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Behavior of Concrete Filled Steel Tubular (CFST) triple-limb laced columns subjected to ISO 834 Standard Fire

*Z.Q. Cui[1], L.H. Han[2] and T.Y. Song[3]

[1], [2] Department of Civil Engineering, Tsinghua University, Beijing 100084, PR China
[3] Institute for Infrastructure Engineering, University of Western Sydney, Penrith, NSW 2751, Australia
[1] cuizq11@mails.tsinghua.edu.cn

ABSTRACT

Finite element analysis (FEA) model is established to study the behavior of concrete filled steel tubular (CFST) triple-limb laced columns subjected to ISO 834 standard fire. Some key mechanics characteristics on the fire performance of CFST triple-limb laced columns, including temperature distribution, failure mode, fire resistance, axial deformation and load distributions, are discussed based on the established FEA model. Finally, parametric analysis is performed to investigate the influence of various parameters, including slenderness ratio, load ratio, load eccentricity ratio, steel and concrete strength, on the fire resistance of CFST triple-limb laced columns, and the major influencing parameters are identified.

Keywords: Concrete filled steel tubular (CFST); Triple-limb laced columns; Fire resistance; Failure mode

1 INTRODUCTION

It is well known that steel-concrete composite structures combine the advantages of both steel and concrete, and thus exhibit excellent structural performance. Concrete filled steel tube (CFST), as one type of typical composite structure, has been widely used in high-rise buildings and underground public buildings (Han 2007).
CFST laced columns can provide a large cross sectional moment of inertia with a relatively small section dimension, and are more economical compared with the CFST single-limb columns. So they are often used as industrial building columns, as shown in Fig.1.
Due to the production demands, industrial buildings are usually used to store combustible gas, combustion-supporting gas, inflammables and explosives. Thus the industrial buildings demand a higher level of fire resistance or fire safety.
Extensive researches have been previously performed on the fire resistance of CFST single-limb columns, such as fire resistance tests (Lie and Chabot 1992, Han *et al*. 2003, Han *et al*. 2003), numerical simulations (Lie and Chabot 1990, Han 2001, Yang *et*

[1] Graduate Student
[2] Professor
[3] Doctor

al. 2008), and simplified calculation methods (Lie and Stringer 1994, Kodur 1999, Wang 2000, Han *et al*. 2003). However, the fire performance of CFST laced columns has seldom been researched. Previously studies on the CFST laced columns focused on the mechanics behavior at the ambient temperature, such as the ultimate bearing capacity (Chen and Ou 2007, Liao 2009).

Fig. 1 CFST laced columns used in real constructions (Han 2007)

Set against this background, a finite element analysis (FEA) model is established to investigate the fire performance of CFST triple-limb laced columns subjected to ISO 834 standard fire in this paper. Based on the established FEA model, fire resistance, temperature distribution, failure mode, axial deformation, and load distributions on each chord are discussed. Finally, parametric analysis is performed to investigate the influence of some important parameters on the fire resistance of CFST triple-limb laced columns.

2 FINITE ELEMENT ANALYSIS MODEL

A Finite Element Analysis (FEA) model is developed to simulate the temperature distribution and deformation of the composite columns in ISO 834 standard fire, and several test results are used to verify the accuracy of the FEA model.

2.1 Establishment of the FEA model

General commercial program ABAQUS software was adopted to establish the heat transfer analysis and mechanical analysis models. For the convenience of data share between the heat transfer model and mechanical model, the geometrical model and meshing were set to be exactly the same. Details of the FEA model are introduced below.

2.1.1 Heat transfer model

In the heat transfer analysis model, the initial temperature of the columns was set as 20 °C before exposed to fire, then ISO 834 standard fire (1999) was applied on the surface of the composite column.

The thermal properties of steel and concrete, including density, specific heat and thermal conductivity, were decided according to the formulas proposed by Lie and Denham (1993). The thermal properties had also been used by Han *et al*. (2003) to predict the temperature distributions of CFST columns, and the predicted results were acceptable when compared with the measured results.

The thermal resistance between steel tube and core concrete was ignored and the constraint type “tie” in ABAQUS was used to simulate the heat transfer between steel tube and core concrete.

The whole column was assumed to be exposed to fire completely, and the effects of heat convection and radiation were considered as boundary conditions, which were defined according to the recommendations in ECCS-Technical Committee (1988). A constant convective heat transfer coefficient 25 W/(m^2K) was assumed for the fire exposed surfaces and the corresponding resultant emissivity was taken as 0.6. The Stefan-Boltzmann constant and the absolute zero temperature were taken as 5.67×10^{-8} W/ (m^{2}°C^{4}) and -273 °C respectively.

The heat transfer 8-node brick elements (DC3D8) were used for core concrete and endplate, and the 4-node quadrilateral shell elements (DS4) were used for chord tubes and lacing tubes.

2.1.2 Mechanical model

In the mechanical model, three steps including initial step, loading steep and heating step were set. Boundary conditions and interaction between steel tube and core concrete were defined in the initial step, and load was applied in the loading step. In the heating step, the thermal analysis results were input into the mechanical model to simulate the heating process.

The stress-strain curves for steel proposed by Lie and Denham (1993) were adopted herein to describe the constitutive behavior of chord tubes and lacing tubes at high temperatures. This model had been adopted by Han (2007) to research the fire resistance of CFST columns, and the predicted results had a good agreement with the test results. Therefore, here, the same tress-strain model was adopted. The initial modulus of elasticity was determined by the slopes of the stress versus strain curves at high temperatures. The Poisson’s ratio was 0.3. The effects of thermal expansion and steel creep at elevated temperatures were considered through a subroutine proposed by Song (2010).

For the concrete, the concrete damaged plasticity model was used in the analysis. The equivalent uniaxial stress-train relationships of concrete during heating phase proposed by Song *et al*. (2010) were adopted, and the accuracy of the concrete stress-strain curves at high temperatures had been validated by comparing with the CFST stub column test results (Song *et al*. 2010) and CFST joint test results (Song 2010). The initial modulus of elasticity was obtained by the slopes of the stress-strain curves, and the Poisson’s ratio for concrete remained at 0.2 unchanged until 150 °C, reduced to 50% of its ambient value at 400 °C and became zero at 1200 °C (Izzuddin 2002). A user subroutine programmed by Tan (2012) was adopted to estimate the effects of thermal expansion, concrete creep and transient thermal strain at high temperatures.

For the interface between steel tube and core concrete, hard contact in the normal direction and Coulomb friction model in the tangential direction were adopted. The friction coefficient was set as 0.25 (Han 2007). The connections between chord tubes and lacing tubes were assumed to be rigid.

Fig. 2 shows the boundary conditions and element division of the mechanical model. The bottom endplate of the column was restrained against the displacements in three directions and rotations in Y and Z directions. The top endplate was restrained against the displacements in X and Y directions and rotations in Y and Z directions. Vertical load (*N*) was applied on the top endplate. Load eccentricity of *H*/1000 (*H* is the height of the column) was adopted to consider the influence of initial imperfection. 4-node shell elements (S4R) were used to model the chord tubes and lacing tubes, and 8-node brick elements (C3D8R) were used to model the core concrete and endplates.

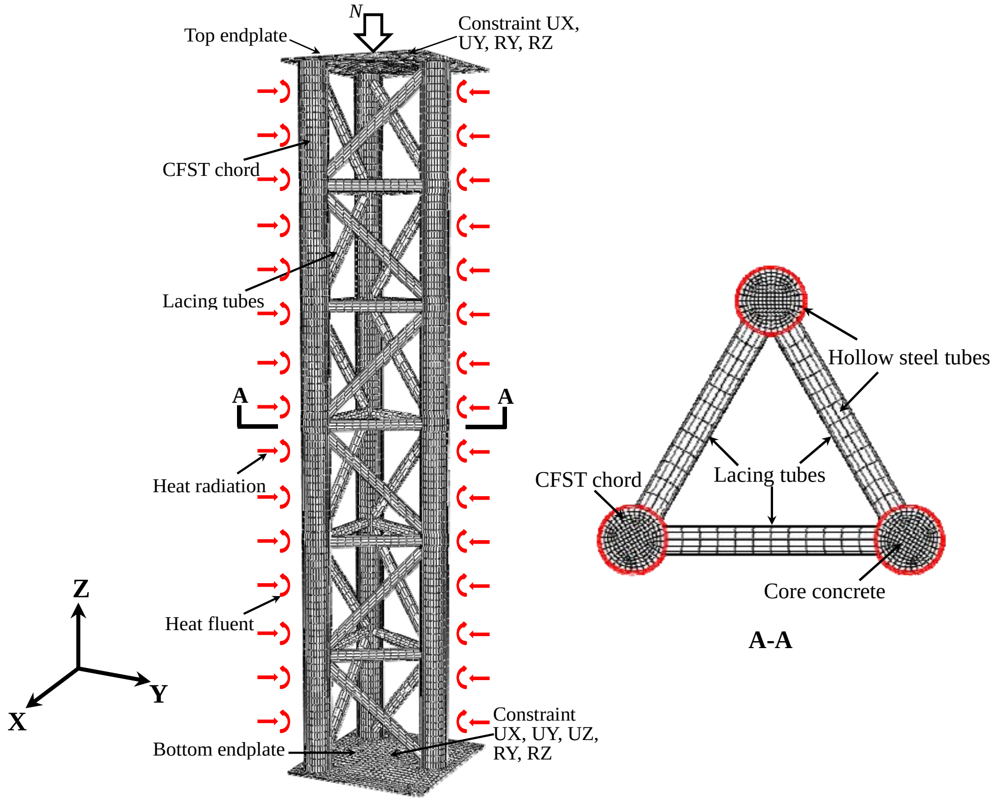
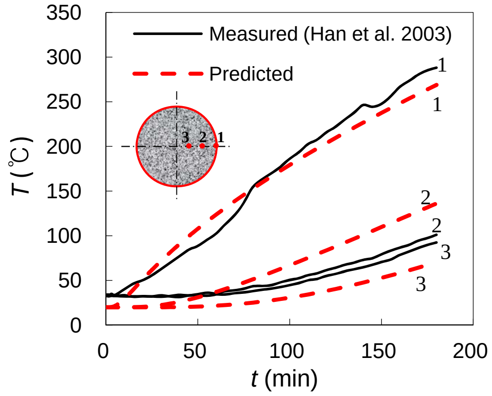


Fig. 2 Meshing and boundary conditions of the FEA model

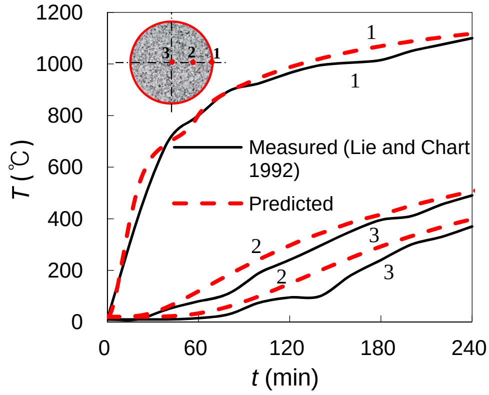
2.2 Verification of the FEA model

For lack of the test data on CFST laced columns in fire, 8 test results on temperature distributions of CFST members (Lie and Chart 1992, Han *et al*. 2003), 10 test results on fire resistance of CFST columns (Han *et al*. 2003) and 7 test results on ultimate bearing capacity of CFST triple-limb laced columns (Liao 2009) were adopted to validate the FEA model.

Fig. 3 compares the temperature (*T*) versus time (*t*) curves between measured and predicted results of CFST members. It can be found that the predicted temperature (*T*) – time (*t*) curves match well with the tested results reported by Lie and Chart (1992) and Han *et al* (2003), where *D* is the diameter of the CFST member.



(a) *D* = 400 mm, with fire protection



(b) *D* = 355.6 mm, without fire protection

Fig. 3 Comparison between measured and predicted temperature (*T*) - time (*t*) curves

Figs. 4-6 show the comparisons of the tested and predicted fire resistance, failure mode and axial deformation (*Δ*) versus time (*t*) curves of CFST columns, respectively. It can be found that the predicted results have a good agree with the tested results.

The comparisons of tested and predicted ultimate bearing capacity, failure mode, load (*N*) versus axial deformation (*Δ*) curves and load ratio (N/N_u) versus lateral displacement (*δ*) curves are shown in Figs. 7-10, respectively. It can be observed that the predicted results show reasonable agreement with experimental results.

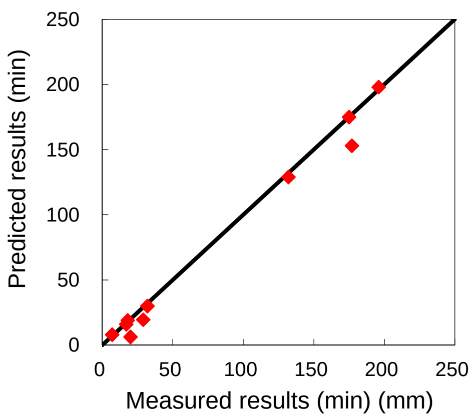
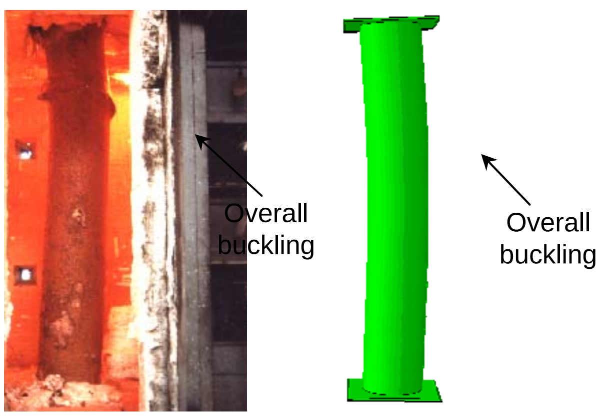
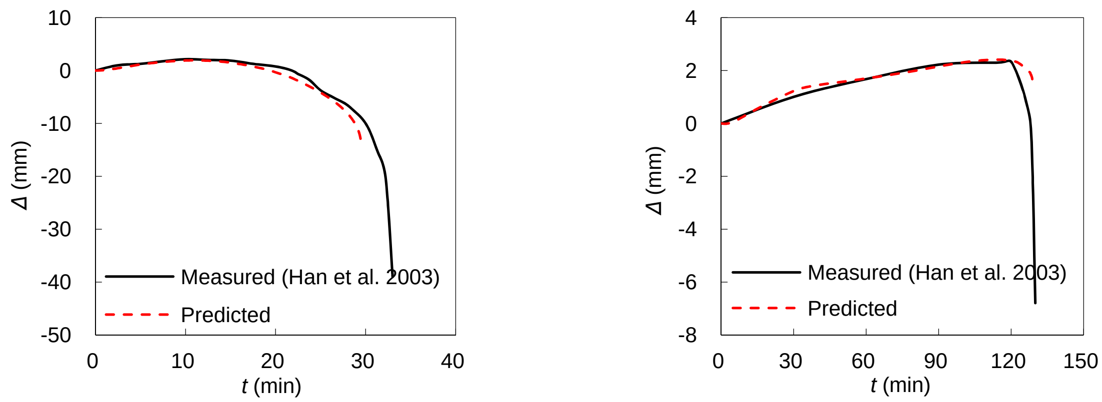


Fig. 4 Comparison of fire resistance



(a) Tested (b) Predicted

Fig. 5 Comparison of failure mode



(a) λ = 31.9, without fire protection (b) λ = 69.6, with fire protection

Fig. 6 Comparison of axial deformation (Δ) - time (t) curves

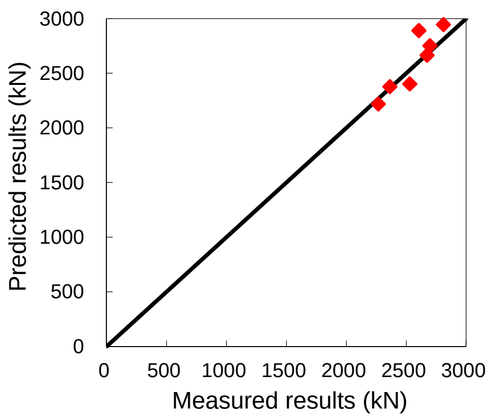
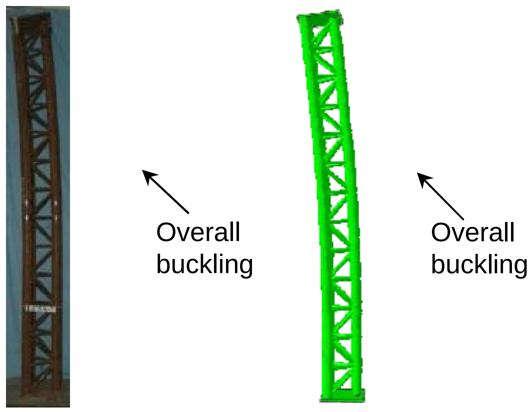


Fig. 7 Comparison of ultimate bearing capacity



(a) Tested (b) Predicted

Fig. 8 Comparison of failure mode

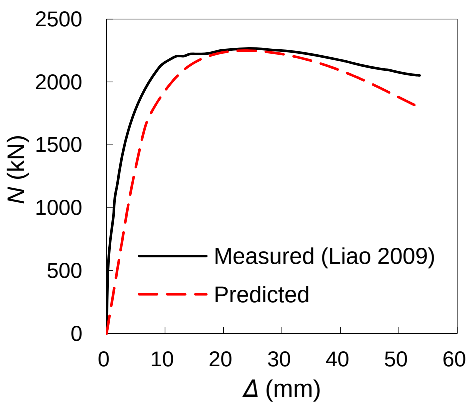


Fig. 9 Load (N) - axial deformation (Δ) curves

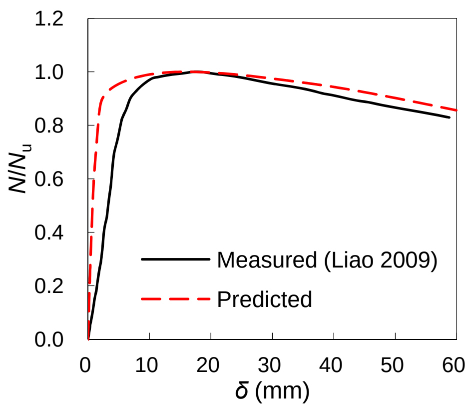


Fig. 10 Load ratio (N/N_u) - lateral displacement (δ) curves

3 ANALYTICAL BEHAVIOR

Based on the proposed modeling method, a FEA model of full-size CFST triple-limb laced column is built to investigate the fire performance of this type of composite column in ISO 834 standard fire. The chord tubes are circular cross section of 400 mm × 12 mm with a height (*H*) of 9600 mm. The distance between the two axial line of the chords is 1600 mm. The lacing tubes are also circular cross section of 160 mm × 5 mm. The tubes adopt carbon steel with a yield strength of 345 MPa, and the concrete strength is 30 MPa. The load ratio ($n=N/N_u$) is 0.3, where N_u is the ultimate bearing capacity of the composite column at ambient temperature and can be calculated by the FEA model. On the basis of the determined calculation conditions, the temperature distribution, failure mode, axial deformation and load distributions are discussed.

3.1 Temperature distribution

Fig. 11 shows the temperature distribution of the column cross section in ISO 834 standard fire. Comparing the temperatures of points 1 and 2, it can be found that the temperature of lacing tubes increases more quickly than chord tubes, which is due to the fact that core concrete absorbs the heat transferred from the chord tubes. Because the thermal resistance between steel tube and core concrete is ignored, the temperatures of points 2 and 3 are almost the same. However, the temperature difference between point 3 and 4 is large, which is due to the fact that the core concrete has a quite small thermal conductivity and transfers heat slowly.

3.2 Failure mode

The CFST triple-limb laced column achieves the fire resistance at 34 min under the determined calculation condition. Fig. 12 shows the typical failure mode of the CFST triple-limb laced column, where “Mises” represents Mises stress of steel tubes in MPa. It can be found that the three main chords can work together very well under the fire conditions, and no local buckling occurs at the chords and lacing tubes. The composite column fails due to the overall buckling. It also can be seen that the maximum Mises stress locates at main chords due to the influence of vertical load, and the Mises stress of chord tubes around the joint zones is larger than other parts of the chord tubes due to the interaction between the lacing tubes and CFST chords.

3.3 Axial deformation

Fig. 13 shows the typical axial deformation (Δ) versus time (t) curve of CFST triple-limb laced column in fire, which can be divided into three phases:

(1) Expansion phase (AB). At the beginning of heating, the column expands due to the influence of the thermal expansion of materials as the increasing of temperature.

(2) Contraction phase (BC). As the increasing of steel tube temperature, the bearing capacity of the column decreases due to the degradation of steel properties at high temperatures, and the column contraction occurs.

(3) Failure phase (CD). When the steel temperature exceeds 600 °C, the compression

deformation increases rapidly and the column reaches the fire resistance and fails.

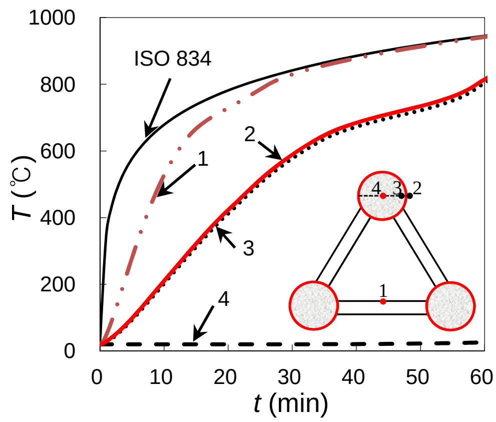


Fig. 11 Temperature (T) – time (t) curves

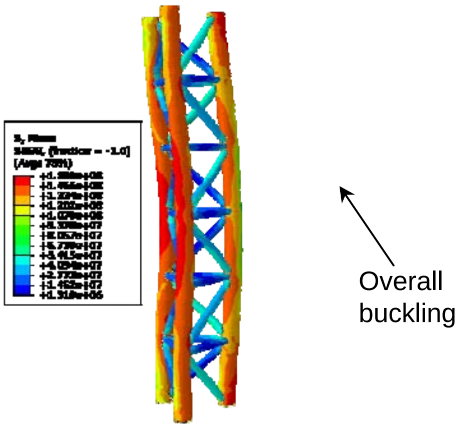


Fig. 12 Typical failure mode

3.4 Load distribution on each chord

Applied external load on the CFST triple-limb laced column is borne by the three CFST main chords and lacing tubes together. In the fire condition, the load distributions on each main chord vary with the temperature variation. Fig. 14 shows the load component borne by each chord (N) versus time (t) curves corresponding to the three CFST chords. It can be found that the load distributions on chords C2 and C3 are close to each other, but they differ from that on chord C1. It can be explained by the fact that chords C2 and C3 locate at the same side of the initial eccentric load, but chord C1 locates at the other side. At the beginning of fire, the similar tendency can be found in the three chords, but as the increasing of temperature, at the end of fire exposure, the influence of eccentric load is enhanced, which leads to the difference between chord C1 and the others.

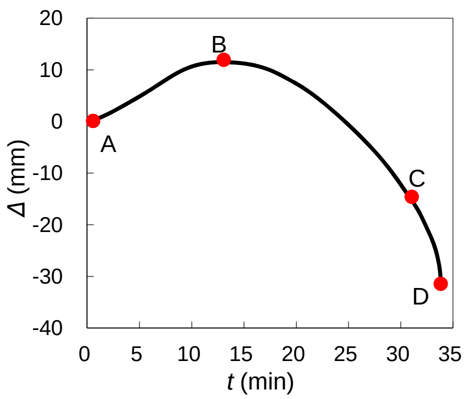


Fig. 13 Axial deformation (Δ) - time (t) curve

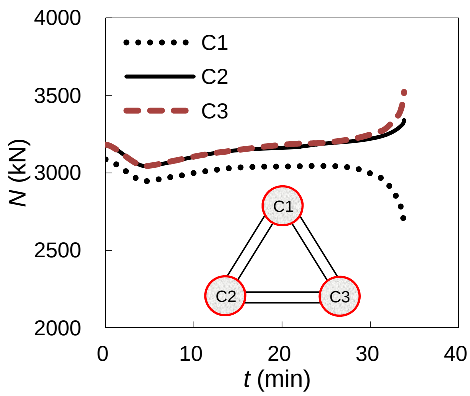
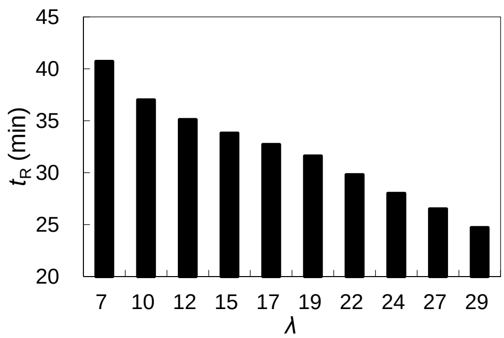


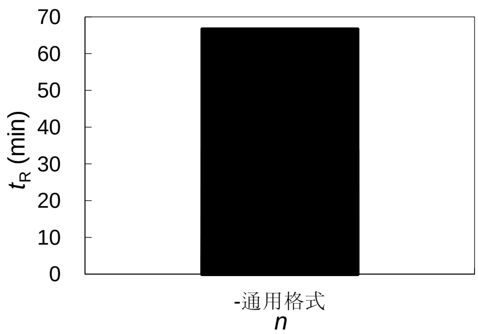
Fig. 14 Load component (N) - time (t) curves of chords

4 PARAMETRIC STUDY

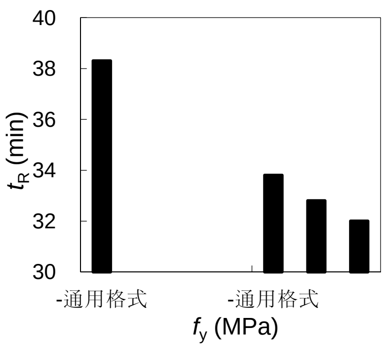
Possible influencing parameters, including slenderness ratio (λ), load ratio (n), load eccentricity ratio (e), steel strength (f_y) and concrete strength (f_{cu}) are chosen to investigate their influence on the fire resistance of CFST triple-limb laced columns. The parameter ranges are $\lambda = 7 \sim 29$, $n = 0.15 \sim 0.9$, $e = 0.001 \sim 0.3$, $f_y = 235 \sim 420$ MPa and $f_{cu} = 30 \sim 80$ MPa. Fig. 15 shows the predicted fire resistance corresponding to the different parameters. It can be concluded that:
(1) Load ratio and slenderness ratio, as shown in Figs. 15(a) and (b), have a significant influence on the fire resistance of the composite column. The results indicate a clear tendency that the fire resistance of the column decreases as the increasing of slenderness ratio and load ratio. This is because with the increasing of column height or applied load, the second order moment in the column increases, which leads to the column occurs overall flexural buckling more easily.



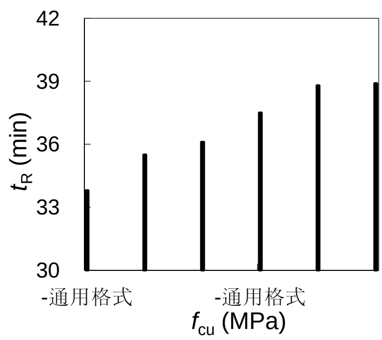
(a) Slenderness raito (λ)



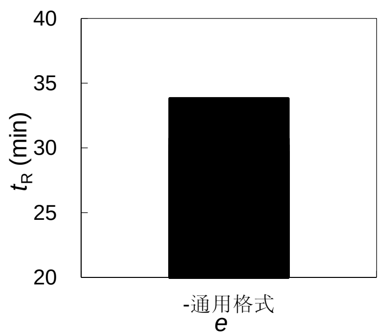
(b) Load raito (n)



(c) Steel strength (f_y)



(d) Concrete strength (f_{cu})



(e) Load eccentricity ratio (e)

Fig. 15 Fire resistance of CFST triple-limb laced columns corresponding to various parameters

(2) From Figs. 15 (c) and (d), it can be found that steel and concrete strength have a moderate effect on the fire resistance. As the increasing of steel yield strength or decreasing of concrete strength, the fire resistance of column decreases. It's attributed to the fact that the outer steel tubes expose to the fire directly, and the material properties degradation of steel tubes is more serious than that of concrete in fire. When the same load ratio is adopted, a higher steel strength or lower concrete strength means the steel tubes bear more external load, and it also means more bearing capacity loss in fire, which leads to the decrease of the fire resistance.
(3) The load eccentricity ratio has minor influence on the fire resistance of the column, as shown in Fig. 15(e). This may be explained by the fact that the ultimate bearing capacity of the column decreases with the increase of load eccentricity, and the load applied on the column decreases when the same load ratio is adopted. Thus the influence of the second order moment in the column is not enhanced as the increasing of load eccentricity.

5 CONCLUSIONS

The following conclusions can be drawn regarding the fire performance of CFST triple-limb laced columns based on the current research work:
(1) A FEA model was proposed to simulate the behavior of CFST triple-limb laced columns in fire, and the accuracy of the FEA model was validated against the relevant test data.
(2) Based on the proposed FEA model, the fire performance of a CFST triple-limb laced column, which has a dimension near to real constructions, was studied, and it's found all the three CFST chords could work together perfectly in fire condition, and an overall flexural failure was observed.
(3) Parametric study indicated that load ratio and slenderness ratio affected the fire resistance of the composite column significantly, and the influences of steel and concrete strength were moderate.

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