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Experimental Investigation on Post-Fire Performances of Fly Ash Concrete Filled Hollow Steel Column

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Abstract

In structural engineering practice, understanding the performance of composite columns under extreme loading conditions such as high-rise bulding, long span and heavy loads is essential to accuratly predicting of material responses under severe loads such as fires or earthquakes. Hitherto, the combined effect of partial axial loads and subsequent elevated temperatures on the performance of hollow steel column filled fly ash concrete have not been widely investigated. Comprehensive test was carried out to investigate the effect of elevated temperatures on partial axially loaded square hollow steel column filled fly ash concrete as reported in this paper. Four batches of hollow steel column filled fly ash concrete (30 percent replacement of fly ash), (HySC) and normal concrete (CFHS) were subjected to four different load levels, n_f of 20%, 30%, 40% and 50% based on ultimate column strength. Subsequently, all batches of the partially damage composite columns were exposed to transient elevated temperature up to 250°C, 450°C and 650°C for one hour. The overall stress – strain relationship for both types of composited columns with different concrete fillers were presented for each different partial load levels and elevated temperature exposure. Results show that CFHS column has better performance than HySC at ambient temperature with 1.03 relative difference. However, the residual ultimate compressive strength of HySC subjected to partial axial load and elevated temperature exposure present an improvement compared to CFHS column with percentage difference in range 1.9% to 18.3%. Most of HySC and CFHS column specimens failed due to local buckling at the top and middle section of the column caused by concrete crushing. The columns failed due to global buckling after prolong compression load. After the compression load was lengthened, the columns were found to fail due to global buckling except for HySC02.

Keywords: Composite columns, fly ash concrete, elevated temperature, load levels, axial capacity

1. Introduction

In recent years, steel-concrete composite columns have gained widespread usage as load bearing constituents in the construction industry for bridges, high-rise buildings and other engineering structured around the world due to the benefits offered in terms of mechanical behavior and construction properties over plain concrete or steel hollow sections (Lie and Chabot, 1992; Han *et al.*, 2014; Du *et al.*, 2017; Shekastehband *et al.*, 2017; Tao *et al.*, 2017; Wan *et al.*, 2017). In addition , the utilization of the new concrete mix designs as a filling material for hollow steel columns (HSC) has gained much attention. Hitherto, limited research has been done on the performance of hollow steel columns filledwith fly ash concrete (HySC) when subjected to partial axial loads and subsequent elevated temperatures. Hence, this paper discusses the performance of partial axially loaded HSC that used pozzolanic material (fly ash) as a cement replacement in the concrete mix design as filling material exposed to elevated temperatures.

Stated that the concrete filled hollow steel short column critical due to outward local buckling caused by concrete crushing (Sangeetha *et al.*, 2018). Meanwhile, slander column which critical due to global buckling due to deflection at mid-span of unbraced column (Yang *et al.*, 2015). It is shown that the failure criteria of the concrete filled hollow steel short column depends on the concrete. Therefore, this research aims to study the effect of concrete consisting 30% fly ash as cement replacement (30% fly ash concrete, 30FC) as concrete filler in hollow steel column, the behaviours of short column were taken into account.

Fly ash generated from the combustion of coal for energy production, is recognized as an environmental pollutant. Fly ash is also known as fuel ash and since the 1920s millions tons of ash and related by-products have been generated (Ahmaruzzaman, 2010). The current annual

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production of coal ash worldwide is estimated to be around 600 million tons, with fly ash constituting about 75-80% of total ash production. Based on its chemical composition, two general classes of fly ash can be defined: low-calcium fly ash (Class F) produced by burning anthracite or bituminous coal and high-calcium fly ash (Class C) produced by burning lignite or sub-bituminous coal. Class F fly ash class F is categorized as a normal pozzolan containing silicate glass, modified aluminium and iron (Papadakis, 1999) which formed cement in the presence of water (pozzolanic reaction). Due to the pozzolanic reaction, the strength increment for pozzolanic concrete continues for a longer period of time than conventional concrete (Saha, 2018).

The utilization of coal combustion by-products, namely fly ash, can be an alternative to industrial resources (Ahmaruzzaman, 2010). In Malaysia, Class F fly ash has been widely used as addtion or replacement to cement and concrete products, structural fills, cover materials, road and pavement. Disposal processes in landfills require proper handling as fly ash particles are considered highly containimating due to the enrichment in toxic trace elements that significant impact pozzolanic reactivity (Earle and Scheetz, 1998). Figure 1 shows the particle size distribution of fly ash (FA), rice husk ash (RHA) and Ordinary Portland cement (OPC) (Chindaprasirt and Rukzon, 2008). Shows that, fly ash is the finest, followed by RHA and OPC.

Previous studies have considered the utilization of fly ash as cement replacement in geopolymer concrete (Hardjito *et al.*, 2004; Ahmaruzzaman, 2010; Ibrahim *et al.*, 2012; Abdulkareem *et al.*, 2014; Sarker *et al.*, 2014; Sarker and McBeath, 2015; Mehta and Siddique, 2017; Fan *et al.*, 2018). Jiang found 30% fly ash replacement of cement in normal strength concrete as optimum for the concrete's porosity (Jiang *et al.*, 2017). Fly ash used as cement replacement occupies a high volume in concrete and significantly influences its behaviour at elevated

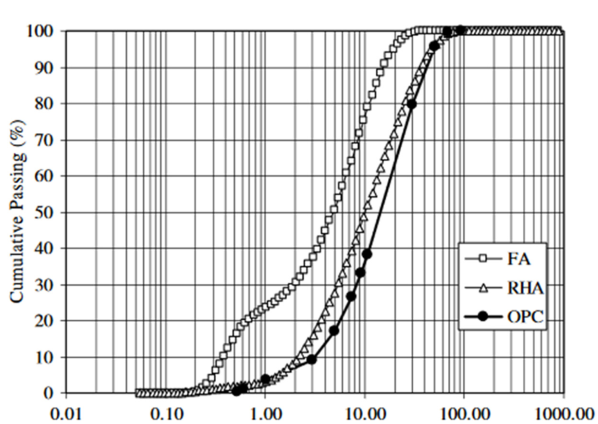


Figure 1. Particle size distribution of fly ash (FA), rice husk ash (RHA) and Ordinary Portland cement (OPC) (Chindaprasirt and Rukzon, 2008).

temperature. Ashish studied the effect of Class F fly ash on the durability of concrete at ambient temperatures to find that, the compressive strength of concrete cylinder at 28 days curing sharply decreased (Saha, 2018). However, for 30% and 40% fly ash, compressive strength of the concrete increased gradually up to 180 days and at 360 days curing the results showed a constant compressive strength. Kondraivendhan studied the flow behaviour and strength of fly ash blended cement paste and mortar, (Kondraivendhan and Bhattacharjee, 2015) with four water to binder ratios of 0.25, 0.35, 0.45, and 0.55; three curing ages, and five fly ash replacement levels (f/c ratio) of 0, 0.1, 0.2, 0.3 and 0.4 at ambient temperature. Found that, as the water binder ratio decreased, the paste mixes tend to become stiff even with the additional of super-plasticizers as indicated by increased flow time.

Wang studied the thermal conductivity of 30% fly ash concrete to elevated temperatures and an effect of micro-environment relative humidity of 20%, 45%, 75%, and 100%. For the micro-environment relative humidity of 45%, the thermal conductivity of ordinary concrete was constantly greater than pozzolanic concrete, however, at 100% humidity , the thermal conductivity of pozzolanic concrete (1.603 W/m.K) was higher than normal concrete (1.599 W/m.K) (Wang *et al.*, 2017). Rahel studied the fire resistance of high-volume fly ash mortars with nanosilica additions and found that high strength mortars had an equivalent residual strength before and after exposure to elevated temperatures as high-volume fly ash and colloidal nanosilica replacement concretes (Ibrahim *et al.*, 2012). This was proven by Field Emisson Scanning Electron Microscope (FESEM) analysis, with magnifications up to 20 KX. After exposure to 400^{o}C, an increase in calcium silicate hydrate was noticed. Thus, from all recent studies, fly ash is expected to offer better performance than Ordinary Portland cement (OPC) at elevated temperatures.

2. Specimens

Concrete consisting of 30% fly ash substitute for Ordinary Portland Cement (OPC) with a design strength of 30 MPa was used as a filler in square hollow steel

Figure 2. Sample of fly ash.

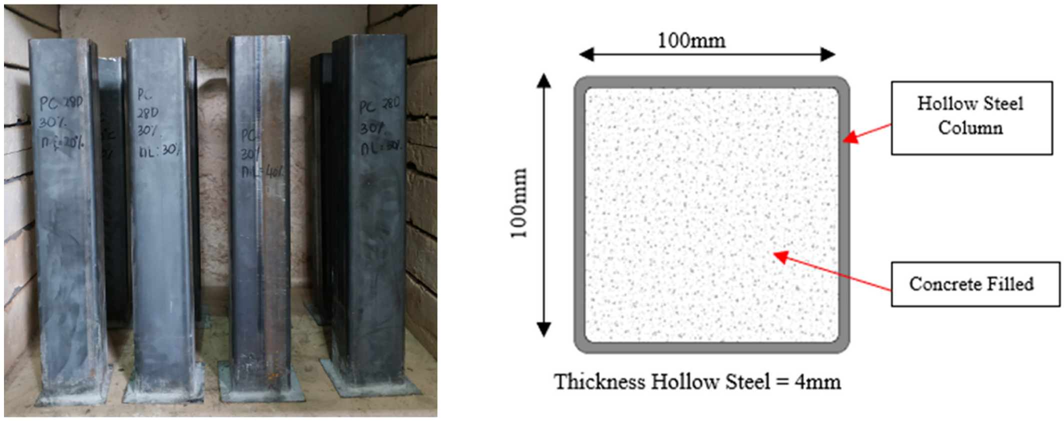


Figure 3. The column specimens and cross section of square column section.

columns with an ultimate yeild strength of 250 MPa. Figure 2 shows the sample of fly ash used for cement replacement. A total of 24 specimens (12 specimens for CFHS and 12 specimens for HySC) were tested. Figure 3 shows a cross section of the column specimens. The column specimens had a width of 100 mm × 100 mm, a height of 600 mm and a thickness of 4 mm. The ratio length to cross-section diameter was 6 for specimens to ensure short column behaviour.

3. Testing Set-up

There are two types of columns green concrete filled hollow steel column (HySC) and normal concrete filled hollow steel column (CFHS) were casted. Total 24 composite columns (HySC and CFHS), were subjected to the load level, n_f = 20%, 30%, 40% and 50% partial axial loads . Partial axial load level, n_f based on the ultimate axial strength of the column tested at ambient temperature shown in Figure 5. Followed by exposure to elevated temperatures of 250ºC, 450ºC and 650ºC for one hour after it reached its targeted temperature in order to make sure every depth of column specimen experience constant temperature. Due to consideration on slow heating rate effect and availability, the electric furnace with heating rate of 2.78ºC/min which considered as slow heating rate, as shown in Figure 4 was used.

Figure 4. Electric furnace 0.8 × 0.8 × 0.8.

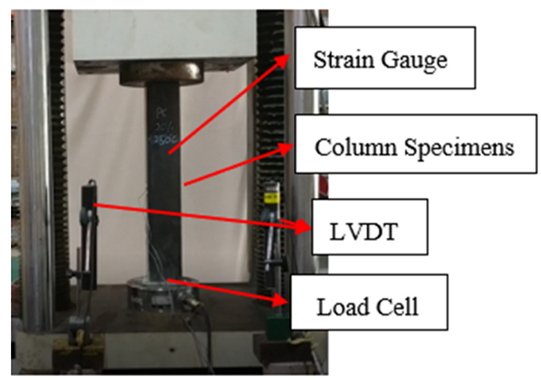


Figure 5. Experimental setup for compression column testing.

The HySC and CFHS columns were tested in a 1500 kN capacity universal testing mechine as shown in Figure 3. Each column was tested to failure by applying concentric axial load gradually by using SOP Tinius Olsen type universal testing machines at a displacement-control load method. A small loading rate of 0.06 mm/sec was used in order to capture accurate failure mode. The load were measured by using load cell located at the bottom of column. Two Linear Variable Differential Transformer (LVDT)s to determine vertical displacement. Meanwhiles, strain gauges were used to determine the longitudinal and circumferential strains at the of the samples as shown in Figure 2. The residual performance of the HySC and CFHS were investigated experimentally. The axial stress – vertical strain curves for all HySC and CFHS columns were developed.

4. Residual Compressive Strength

Figure 6 shows the axial stress-vertical strain for HySC

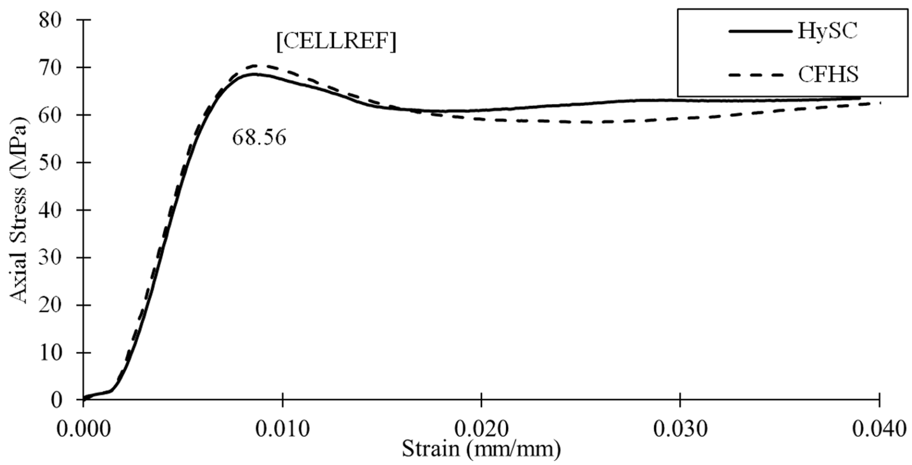


Figure 6. Axial strength versus strain for composite columns at ambient temperature.

and CFHS at ambient temperature. The results from this test were used to determine the load level, n_f. At ambient temperature, the capacity of CFHS and HySC specimens were recorded to be 70.31 MPa and 68.11 MPa, respectively with 1.03 relative difference. It can be observed that the compressive strength strain rate for HySC was higher compared to CFHS column specimen. This is due to the lower strength provided from fly ash concrete at ambient temperature caused reduction in confinement effect of concrete filled hollow steel column. Similar trend were recorded where concrete filled hollow steel (CFHS) column containing fly ash concrete presents lower compressive strength than CFHS column with conventional concrete (Ordinary Portland cement, OPC). As reported, the increase in ultimate capacity of the columns mainly depends on strength of the filler materials (Arivalagan and Kandasamy, 2010; Tao *et al.*, 2018).

Table 1 shows the axial capacity of columns specimens after subjected to different load level and elevated tempertaure. It can be observed that the performance of normal concrete rapidly decreased when exposed to elevated temperatures while for fly ash concrete high temperature can activate a second hydration process through pozzolanic reactions. As been reported, partial loading do not show significant reduction of the residual compressive strength. Because, the maximum range reduction of residual ultimate compressive strength when partial axial load applied 20% to 50% are in range 0.74% to 2.74%. HySC present the higher residual compressive strength compared to CFHS with percentage differences in range 1.9% to 18.3%.

Figure 7 shows the tested curves of residual compressive strength vs vertical strain of HySC and CFHS column specimens subjected to 250°C targeted elevated temperature. The results show that the strength of HySC 20 (20%

Table 1. Comparison of compressive strength for HySC and CFHS column specimens

Features	Temperature Exposure (°C)	Partial Loading, n_f (%)	Compressive Strength		Percentage Difference (%)
			HySC	CFHS	
-	27°C	-	68.11	70.31	2.6
01	250°C	20	73.61	68.51	7.4
02		30	73.06	68.49	6.7
03		40	72.37	68.46	5.7
04		50	71.89	60.76	18.3
05	450°C	20	71.49	66.68	7.2
06		30	70.35	65.69	7.1
07		40	69.36	65.30	6.2
08		50	69.14	60.24	14.8
09	650°C	20	64.61	63.31	2.1
10		30	63.81	62.63	1.9
11		40	63.40	59.14	7.2
12		50	61.66	58.28	5.8

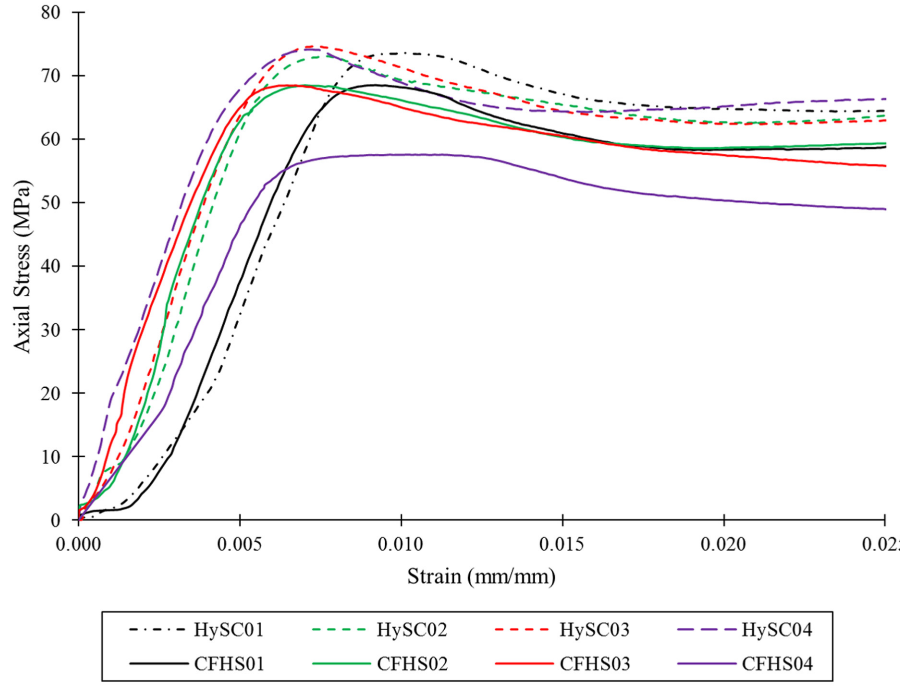


Figure 7. Residual compressive strength *vs* strain for partially loaded HySC and CFHS column after exposed to 250ºC.

partial axially load level) reached the highest with 73.61 MPa. Meanwhile, CFHS columns with 50% partial axially load level reported the lowest value of 60.76 MPa. Based on the observation, it can be concluded that HySC can offer a better residual performances than CFHS column specimens. This can be proved when the HySC columns were subjected to partial axial load level (20%, 30%, 40% and 50%), the residual compressive strength were recorded to be 73.61 MPa, 73.06 MPa, 72.37 MPa and 71.89 MPa, respectively. Meanwhile, 68.51 MPa, 68.49 MPa, 68.46 MPa and 60.76 MPa for CFHS columns, which means that even when HySC columns were subjected to 50% partial axially load level, its residual performance can be better than CFHS columns after exposure to high temperature. In fact, a large drop in residual performance of CFHS columns at 50% load level

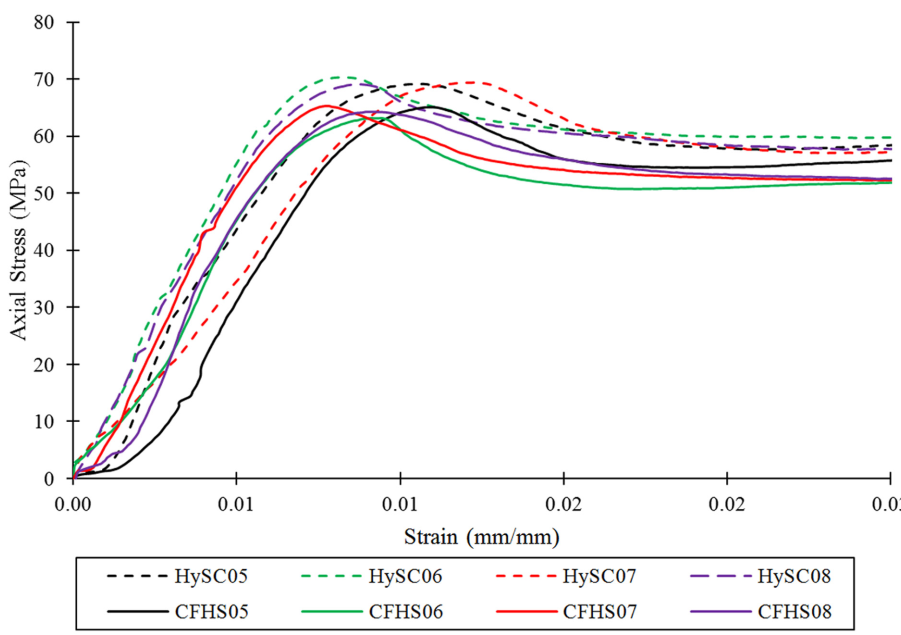


Figure 8. Residual compressive strength vs strain for partially loaded HySC and CFHS column after exposed to 450ºC

can be observed.

Figure 8 shows residual axial stress-vertical strain curves of HySC and CFHS after exposure to 450°C at different partial load levels (20%, 30%, 40% and 50%). Upon further heating to 450°C, the residual compressive strength of HySC 20, HySC 30, HySC 40 and HySC 50 gives small reduction of about 2.9, 3.7, 4.2 and 3.8%, respectively. Meanwhile, the residual compressive strength for CFHS columns reduced in range 0.8 to 4.5%. It can be observed that even after exposure to higher temperature, the graph gives similar trend. In fact, it can be concluded that increase in temperature up to 450°C does not give high impact to concrete filled hollow steel columns. At 650°C, the residual compressive strength of HySC columns were decreased in range 12.2 to 14.2%, and 4.1 to 13.6% for CFHS columns as shown in Figure 9. This is due to the dehydration of water from calcium silicate hydrate (CSH) cause reduction in compressive strength of concrete. Slow heating rate can affect the high rate of dehydration process in the concrete that can cause drop of mechanical properties (Le, 2016). It also caused steam pressure from water released during heating combined with compressive thermal and static stresses at the fire exposed surface (Hertz, 2005). This is especially true for fly ash, which is critical to spalling effect. In fact, at higher temperature above 500°C, the important element in cement calcium hydroxide ($Ca(OH)_2$) dissociates resulting in shrinkage in concrete element. As a result, a significant reduction in residual compressive strength of the column with 650°C targeted temperature.

5. Stiffness

The stiffness of the columns was measured and presented in Table 2 based on Figures 7, 8 and 9. Stiffness of the column was measured based Equation 1, where, S for stiffness, F for load and δ for deformation. Based on the results, the stiffness of HySC specimens at ambient temperature was about 136.77 kN/mm compared to CFHS column with 130.20 kN/mm. However, stiffness of the columns degrade with elevated temperature exposure. At 250°C elevated temperature exposure, the stiffness of HySC was in range 106.99 kN/mm to 124.84 kN/mm, compared to CFHS columns in range 67.99 kN/mm to 114.58 kN/mm, thereby stiffness enhances load carrying capacity of the columns. It is proved, the residual compressive strength of the HySCs were higher compared to CFHS columns with maximum percentage of 18.3% at 250°C. Further heating up to 450°C, the stiffness of HySC was recorded to be maximum of 101.60 kN/mm compared to CFHS columns with 95.74 kN/mm. At 650°C elevated temperature exposure, the maximum stiffness was recorded to be 69.65 kN/mm and 63.30 kN/mm for HySC and CFHS column specimens, respectively. The stiffness of the columns presents massive degradation at 650°C elevated temperature exposure. Consequently, the load shares of HSC and concrete core are gradually redistributed to the concrete core before failure, resulting losses of composite action. It can be summarized that HySC behave in good performances as compared to CFHS column as control specimens.

$$S = F/\delta \tag{1}$$

6. Failure Mode

Table 2 shows the failure mode for HySC and CFHS after subjected to 30% partial axial load and elevated temperature exposure at 250°C, 450°C and 650°C. Failure modes were divided into two phases, ultimate axial capacity

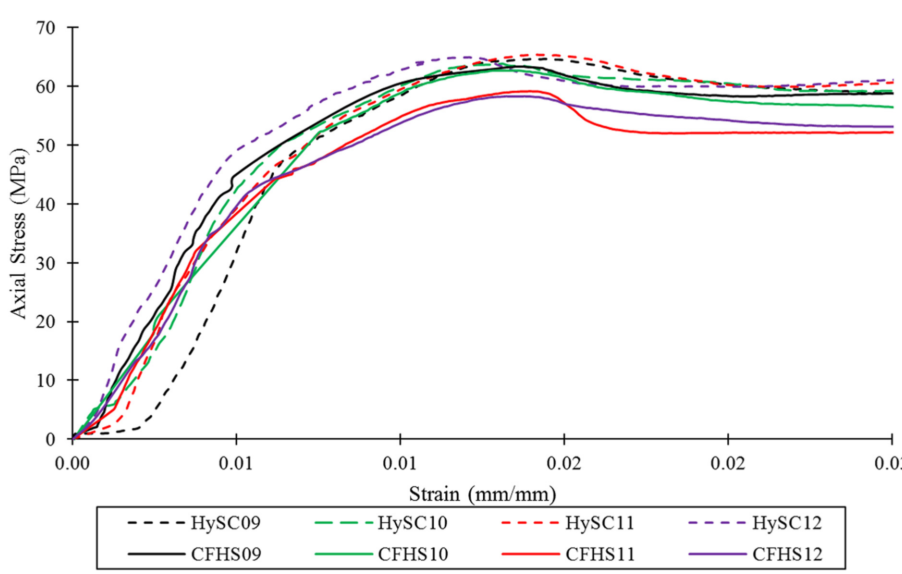


Figure 9. Residual compressive strength *vs* strain for partially loaded HySC and CFHS column after exposed to 650°C.

Table 2. Stiffness of the column specimens

Temperature Exposure (°C)	Column	Stiffness (kN/mm)	Column	Stiffness (kN/mm)
27	HySC	136.77	CFHS	130.20
250	HySC01	106.99	CFHS01	112.49
	HySC02	122.69	CFHS02	108.26
	HySC03	124.84	CFHS03	114.58
	HySC04	114.83	CFHS04	67.99
450	HySC05	100.52	CFHS05	81.82
	HySC06	101.60	CFHS06	80.47
	HySC07	77.12	CFHS07	95.74
	HySC08	91.31	CFHS08	79.09
650	HySC09	69.65	CFHS09	63.30
	HySC10	61.91	CFHS10	58.89
	HySC11	56.31	CFHS11	58.21
	HySC12	63.92	CFHS12	58.11

and plastic behaviour. The deformation of local buckling for all HySC and CFHS column specimens were observed once the column reach its ultimate axial strength. While, the plasticity behaviour was observed by lengthened the axial load for about 15 minutes once the column reach its ultimate axial strength.

As for first phase, it can be observed that HySC presented a local buckling at the one sixth of top column section. In contrast with CFHS column which presented local buckling at the middle section of the column at 250°C. At 450°C, it can be noticed that HySC shows uniform budge compared to CFHS column at 450°C. The formation of

Table 3. Mode failure of column specimens after being subjected to 30% partial axial loading and temperatures up to 250°C, 450°C and 650°C

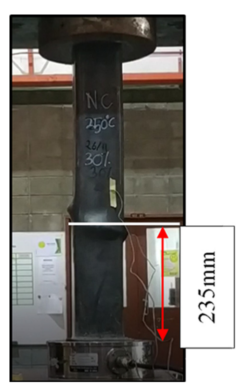
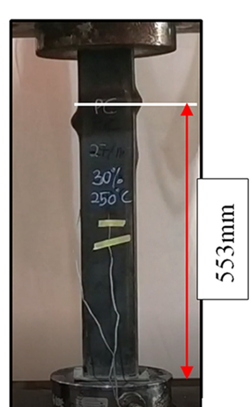
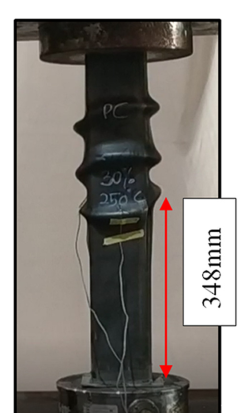
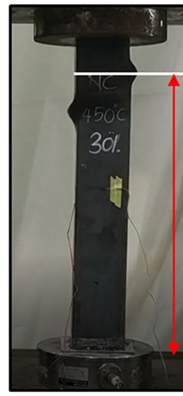
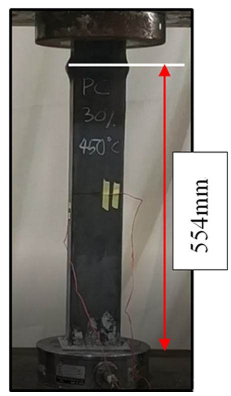
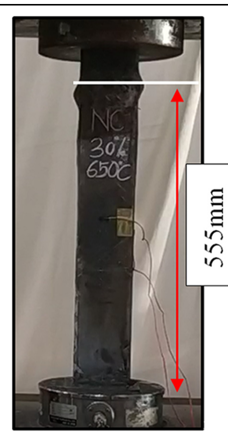
Specimens	Temperature Exposure (°C)	Phase 1	Phase 2
CFHS	250°C		
HySC			

Table 3. Continued.

Specimens	Temperature Exposure (ºC)	Phase 1	Phase 2
CFHS	450ºC		
HySC			
CFHS	650ºC		
HySC			

several bulges were noticed at the top of HySC and CFHS column specimens at 650°C targeted elevated temperature as both HySC and CFHS column specimens experienced severe corrosion of coating layer. There is no obvious change of failure phenomenon with different of concrete type, partial axial load and targeted elevated temperature. Base on the previous study, it is stated that the typical local buckling of concrete filled hollow steel short column occurred at top or bottom column section (Tao *et al.*, 2018). However, the tendency of local buckling to occur in the middle of column height should not be neglect (Han *et al.*, 2017). The effect of concrete type, partial axial load and elevated temperature on the mechanical behaviour of HySC after elevated temperature should be made futher research.

As for plastic behaviour, clearly shows that CFHS column experienced global buckling in second phase of axial loading, in contrast with HySC which remaind straight at 250°C targeted temperature. Hence, HySC resisted loading better than CFHS, as bending can induce eccentricity not equal to zero for the columns. At 450°C targeted temperature, CFHS column presented a severe global buckling due to uneven local buckling in the first phase compared to HySC which experienced slight global buckling. At 650°C a grey colour were observed for both HySC and CFHS columns with corrosion of coating layer. In addition, CFHS column specimens experienced severe failure at the middle height of the column in second phase. Proved that, a significant reduction in residual compressive strength were recorded by CFHS column after 650°C elevated temperature exposure compared to HySC. It can be summarized that all HySC and CFHS column specimens experienced global buckling at mid-height of the column.

7. Conclusions

This paper presents the residuals performance of partially loaded hybrid columns (fly ash concrete filled hollow steel columns) subjected elevated to temperatures of 250°C, 450°C and 650°C. Based on the experimental results, the following conclusion can be drawn:

1. The enchanced structural behavior of the columns can be explained in terms of "Composite Action" between the HSC and concrete core.
2. CFHS colum recorded to have higher ultimate axial strength than HySC at ambient temperature with 1.03 ratio of difference.
3. It can be observed that HySC can perform better than CFHS columns even after subjected to partial axial load and elevated temperature with percentage differences in range 1.9% to 18.3%.
4. The loss of the strength of column HySC and CFHS afer exposed to elevated temperature up to 650°C proved that fire exposure influence the residual compressive strength of concrete filled hollow steel column (composite column).
5. Most of HySC and CFHS column specimens failure due to l°Cal buckling., there is no significant change induced by type of concrete, partial axial load and elevated temperature exposure on failure mode.
6. Thus, HySC showed better residual ultimate axial strength compared to CFHS subjected to partial axial loads and elevated temperatures.

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