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Behavior of Steel Beam to Concrete Filled Tubular Steel Joints after Exposure to Fire

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ABSTRACT:

This paper presents two series of tests on steel beam to concrete filled steel tubular (CFST) column connections and repaired concrete-filled steel tubular connections after exposure to the ISO-834 standard fire. Each test specimen consisted of a CFST column and two steel beam segments connected by external rings to the CFST column on each side to represent an interior joint in a building. The test parameters included the column cross-sectional type, the level of axial load in the column, the beam-column strength ratio and the fire duration time. The applied load in the column was a constant axial load followed by a cyclical lateral load (applied via the connected beams) with increasing amplitude until failure. It was found that the composite joints after exposure to fire exhibited very high levels of energy dissipation and ductility. The tested results were verified by a nonlinear finite element analysis (FEA) model developed by the authors. Comparisons between the test and finite element simulation results show good agreement, which enable the finite element model to be applied in further research studies of the earthquake-resistant behavior of retrofitted fire damaged concrete-filled steel tubular structures.

KEYWORDS: Concrete filled steel tubular (CFST) columns, Joints, Post-fire, Residual strength, Failure mode, Retrofitting

1. INTRODUCTION

Concrete filled steel tubular (CFST) columns have many design and construction merits, which have made this type of construction widely used as columns in the construction of tall buildings in earthquake zones. However, with increased use, the risk of fire damage is increased. Since the economic consequence associated with repairing fire damaged high-rise buildings is high, it is important that the fire damaged building is put back to service with minimum post-fire repair. Against this background, the authors have recently been engaged in research studies to determine the residual strength and seismic behavior of CFST column construction after exposure to fire. Han and Huo (2003), Han et al. (2005) and Han and Lin (2004) have theoretically and experimentally studied the behavior of CFST columns after exposure to the ISO-834 standard fire. Their test results showed that fire exposure did not deteriorate the column ductility, nor the dissipated energy of the CFST columns.

This paper provides experimental studies on the seismic performance of steel beam to CFST column connections after exposure to fire and behavior of repaired CFST joints with the fire damaged steel beams and rings connecting the steel beams and the CFST column being replaced by new ones. The two series of tests on steel beam to CFST column connections after exposure to fire had been described in Huo and Han (2006), Han et al. (2007). The latter series of tests were carried out to simulate a possible repair scheme after exposure to fire. This paper will only present some of the main findings of these tests and the more profound analysis of the test results. The tested results were used to verify a nonlinear finite element analysis (FEA) model developed by the authors. Comparisons between the test and finite element simulation results show good agreement, which enable the finite element model to be applied in further research studies of the earthquake-resistant behavior of retrofitted fire damaged concrete-filled steel tubular structures.

2. EXPERIMENTAL PROGRAM

2.1. Specimen Preparation

Fourteen steel beam to CFST column connection specimens using external ring, including 6 with circular cross-sections and 8 columns with square cross-sections, were tested under a constant axial load and a cyclically increasing flexural load. The fourteen tests were classified into two series of tests as shown in Table 1. The two series of tests were conducted to experimentally study the behavior of steel beam to concrete-filled steel tubular column connections after exposure to fire and the behavior of repaired steel beam to concrete-filled tubular column joints after exposure to fire respectively. Even though the connection specimens were unloaded during fire exposure, some steel beams and external rings were found to have distorted and were replaced with new ones.

Table 1 lists the details of each specimen, where h, b_f, t_w, and t_f are the overall height, overall width, web thickness and flange thickness of the I-beam respectively; D and t_s are the overall dimension and thickness of the steel tube respectively. The column height was 1570 mm. Table 1 also gives the calculated nominal strength $N_u(t)$ for the CFST columns, necessary to determine the test axial load in the columns. This value was calculated by using the mechanics model described in Han and Huo (2003).

Table 1 Summary of Specimen Information

Specimen number	Beam dimension $h \times b_f \times t_w \times t_f$ (mm)	Column dimension $D \times t_s$ (mm)	Ratio of beam to column bending moment capacity k_M	Fire duration time(min.)	$N_u(t)$ (kN)	Load ratio n	P_{ue} (kN)	E (kN·m)	P_{uc} (kN)	P_{uc}/P_{ue}	Test series
C11	140×75×2.9×2.9	133×4.7	0.54	90	455	0.4	15.8	11.1	13.5	0.86	Series I: Behavior of steel beam to CFST column connections after exposure to fire
C12	170×75×2.9×2.9	133×4.7	0.70	90	455	0.4	20.8	15.4	18.2	0.87	
C13	170×75×2.9×2.9	133×4.7	0.70	90	455	0.6	19.3	10.5	15.6	0.81	
S11	140×75×2.9×2.9	120×2.9	0.65	0	873	0.2	25.5	11.8	18.4	0.72	
S12	140×75×2.9×2.9	120×2.9	0.80	90	298	0.2	15.8	19.00	14	0.89	
S13	170×75×2.9×2.9	120×2.9	1.05	90	298	0.2	16.7	27.3	17.6	1.06	
C21	170×75×2.8×2.8	133×4.7	0.96	90	455	0.4	26.3	22.99	23.1	0.88	Series II: Behavior of repaired steel beam to CFST column joints after exposure to fire
C22	150×75×2.8×2.8	133×4.7	0.81	90	455	0.4	24.2	24.35	20.5	0.85	
C23	180×75×2.8×2.8	133×4.7	1.04	90	455	0.4	28.3	28.90	25.0	0.88	
S21	170×75×2.8×2.8	120×2.9	1.44	90	298	0.4	17.7	15.42	15.3	0.86	
S22	170×75×2.8×2.8	120×2.9	0.86	0	873	0.4	28.4	14.51	23.6	0.83	
S31	160×75×2.8×2.8	120×2.9	1.32	90	298	0.2	28.7	-	20.1	0.70	
S32	160×75×2.8×2.8	120×2.9	1.32	90	298	0.4	17.1	44.14	15.1	0.88	
S33	160×75×2.8×2.8	120×2.9	1.32	90	298	0	20.3	19.99	19.9	0.98	

The fourteen tests were designed to investigate the effects of changing the following parameters:

- the column cross-section type (circular or square);
- the standard fire duration time (t =0 or 90 minutes);
- the level of axial load in the column, n (=0.2, 0.4 and 0.6), which is defined as the ratio of the axial load applied in the column, N_o to the axial compressive capacity, $N_u(t)$ of the column at ambient temperature but after exposure to the ISO-834 Standard fire for a fire duration time of t;
- the beam-column strength ratio, k_M (0.54~1.05), which is defined as the ratio of the ultimate bending strength, $M_{ub}(t)$ of the steel beam to the ultimate bending strength, $M_{uc}(t)$ of the CFST cross-section corresponding to fire duration time (t).

The dimensional and constructional details of the test specimen and mechanical properties of steel and concrete were presented in Huo and Han (2006) and Han et al. (2007). All the specimens herein were made according to the same manufacturing process and welding method as described in Han et al. (2007).

2.2. Fire Exposure and Cyclic Loading Apparatus

Prior to structural testing at ambient temperature, twelve of the 14 specimens in Table 1 with a fire exposure duration of 90 minutes were heated simultaneously in a furnace specially built for testing building structures in Tianjin, China. The fire exposure of the tested specimens was described in Han et al. (2007).

After exposure to fire for 90 minutes, the specimens were allowed to cool down in the furnace. Each specimen was then tested under a combined constant axial load and cyclically increasing flexural load. Figure 1 shows the test setup. The lateral loading history was determined based on ATC-24 (1992) guidelines for cyclic testing of structural steel components. All the specimens were loaded to failure under displacement control, as described in Han et al. (2007).

Figure 1 Arrangement of test set-up

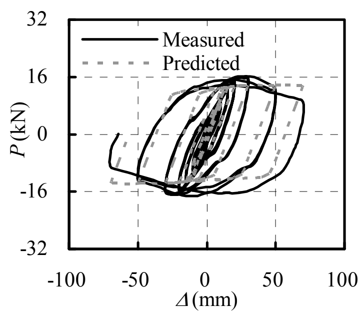
3. TEST RESULTS, ANALYSIS AND DISCUSSIONS

3.1. Experimental Results and Specimen Behavior

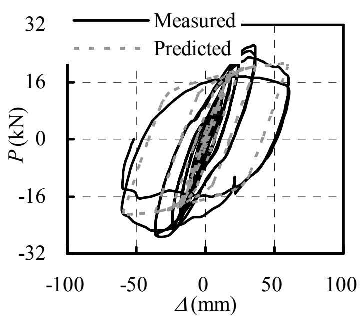
Some typical recorded curves of lateral load (P) versus lateral displacement (Δ) at the column top are shown in Figure 2. Table 1 gives the maximum lateral loads (P_{ue}) obtained by averaging the maximum lateral loads in both directions of load application.

All of the test specimens behaved in a ductile manner and the test proceeded in a smooth and controlled way. All of the Series I test specimens failed due to severe buckling of the steel beams with lateral torsional movements, accompanied by excessive reduction in the lateral load bearing capacity. It can be concluded that the specimens failed in the strong column/weak beam mode after the fire exposure, which is the same as the intended failure mode before fire exposure as indicated in Table 1 by the ratios of the beam to column bending moment capacity being less than 1 generally.

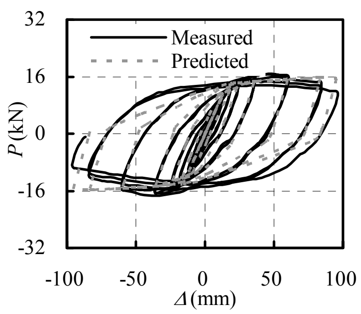
Figure 3 shows all the failed specimens of Series II after the loading tests. It can be seen that except for C21 and C23, all the specimens failed in the intended failure mode as indicated by the beam to column strength ratios shown Table 1. Specimen C23 failed due to formation of plastic hinges in the steel beams, even though it was designed based on the strong beam–weak column concept. Specimen C21 failed due to column yielding, even



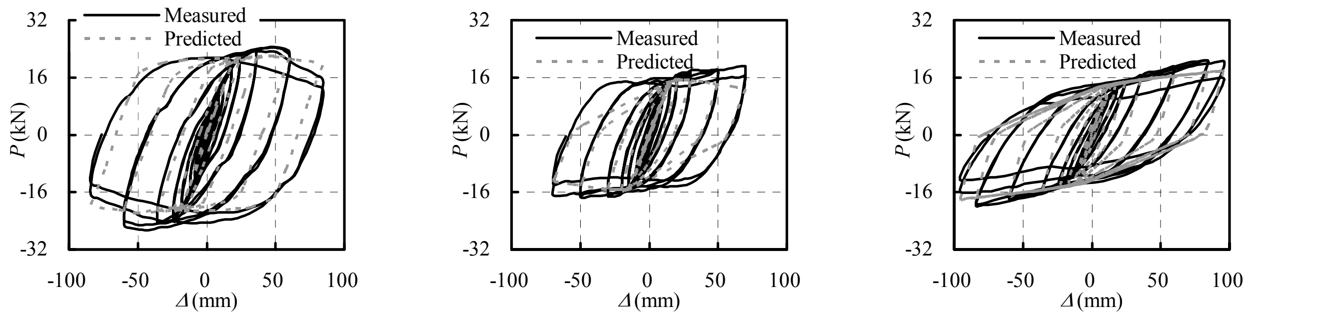
(a) C11



(b) S11



(c) S12



(d) C22-BNC90 (e) S21-BNC90 (f) S33-BNC90

Figure 2 Cyclic load (P) versus lateral deflection (Δ) curves

though it was designed based on the weak beam–strong column concept. The beam-column strength ratios k_M of Specimens C21 and C23 are very close to 1, as shown in Table 1. Therefore, due to inaccuracy in calculations of k_M, it is not surprising that Specimens C21 and C23 failed in modes different from the intended. Specimens S31, S32 and S33 failed due to connection core yielding and local buckling was found to have occurred in the steel tube of the panel zone.

(a) C21 (b) C22 (c) C23 (d) S21

(e) S22 (f) S31 (g) S32 (h) S33

Figure 3 Joint specimen failure mode

3.2. Effects of Different Test Parameters on Column Behavior

3.3.1 Effects of axial load level (n)

Figure 4 shows that the axial load level (n) not only influences the ultimate lateral load (P_{ue}), but also ductility of the specimen. The lateral load resistance increases with increasing the axial load when the axial load is lower than a limit, then decreases with increasing the axial load when the axial load is higher than the limit. This follows from the bending moment – axial load interaction curve of a composite column with low slenderness of 45.3 and 47.2 for CHS and SHS columns respectively. Figure 4 demonstrates that the axial load limit should be between n=0.2 and n=0.4.

Figure 5 shows the effect of axial load level (n) on the ductility coefficient (μ) of the connections with and without repair. The influence of the axial load level on the ductility coefficient μ of the repaired connections is similar to that of fire damaged specimens without repair. It can be shown that the ductility of the retrofitted connections did not deteriorate and was greater than that of the connections without fire exposure. Table 1 shows that the axial load level (n) has a similar effect on the total dissipated energy, E, of the column.

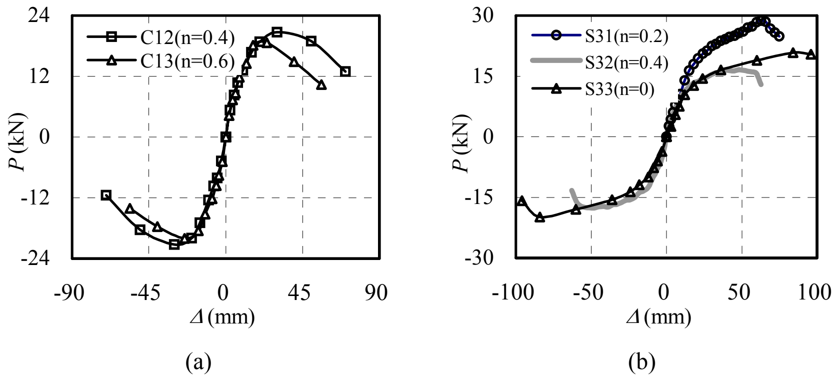


Figure 4 Influence of axial load level on P versus Δ envelope curve

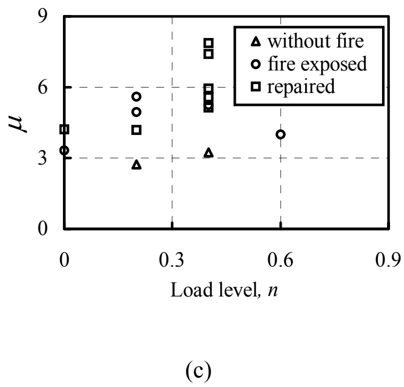


Figure 5 Effect of axial load level on ductility coefficient, μ

3.3.2 Effects of fire duration time (t)

Figure 6 shows the lateral load (P) versus lateral deflection (Δ) envelope curves of specimens S11 (no fire exposure) and S12 (fire exposure 90 minutes). It can be found that for specimen S12, while the stiffness and ultimate lateral load (P_{ue}) are lower, the ultimate lateral deflection corresponding to the ultimate lateral load is larger, indicating improved ductility. This is not surprising because not only the concrete inside the steel tube after fire exposure would have suffered strength and modulus losses, also the concrete would have developed higher strains at the peak stress and its descending stress-strain relation would be less steep than that of the concrete without fire exposure. As a result, the P-Δ curve of the fire exposed specimen tends to be flatter than that of the specimen without fire exposure, thus the dissipated energy of the specimen after fire exposure is greater than that of the specimen without fire exposure. What this shows is that fire exposure would not deteriorate ductility, nor the dissipated energy of steel beam to CFST column connections so that when repairing concrete filled tubular columns after fire, it would be sufficient just to restore the structure's load carrying capacity.

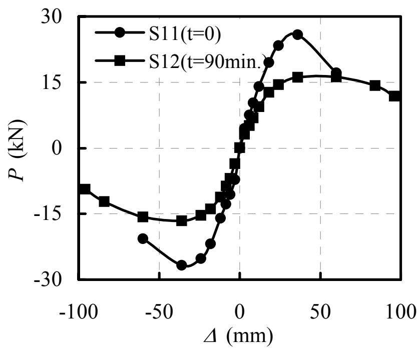


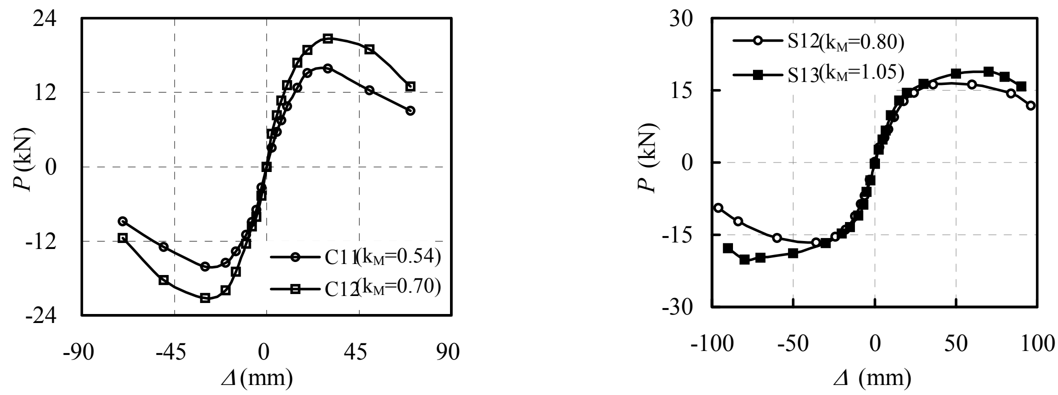
Figure 6 Influence of fire duration time on P-Δ envelope curve

3.3.3 Effects of beam-column strength ratio (k_M)

Figure 7 shows influence of beam-column strength ratio (k_M) on the lateral load (P) versus lateral deflection (Δ) envelope curves. It can be found that, as expected, because beam failure governed the specimen failure mode, the ultimate lateral load (P_{ue}) increases with an increase in beam-column strength ratio (k_M).

The above-mentioned test results show that, it would be sufficient just to restore the structure's load carrying capacity when repairing CFST columns after fire because the ductility and energy dissipated ability of steel beam to CFST column connections after exposure to fire would not deteriorated but improved to a certain degree.

It is well known that framed structures must be designed based on the strong column/weak beam concept. The beam to column strength ratio (k_M) of the connection after exposure to fire would be different from that at room temperature because of different fire-damage losses in the steel beams and the composite columns. Generally, the ratio of k_M after fire would be larger than that at room temperature as shown in Table 1. Although specimen S13 failed as a result of the plastic hinge in the steel beam ends and buckling of the steel tube in the connection zone and finally the welding ruptured, its ductility did not deteriorate.

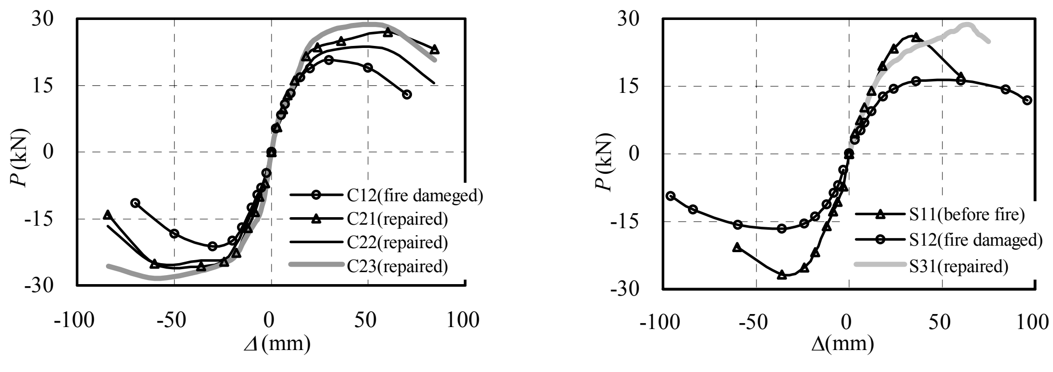


(a) circular columns (b) square columns

Figure 7 Influence of beam-column strength ratio on P - Δ envelope curves

3.3.4 Effects of replacing the fire damaged steel beams and connecting rings

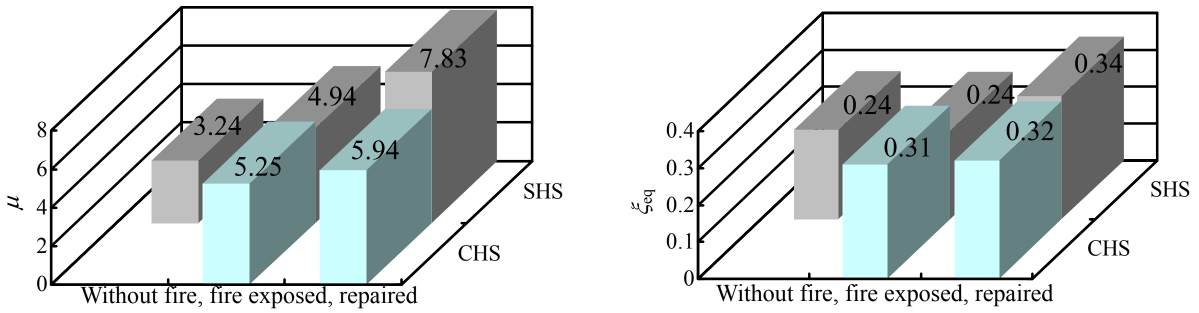
Figure 8 compares the lateral load (P) versus lateral deflection (Δ) envelope curves of the retrofitted connections and those of specimens without fire exposure (S11) and specimens after fire damage but not retrofitted (C12 and S12). It can be found from Figure 8-(a) that for the specimen with fire-damaged steel beams and steel rings replaced by new ones (C21, C22, C23), not only the stiffness and ultimate lateral load (P_{ue}) are higher than those of the specimen with fire damaged steel beam and connection ring (C12), but also the ultimate lateral deflection corresponding to the ultimate lateral load is larger, indicating improved ductility. It can be found from Figure 8-(b) that for the fire-exposed connection (S12), the stiffness and ultimate lateral load (P_{ue}) are lower than that of the specimen without fire exposure (S11); but if the fire damaged steel beams and connecting ring were repaired (S31), the stiffness and ultimate lateral load (P_{ue}) were almost restored to the status without suffering any fire exposure.



(a) Circular CFST column (b) Square CFST column

Figure 8 Influence of fire exposure and repair scheme on P versus Δ curves

Figure 9 shows the effects of fire exposure and retrofitting on ductility coefficient (μ) and accumulative equivalent damping coefficient (ξ_{eq}). Here the ductility coefficient is defined as $\mu = \Delta_u / \Delta_y$, where Δ_y is the lateral displacement at material yield and Δ_u the lateral displacement when the lateral load falls to 85% of the maximum lateral strength (P_{ue}). The equivalent damping coefficient (ξ_{eq}) was determined by the calculation method described in Han et al. (2007). The test results indicate that ductility and the dissipated energy of steel beam to CFST column connections did not suffer any deterioration regardless whether or not the fire-damaged steel beams and connecting rings were replaced.



(a) Ductility coefficient (μ) (b) Accumulative **e**quivalent damping coefficient (ξ_{eq})

Figure 9 Effect of fire exposure and retrofitting on μ and ξ_{eq}

Specimens C21, C22 and C23 were designed based on the weak beam–strong column concept, while specimens S31, S32 and S33 were designed based on the strong beam–weak column concept. The test results show that the strong beam–weak column type connection specimens failed owing to the failure of CFST columns or the rupture of steel tube in the connection zone as shown in Figure 3. Generally, framed structures in seismic zones are designed based on the strong column–weak beam concept. Therefore, the fire-damaged steel beam to CFST column connections should be retrofitted back to the strong column–weak beam type connections. However, in this paper, the fire-damaged connections were also retrofitted into strong beam–weak column type connections to investigate the behavior of retrofitted strong beam–weak column type connections.

The retrofitting scheme used in this paper is easy and safe to carry out and the retrofitted fire-damaged CFST connections did not suffer any deterioration in their ductility and energy dissipation ability. Therefore, when using the retrofitting scheme of replacing the fire-damaged steel beams and connecting rings by new ones, the retrofitting design decision would now simply become a matter of restoring the steel beam and the CFST column's load bearing capacity and stiffness to their original values. Ding and Wang (2007) carried out experimental study on structural fire behavior of steel beam to CFST column assemblies using different types of joints. The test results show that the composite assemblies failed mainly due to structural failure in the joints under tension when the beam was clearly in substantial catenary action, while the CFST columns remained in good condition. Therefore, the retrofitting scheme presented in this paper can be used to repair the fire-damaged composite assemblies.

3.3.5 Verification of a nonlinear finite element model

A nonlinear finite element analysis (FEA) model was developed to predict the hysteretic load-deflection responses of CFST connections after exposure to fire. The FEA model was described in details in Han et al. (2007). Figure 2 compares the load versus deformation hysteretic curves between the FE predictions and the test results. Tables 1 compares the predicted ultimate load bearing capacities, P_u of the connection specimens with the test results.

Figure 2 and Table 1 indicate that the FEA model can predict the P-Δ relations of steel beam to CFST column connections and the column lateral load resistance with reasonable accuracy. It can be found that, in general, the predicted lateral load resistance tends to be lower than the measured values, and therefore the predicted values are on the safe side. The FEA model accurately predicts the connection stiffness during the initial stage of loading. The differences between the calculated and experimental curves become significant during the later stages of loading, especially the descending stages, with the FEA predictions giving higher results. This may be attributed to the higher beam moment versus curvature model adopted in this paper which neglected the repeated damage of materials for the steel beams and CFST columns.

5. CONCLUSIONS

This paper introduced two series of new tests on steel beam to CFST column connections using external ring after exposure to the ISO-834 standard fire to assess the effects of different test parameters on the performance of the composite connections.

After fire exposure, the column lateral load carrying capacity and stiffness of the fire-damaged connections were reduced, but the connection ductility and energy dissipation were increased. The damping coefficient was almost unchanged. It can be concluded from the experimental results of Series II repaired CFST connections after exposure to fire that the retrofitting scheme used in this paper is easy and safe to carry out and the retrofitted fire-damaged CFST connections did not suffer any deterioration in their ductility and energy dissipation ability. The tested results were also verified by a nonlinear finite element analysis (FEA) model developed by the authors. Comparisons between the test and finite element simulation results show good agreement, which enable the finite element model to be applied in further research studies of the earthquake-resistant behavior of retrofitted fire damaged concrete-filled steel tubular structures.

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