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Post-fire behaviour of concrete-filled steel tubular column to axially and rotationally restrained steel beam joint

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ABSTRACT

This paper presents a numerical investigation on the post-fire behaviour of concrete filled steel tubular (CFST) column to restrained steel beam joint. An entire loading and fire phase, including ambient loading, heating with constant loads, cooling with constant loads and post-fire loading, was employed in the numerical analysis, and a finite element analysis (FEA) model was built to simulate the behaviour of CFST column to axially and rotationally restrained steel beam joints with external diaphragm connections under the entire loading and fire phase. For validation, the proposed modelling method was used to predict the test results of CFST columns and joints in fire and post-fire. The comparison demonstrates that the accuracy of the proposed FEA model is acceptable. Afterwards, the FEA model was used to analyse the mechanics characteristics of CFST column to restrained steel beam joints in the entire loading and fire phase. Based on the numerical analysis, the joint moment versus relative rotation angle relationship in the entire loading and fire phase was addressed, and the residual joint strength index and stiffness index were defined to evaluate the post-fire performance of joints. Finally, simplified calculating formulas were proposed to calculate the two indexes, which provide a simply and feasible method to evaluate the post-fire performance of external diaphragm joints in the CFST column - steel beam framed structure.

1. Introduction

Concrete filled steel tubular (CFST) structure has been found increasingly wide applications in the modern construction of new buildings. In a CFST framed structure, the internal forces or bending moments are transmitted between the beam and column components through the connections [1], and the failure of joints could cause the redistribution of internal forces and moments, thus induce to the failure of the overall structure. Therefore, it is reasonable to believe that joints are probably the most important part of a framed structure [2].

Fig. 1 shows a type of CFST column to steel beam joint, which adopts a typical rigid connection with external diaphragms. This type of CFST joint has been used in several high-rise buildings in China due to its advantages, such as better plastic deformation capacity, high stiffness and strength [1]. Design methods on CFST column to steel beam joints with external diaphragm connections were given by Zhao et al. [1], but it focused on designing the ultimate capacity of joints at ambient temperature. When the influence of a fire is considered, this type of CFST joint that was assumed rigid at ambient temperature could exhibit different characteristics in fire and post-fire conditions. Therefore, in terms of the fire safety design and post-fire repair to this type of CFST joint, extensive research work needs to be done.

The reported research on CFST column to steel beam joints with external diaphragm connections in fire is very limited, but studies on CFST joints with other types of connections still can provide useful references to this research field. Researchers in University of Manchester conducted a series of research to investigate the fire performance of CFST column to steel beam assemblies with different types of joints, including fin plate, reverse channel and T-stub joints [3–7], and some conclusions can be drawn: (1) different components located at the same region of the joint may have the same temperature [3]; (2) steel beams of specimens could reach very high deflections at the end of the fire test, and it can be concluded that the joint should be strong enough to enable the development of the beam's catenary action [4]; (3) the fire performance of reverse channels joints could be improved by changing the dimensions of some components [5,6]; (4) the risks for failure of the reverse channel joints using flexible endplate are very high during the cooling phase [7]. Huang et al. [8] reported an experimental investigation on reverse channel connections between steel beam and CFST column in fire. The test results indicated that this connection provided a significantly enhanced

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Nomenclature

a_b	thickness of beam fire protection
a_c	thickness of column fire protection
b_f	width of steel beam
b_{slab}	width of slab
D_c	outside diameter of CFST column section
f_{yb}	yield strength of steel beam
f_{cuc}	strength of core concrete in CFST column
f_{cus}	strength of concrete in RC slab
f_{ybs}	yield strength of steel bars in RC slab
f_{yc}	yield strength of steel tube
h	height of steel beam
H	height of column
K	line stiffness ratio of beam to column, given by $[(EI)_b/L]/[(EI)_c/H]$, where $(EI)_b$ and $(EI)_c$ are the flexural stiffness of beam and column, respectively
k_m	ultimate bending moment ratio of beam to column, given by M_{bu}/M_{cu}, where M_{bu} and M_{cu} are ultimate bending moments of beam and column at ambient temperature, respectively
k_{oa}	joint stiffness at ambient temperature
k_{op}	joint stiffness at post-fire phase
K	residual joint stiffness index
L	length of steel beam
L_{slab}	length of slab
m	beam load ratio
M	joint moment
M_b	internal moment of beam
M_{ua}	joint ultimate moment at ambient temperature
M_{up}	joint ultimate moment at post-fire phase
n	column load ratio
N	load
N_b	internal axial force of beam
N_c	internal axial force of column
N_{cr}	critical load
N_F	initial axial load on the column
N_o	initial load
N_u	axial compressive capacity of column at ambient temperature
q	uniform load on the beam
q_F	initial uniform load on the beam
q_u	ultimate capacity of steel beam with RC slab under uniform load at ambient temperature
Q	shear force of a shear connector
R	residual joint strength index
t	time
t_d	time that structural component temperature drops to ambient temperature
t_f	thickness of flange
t_h	heating time
t_o	heating time ratio
t_p	time that environmental temperature drops to ambient temperature
t_r	fire resistance of joint
t_s	thickness of steel tube
t_{slab}	thickness of slab
t_w	thickness of web
T	temperature or environmental temperature
T_h	environmental temperature corresponding to heating time
α_c	steel ratio of column, given by A_s/A_c, where A_s and A_c are the sectional areas of steel tube and concrete core, respectively
δ	slippage
δ_b	deflection of beam
Δ_c	axial deformation of column
Δ_i	lateral deformation of column
$\varepsilon_{cr,s}$	steel high-temperature creep
θ_r	relative rotation angle between column and beam

ductility compared to the flush endplate connection. Tan et al. [9] experimentally and theoretically investigated the fire performance of CFST column to reinforced concrete beam joints in fire, and it is found that the composite joint could be classified as semi-rigid in fire.

Since 2007, the authors of this paper were involved in a few research work on CFST column to steel beam joints with external diaphragm connections after exposure to fire. Han et al. [10] experimentally and theoretocally studied the cyclic performance of this type of joint after fire. It is found that the column lateral load carrying capacity and stiffness were reduced, but the connection ductility and energy dissipation were increased after fire. Huo et al. [11] conducted the post-fire test on repaired joints. After fire exposure, the steel beam and external diaphragms were replaced with new ones, and then the joints were tested under cyclically lateral loading. The results indicated that this repairing method would not deteriorate the ductility and energy dissipated ability of the retrofitted joints. The studies of Han et al. [10] and Huo et al. [11] concentrated on the post-fire performance of joints, but they ignored the influence of initial loads before fire exposure. Actually, for a real structure after exposed to fire, the structural components may experience an entire time (t) – environmental temperature (T) – load (N) path, as shown in Fig. 2, which can be divided into four phases:

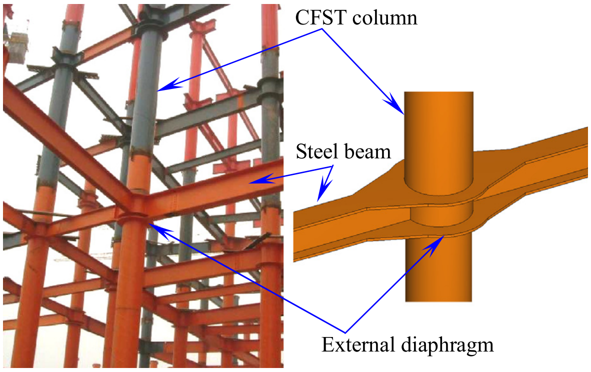

Fig. 1. CFST column to steel beam joint with external diaphragm connection.

(1) Ambient loading phase (AA′). Apply initial load (N_o) on the structural component at ambient temperature (T_o). The elapsed time of this phase is zero because the assumption that the structural component is already loaded to N_o before fire exposure is adopted;

(2) Heating phase (A′B′). Increase the environmental temperature and keep the initial load (N_o) constant;

(3) Cooling phase (B′C′D′). After the environmental temperature reaches the highest temperature (T_h) corresponding to the heating time (t_h), the environmental temperature starts to drop. At time t_p, the environmental temperature drops to the ambient temperature (T_o), and then, as the elapsing of time, the structural component temperature drops to the ambient