results obtained by Chabot and Lie [9]. Various parameters such as the column

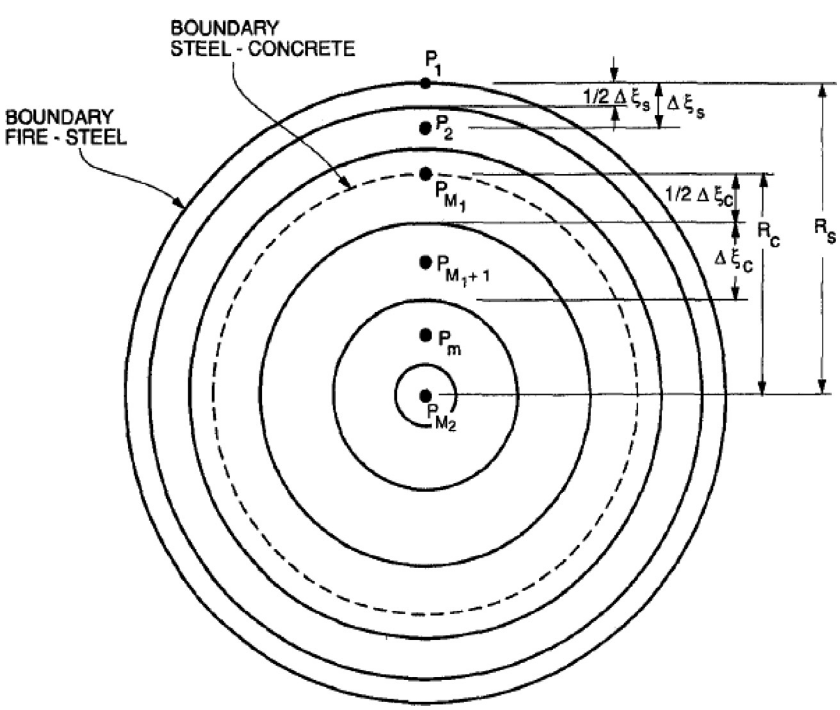

Fig. 1. Arrangement of layers in section of concrete-filled steel column.