STRUCTURAL FIRE DESIGN OF CONCRETE FILLED STEEL TUBULAR COLUMNS AND A PROPOSAL OF A NEW METHOD TO AS/NZS 2327

Kingsley Ukanwa[1]; Charles Clifton[2]; Jonny Papa[1]

[1]Built Environment, Aurecon, New Zealand
[2]Department of Civil & Environmental Engineering, The University of Auckland, New Zealand

SUMMARY

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 column in fire and provides a simple design procedure to calculate the axial capacity of the CFST columns at elevated temperature. The examples given in the later section provide a step by step design procedure for practicing engineers to calculate the axial capacity of both concentrically and eccentrically loaded CFST columns in fire.

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 need to ensure column stability under compression or, combined compression and bending for FRRs from 30 to 90 minutes typically, but up to a maximum of 300 minutes.

Design equations have been developed by various researchers (Albero et al., 2016; Espinos, Romero, & Hospitaler, 2012; V. Kodur & Raut, 2009; R. Wang, Zhang, & Li, 2016) 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 minutes, 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 (AS/NZS 2327, 2017) and Eurocode 4 Part 1-2 (CEN, 2005b) 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 authors(K. Ukanwa, Clifton, Lim, Hicks, & Sharma, 2017; K. U. Ukanwa, Clifton, Lim, Hicks, Sharma, & Abu, 2018b) have been developed and validated against the results of 238 Standard Fire tests undertaken worldwide over the past 36 years. These also include laboratory tests conducted by the Authors (K. U. Ukanwa et al., 2017a; K. U. Ukanwa et al., 2017b). Three different types of concrete infill have been used; plain, steel fibre and bar reinforced concrete.

EXISTING DESIGN EQUATIONS

Lie (Lie, 1994) 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.

Kodur (V. Kodur, 1999), through various experimental and numerical studies, developed a simplified design equation to calculate the fire resistance ratings of a CFST column, as given in equation (1).

$$R = f \times \frac{(f_c' + 20)}{(KL - 1000)} \times D^2 \times \frac{\sqrt{D}}{C} \tag{1}$$

Where R is the fire resistance time in minutes; f_c' is the specified 28 day concrete strength in MPa; D is the outside diameter or width of the column in mm; C is the applied load in kN; K is the effective length factor as per CAN/CSA-S16.1-M89 (Canadian Standard Association, 2001) standard; L is the unsupported length of the column in mm; f is a parameter to account for the type of concrete filling (Plain concrete, Bar reinforced Concrete and Steel fibre reinforced concrete), the type of aggregate used (carbonate or siliceous), the percentage of reinforcement, the thickness of concrete cover, and the cross-sectional shape of the hollow steel section column (circular or square).

Espinos *et al*. (Espinos et al., 2012) reviewed the current design guidelines available worldwide for calculating the fire resistance of CFST columns, with the recommendations focusing on the Eurocode 4 part 1-2 (CEN, 2005b) design approach. The aim was to demonstrate a new method for calculating the fire resistance of axially loaded unreinforced concrete filled circular hollow section columns. In order to determine the important parameters which affect the fire behaviour of CFST columns, a parametric study using validated numerical models was required. For the concrete model, it was assessed that the flexural stiffness reduction coefficient can be taken as 0.8 to account for the thermal stresses.

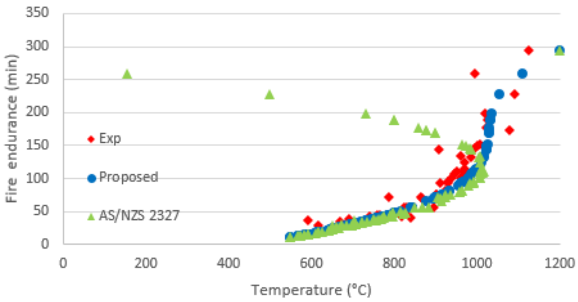
The structural steel tube required a more conservative value for its reduction coefficient. The method proposed was validated against laboratory experiments and it was observed that the equations can predict the axial capacity of circular columns filled with un-reinforced concrete loaded concentrically. The proposed equations were further extended by Espinos (Espinos, Romero, & Hospitaler, 2013) to cover reinforced concrete infill and elliptical CFST columns.

More recently, Albero *et al*. (Albero et al., 2016) conducted a parametric study using numerical models developed by Universitat Politècnica de València (UPV) and Centre Technique Industriel de la Construction Métallique (CTICM). The numerical study was validated against laboratory experiments conducted within the FRISCC (Fire Resistance of Innovative and Slender Concrete Filled Tubular Composite Columns) project in Europe. The new method was used in calculating the fire resistance time of unprotected CFST columns subjected to loads under a standard fire condition. The new method also allows for displacement to occur between the concrete core and the outer structural steel tube and the axial capacity of the column is thereby calculated using the methods given in section 6.7.3 of Eurocode 4 Part 1-1 (CEN, 2005a) but, taking into account the equivalent temperatures.

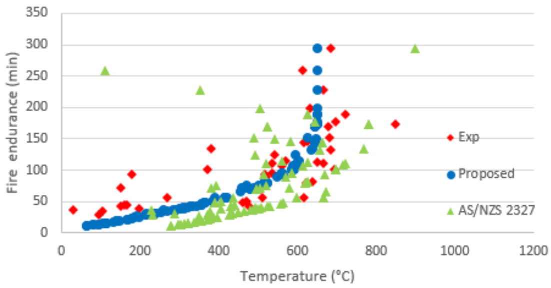
DEVELOPMENT OF NEW DESIGN EQUATIONS

The proposed design procedure in this paper were developed using laboratory experiments carried out on CFST columns loaded axially (concentrically and eccentrically) by various researchers (Chabot & Lie, 1992; Espinos, Romero, Serra, & Hospitaler, 2015; Grimault, 1980; Han, Yang, & Xu, 2003; Hong & Varma, 2009; V. K. R. Kodur & Latour, 2005; V. Kodur & Lie, 1996; V. Kodur, 1998; Lie, 1994; Lie & Irwin, 1995; Moliner, Espinos, Romero, & Hospitaler, 2013; Myllymäki, Lie, & Chabot, 1994; Romero, Moliner, Espinos, Ibañez, & Hospitaler, 2011; K. Wang & Young, 2013) over the past 36 years, including those of the authors (K. U. Ukanwa et al., 2017a; K. U. Ukanwa et al., 2017b). The proposed procedure follows the same steps as that given in EN 1994-1-1 (CEN, 2005a) and AS/NZS 2327 (AS/NZS 2327, 2017) for the design of CFST columns at ambient temperature, but utilises strength reduction factors that are dependent on member temperature and introduces a modification factor dependent on the structural fire rating, distance of eccentricity (e) and section factor of the column. These modifications have been made to improve the accuracy of the equations and reduce variability in relation to the experimental tests. The condition of application and step by step design approach including equations can be taken from the paper published by the authors(K. Ukanwa et al., 2017; K. U. Ukanwa et al., 2018b).

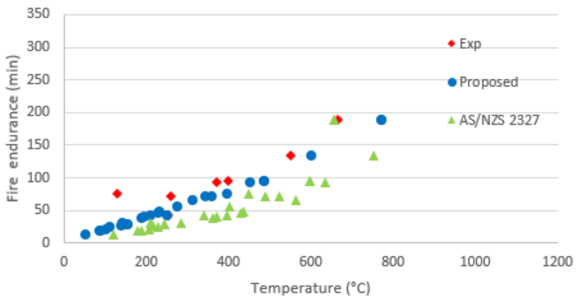
This section presents the key factors used in calculating the axial capacity of CFST columns when using the proposed approach. The evaluation of design temperatures for the cross-section components (concrete, structural steel and bar reinforcement) is presented in this Section. The design temperatures of the components are a key factor which influence the axial capacity of the column in fire. The concrete temperature is taken as the average temperature across the whole concrete section when divided into various sections. It should be noted that the structural steel temperature is constant regardless of the steel tube thickness, due to the high conductivity of steel and the heat sink of the concrete; the bar reinforcement temperature is dependent on the concrete cover. Figure 1 a-f shows a comparison between the laboratory experiments, AS/NZS 2327 and the proposed equation for the temperatures for structural steel section, concrete core and reinforcing bars.



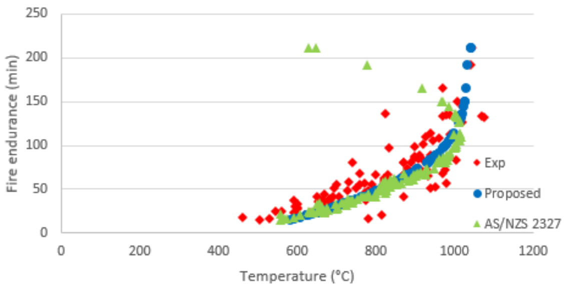
1a: Temperature of steel section for circular column



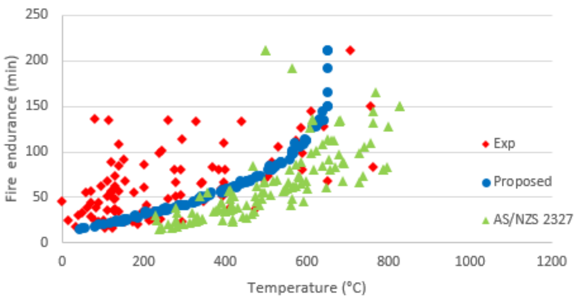
1b: Effective temperature of concrete core for circular column



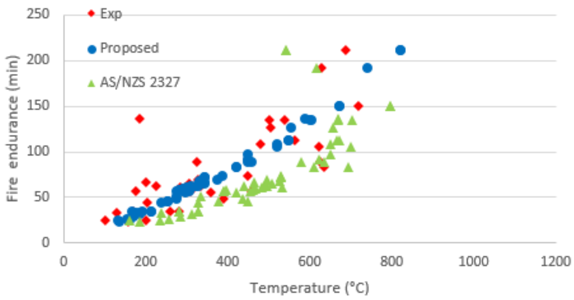
1c: Temperature of reinforcing bars for circular column



1: Temperature of steel section for square column



1: Effective temperature of concrete core for square column



1: Temperature of reinforcing bars for square column

Figure 1: Comparison of the temperature values from the design procedure against the experimentally recorded ones.

When calculating the design compression resistance of the composite cross-section, modification factors for the capacities of concrete, structural steel and bar reinforcement are applied. The thickness factor (A_s/P_s) is the ratio of the area of the structural steel member (not including the concrete core) to the perimeter of the structural steel exposed to fire, which is used to calculate the modification factor for the design compression load in fire. The current design guide (AS/NZS 2327, 2017) is conservative for columns developing higher structural fire resistance, because the structural steel, concrete and bar reinforcement reduction factors at elevated temperatures have been developed for individual members and not for composite members. For a composite member, the structural steel will be prevented from buckling inwardly due to the concrete infill. The concrete will also have a higher compressive strength because it is confined by the structural steel. The modification factors developed herein account for the composite action of the structural steel tube and concrete core for varying structural fire ratings. It was also observed that the AS/NZS 2327 provisions are too conservative for columns having R values greater than 90 minutes.

To determine the conservativeness of the proposed equations, the compression load used in the experiments has been divided by the axial capacity obtained using the proposed equations, it is observed that the proposed equations yield results that are more conservative (where Exp/Cal > 1) and accurate (COV closer to zero) while many of the AS/NZS 2327 results are either too conservative or in some cases un-conservative. This was due to the equations developed by the authors (Espinos et al., 2012) only being applicable to un-reinforced circular CFST columns. The various parameters which show the accuracy of the design procedure when compared to AS/NZS 2327 is given in Tables 1 a-c and shown graphically in Figure 2 a-c when the columns are safe or unsafe.

Table 1: Comparison of design provisions with experimental test

Section Type	Circular		Square		Rectangular	
Comparison	Proposed	AS/NZS	Proposed	AS/NZS	Proposed	AS/NZS
Average	1.56	1.12	1.52	1.32	2.83	1.94
Standard Deviation	0.51	0.60	0.43	0.89	1.57	1.18
Coefficient of Variation	0.33	0.54	0.28	0.68	0.56	0.61

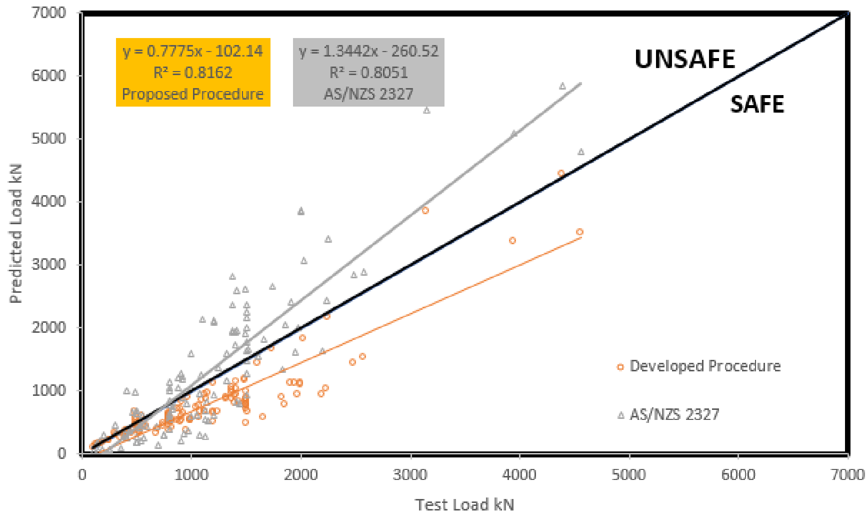


Figure 2a: Square Columns

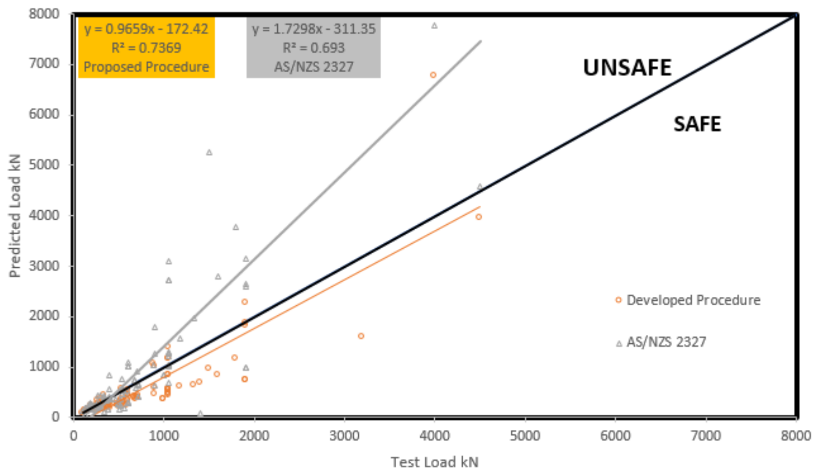


Figure 2b: Circular Columns

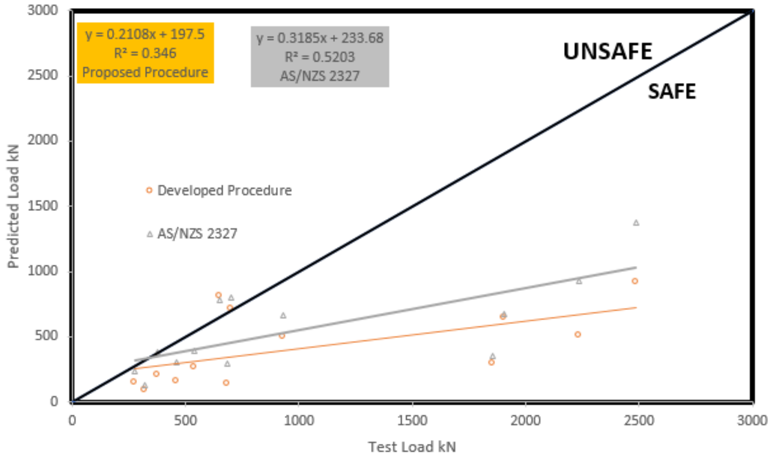


Figure 2c: Rectangular Columns

Figure 2: Comparison of design axial capacity of the column between proposed procedure, AS/NZS 2327 and experimental test

DESIGN EXAMPLE

The design example presented is taken from a laboratory fire test carried out by Lie & Irwin (Lie & Irwin, 1995) on a square CFST column filled with bar reinforced concrete and all equations are taken from the paper published by the authors (K. U. Ukanwa et al., 2018b).The test data used for the calculation are given below:

R = 150 Minutes
Column Length = 3.81m
Fixed – Fixed "k_e" = 0.7
Square hollow section = 203 mm x 203 mm x 6.35 mm
Yield strength of structural steel, f_y = 350 MPa
Compressive strength of concrete, f_c = 47 MPa
Yield strength of reinforcing bars, f_{sy} = 400 MPa
Diameter of reinforcing bars = 16 mm
Number of reinforcing bars = 4
Axis distance of reinforcing bar, u_s = 23 mm

Step 1: Design temperatures in the column section

$$T_s = 448.573 + 9.734\times150 - 0.0552\times150^2 + 0.000106\times150^3 = 1024°C$$

$$T_c = -112.35 + 13.194\times150 - 0.0778\times150^2 + 0.0001654\times150^3 - 4.101\times6.15 = 649°C$$

$$T_r = 28.09 + 5.728\times150 - 0.0092\times150^2 - 0.0003851\times23^{3.186} = 672°C$$

Step 2: Modification factors for the design capacity of the column cross-section

Section factor = $\frac{A_m}{V} = \frac{(2\times(203+203))\times1000}{203\times203} = 19.7m^{-1}$

Thickness factor = $\frac{A_s}{P_s} = \frac{4995}{203 \times 4} = 6.15mm$

$a = 1.11 - 0.04 \times 6.15 = 0.864$

$b = -0.17 + 0.04 \times 19.7 = 0.618$

$c = \left(6.15/19.7 - 0\right) = 0.31$

$\varphi_c = (0.02945 \times 150^{0.864}) \times 0.86 \times 0.618 = 1.19$

$\varphi_s = (0.02793 \times 150^{0.883}) \times 0.864 \times 0.618 = 1.24$ Therefore, use 1.2

$\varphi_r = (0.02257 \times 150^{0.889}) \times 0.864 \times 0.618 = 1.04$

Step 3: Design compression capacity at the fire limit state

$$\mathrm{N}_{\mathrm{c},fi,\mathrm{Rd}} = \varphi_c \times A_c \times f_{c,T} + \varphi_s \times A_s \times f_{y,T} + \varphi_r \times A_r \times f_{sy,T}$$

$k_c(649°C) = 0.376$

$k_y(1024°C) = 0.035$

$k_{sy}(672°C) = 0.285$

$$\mathrm{N}_{\mathrm{c},fi,\mathrm{Rd}} = 1.19 \times 35410mm^2 \times 47MPa \times 0.376 + 1.2 \times 4995mm^2 \times 370MPa \times 0.035$$
$$+1.04 \times 804mm^2 \times 400MPa \times 0.285 = 916kN$$

Step 4: Effective flexural stiffness in fire

$$(EI)_{fi} = E_{c,Sec,T} \times I_{c,T} + E_{s,T} \times I_{s,T} + E_{r,T} \times I_{r,T}$$

$k_{ec}(649°C) = 0.025$

$k_{ey}(1024°C) = 0.04$

$k_{esy}(672°C) = 0.169$

$$(EI)_{fi} = ((47MPa \times 0.376)/0.025) \times 105978701mm^4 + 200000MPa \times 0.04 \times 32226781mm^4$$
$$+200000MPa \times 0.169 \times 3309658mm^4 = 442kNm^2$$

Step 5: Euler buckling load at elevated temperature

$$N_{f,omb} = \frac{\pi^2 EI_{fi}}{L_{e,T}^2}$$

$$N_{f,omb} = \frac{\left(\frac{22}{7}\right)^2 \times 442 \times 10^9\, Nmm^2}{(0.7 \times 3810mm)^2} = 613kN$$

Step 6: Relative slenderness in fire

$$\lambda_{r,T} = \sqrt{\frac{N_{c,fi,Rd}}{N_{f,omb}}}$$

$$\lambda_{r,T} = \sqrt{\frac{916}{613}} = 1.22$$

Therefore,

$$\lambda_{\eta,T} = \lambda_{r,T} \times 90 = 110$$

Reduction factor "α_c" = 0.48, accounting for the column slenderness, is taken from Table 6.3.3(2) of NZS 3404 using 0 as the value of α_b as given in subsection 3.3.6 (K. U. Ukanwa, Clifton, Lim, Hicks, Sharma, & Abu, 2018a).

Step 7: Design load in fire

$$N_{fi,d} = \alpha_c \times N_{c,fi,Rd}$$

$$0.48 \times 916 = 437kN$$

Using the proposed design procedure, the column axial capacity is 437 kN for 150 minutes FRR. However, during the standard fire test, the column was subjected to an axial load of 500 kN which is 15% higher than the calculated axial load. This indicates that the design for the column is conservative.

CONCLUSIONS

During the design stage of buildings, the column is usually designed to withstand compression loads or combined compression and bending loads for a FRR of 30 to 90 minutes typically, but can be as high as 300 minutes. The longitudinal expansion of the steel tube relative to the concrete will be restrained by the length of column above and below the floor in a continuous column construction system. The proposed design equations presented in this paper have been developed and validated against 238 (121 square, 104 circular and 13 rectangular) Standard Fire test results undertaken worldwide over the past 36 years. The proposed procedure has been shown to be conservative for CFST columns filled with either plain, steel fibre or bar reinforced concrete. Following a sensitivity analysis, a new member section constant "α_b" value has been proposed for columns having different concrete infill type to calculate the buckling member factor of the column in fire. The proposed design procedure has shown to be more accurate than the current AS/NZS 2327 method because, the equations in the standard was developed for circular CFST un-reinforced columns.

ACKNOWLEDGEMENTS

The authors wish to express gratitude to the New Zealand Heavy Engineering Educational & Research Foundation (HEERF) for their scholarship support.

REFERENCES

Albero, V., Espinos, A., Romero, M. L., Hospitaler, A., Bihina, G., & Renaud, C. (2016). Proposal of a new method in EN1994-1-2 for the fire design of concrete-filled steel tubular columns. *Engineering Structures, 128*, 237-255.

AS/NZS 2327. (2017). Composite steel-concrete construction for buildings. *Australian/New Zealand Standards,*

Canadian Standard Association. (2001). Can/csa s16-01. *Limit States Design of Steel Structures,*

CEN. (2005a). EN 1994-1-1, eurocode 4: Design of composite steel and concrete structures– Part 1-1: General rues and rules for buildings. *Comité Européen De Normalisation,*

CEN. (2005b). EN 1994-1-2, Eurocode4: Design of composite steel and concrete structures–Part 1-2: General rules–Structural fire design. *European Standards, London,*

Chabot, M., & Lie, T. T. (1992). Experimental studies on the fire resistance of hollow steel columns filled with bar-reinforced concrete.

Espinos, A., Romero, M. L., Serra, E., & Hospitaler, A. (2015). Circular and square slender concrete-filled tubular columns under large eccentricities and fire. *Journal of Constructional Steel Research, 110*, 90-100.

Espinos, A., Romero, M. L., & Hospitaler, A. (2012). Simple calculation model for evaluating the fire resistance of unreinforced concrete filled tubular columns. *Engineering Structures, 42*, 231-244.

Espinos, A., Romero, M. L., & Hospitaler, A. (2013). Fire design method for bar-reinforced circular and elliptical concrete filled tubular columns. *Engineering Structures, 56*, 384-395.

Grimault, J. (1980). Détermination de la durée au feu des profils creux remplis de béton. *Rapport Final Etablipar Cometube, Commission Des Communautes Europeennes, Recherche Technique Acier, Paris,*

Han, L., Yang, Y., & Xu, L. (2003). An experimental study and calculation on the fire resistance of concrete-filled SHS and RHS columns. *Journal of Constructional Steel Research, 59*(4), 427-452.

Hong, S., & Varma, A. H. (2009). Analytical modeling of the standard fire behavior of loaded CFT columns. *Journal of Constructional Steel Research, 65*(1), 54-69.

Kodur, V. K. R., & Latour, J. (2005). *Experimental studies on the fire resistance of hollow steel columns filled with high-strength concrete.* (Fire Research Program). National Research Council Canada: Institute for Research in Construction. doi:doi.org/10.4224/20378403

Kodur, V. (1998). Performance of high strength concrete-filled steel columns exposed to fire. *Canadian Journal of Civil Engineering, 25*(6), 975-981.

Kodur, V. (1999). Performance-based fire resistance design of concrete-filled steel columns. *Journal of Constructional Steel Research, 51*(1), 21-36.

Kodur, V., & Lie, T. (1996). Fire resistance of circular steel columns filled with fiber-reinforced concrete. *Journal of Structural Engineering, 122*(7), 776-782.

Kodur, V., & Raut, N. (2009). Design equation for predicting fire resistance of reinforced concrete columns. *Structural Concrete, 10*(2), 73-86.

Lie, T. (1994). Fire resistance of circular steel columns filled with bar-reinforced concrete. *Journal of Structural Engineering, 120*(5), 1489-1509.

Lie, T., & Irwin, R. (1995). Fire resistance of rectangular steel columns filled with bar-reinforced concrete. *Journal of Structural Engineering, 121*(5), 797-805.

Moliner, V., Espinos, A., Romero, M. L., & Hospitaler, A. (2013). Fire behavior of eccentrically loaded slender high strength concrete-filled tubular columns. *Journal of Constructional Steel Research, 83*, 137-146.

Myllymäki, J., Lie, T. T., & Chabot, M. (1994). *Fire resistance tests of square hollow steel columns filled with reinforced concrete.* (No. Internal Report 673). Canada: Institute for Research in Construction. doi:http://dx.doi.org/10.4224/20375972

Romero, M. L., Moliner, V., Espinos, A., Ibañez, C., & Hospitaler, A. (2011). Fire behavior of axially loaded slender high strength concrete-filled tubular columns. *Journal of Constructional Steel Research, 67*(12), 1953-1965.

Ukanwa, K., Clifton, G. C., Lim, J. B., Hicks, S. J., & Sharma, U. K. (2017). (2017). *Fire design of continuous concrete filled steel tubular column for a multi-storey building*. Paper presented at the *Applications of Structural Fire Engineering Conference,* Manchester, United Kingdom.

Ukanwa, K. U., Sharma, U. K., Hicks, S. J., Abu, A. K., Lim, J. B. P., & Clifton, G. C. (2017a). Behaviour of continuous concrete filled steel tubular columns loaded concentrically in fire . *Journal of Constructional Steel Research, 136*(C), 101-109.

Ukanwa, K. U., Clifton, G. C., Lim, J. B., Hicks, S. J., Sharma, U., & Abu, A. (2018a). Design of a continuous concrete filled steel tubular column in fire. *Thin-Walled Structures, 131*, 192-204.

Ukanwa, K. U., Clifton, G. C., Lim, J. B., Hicks, S. J., Sharma, U., & Abu, A. (2018b). Simple design procedure for concrete filled steel tubular columns in fire. *Engineering Structures, 155*, 144-156.

Ukanwa, K. U., Lim, J. B., Sharma, U. K., Hicks, S. J., Abu, A., & Clifton, G. C. (2017b). Behaviour of continuous concrete filled steel tubular columns loaded eccentrically in fire. *Journal of Constructional Steel Research, 139*, 280-287.

Wang, K., & Young, B. (2013). Fire resistance of concrete-filled high strength steel tubular columns. *Thin-Walled Structures, 71*, 46-56.

Wang, R., Zhang, C., & Li, G. (2016). Simple approach for performance-based fire safety design of circular CFT columns in large enclosure. *Advanced Steel Construction, 12*(1), 32-43.