



FIRE RESISTANCE OF NORMAL AND HIGH-STRENGTH CONCRETE WITH CONTAINS OF STEEL FIBRE

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ABSTRACT

This paper presents the behavior of steel fibre concrete post material burned under compression. Experimental program is carried out by making a concrete cylinder specimen in which the parameters being reviewed are the concrete compressive strength and the temperatures. The results of such experiments is that the degradation of steel fibre concrete compressive strength of the specimen average quality control on all concrete is about 10% to 20% when the specimen is burned at a temperature of 300°C, where the degradation increases with the increasing compressive strength of concrete. The degradation of concrete compressive strength on the control specimens is significantly (50%-60%) will occur when the specimen is burned at a temperature of 600°C. When the firing temperature is increased to 900°C the degradation of compressive strength will fall, and the maximum compressive strength loss occurs on high strength concrete that is approximately 75%. The stress-strain models of steel fibre concrete at various temperature levels are developed, and the results of validation show the behavior before and after the peak which are relatively close to the experimental results.

Keywords: Concrete; steel fibre; compressive strength; temperature; stress-strain.

1. INTRODUCTION

1.1 Background

In recent years the civic buildings often have fires for various reasons, due to short circuit, blast stove/gas cylinders, bombs, lightning strikes, or because of the unrest deliberately burned the innocent buildings. When the fire happens long enough it is possible to reach temperatures of 900°C or even more, so it should be investigated if the steel fibre concrete is at a very high temperature.

It has been generally known that one of the advantages of concrete fibre to normal concrete (without fibre) is more ductile so that their use in energy dissipation properties of

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the structure that is better against earthquake loads. The types of fibre as one of the concrete mix is also growing [1,2], and one of the fibres that is easily available in the market is the steel fibre. In general, the addition of steel fibre in concrete can increase the compressive strength and peak strain, and also more sloping post- peak curve in the stress-strain behavior. Another advantage by using this type of materials in the structure is higher cracking capacity and the increased resistance to fatigue compared with the normal concrete material [3-5]. Workability problems on steel fibre concrete can also be resolved by adding additional ingredients such as superplasticizer or viscocrete with certain doses. During its development, high strength steel fibre concrete has also been able to be produced [6, 7], so that the material has excellent prospects in use for earthquake-resistant structures.

1.2 Research Significance

Steel fibre concrete is very sensitive to temperature, especially at high temperatures, compared with normal concrete (without fibre). The content of steel fibre in concrete is a major factor affecting the mechanical behavior (such as compressive strength) of fibre concrete. Until the last decade, equation fibre concrete compressive strength degradation, especially steel fibres, at various levels of relative temperature is not widely produced. The constitutive models are generally of steel fibre developed still limited to the normal concrete [8-14]. Therefore, the constitutive equations fibre concrete, especially steel fibre, the many-level temperature is required. Fibre concrete modeling at various levels of high temperatures is very important because it will determine the feasibility of the use of post-fire structure.

1.3 Objective

This paper describes experimental results of steel fibre concrete under load until collapse, both before and after being burned at various temperature levels, in order to know more deeply the behavior of steel fibre concrete at a certain temperature. The discussion will revolve around the strength of concrete at various temperatures, including normal to high strength concrete and stress-strain behavior. Experimental data are then used to develop a model of the stress-strain steel fibre concrete at various temperature levels.

2. COMPRESSIVE STRENGTH OF CONCRETE AT ELEVATED TEMPERATURES

The compressive strength of concrete is the main mechanical scale and a reference to determine the quality of the concrete, so that the compressive strength degradation becomes the most scrutiny when the concrete is at the higher temperature. High temperature due to fires affects the strength and stiffness of the various elements of reinforced concrete structures, such as columns, beams and floor slabs. The addition of steel fibres and polypropylene in concrete mixtures is known to maintain stability or degradation of compressive strength or compressive strength is only about 25% of the control if the compressive strength of concrete is up to temperature of 500°C, but the compressive strength of fibre concrete will immediately drop and collapse when the temperature is at 500°C [15].

The constitutive equations of normal concrete (without fibre) compressive strength at

various temperatures that have been developed in the last ten years are shown in Table 1.

Table 1: Model of concrete compressive strength at various temperatures

Authors	Compressive strength of concrete at elevated temperatures
British Standard, BS EN 1992 [2004]	$f'_{cT} = f'_c \quad ; \quad T \leq 100^\circ C$ $f'_{cT} = f'_c (1.067 - 0.00067T) \quad ; \quad 100^\circ \leq T \leq 400^\circ C$ $f'_{cT} = f'_c (1.44 - 0.0016T) \quad ; \quad T \geq 400^\circ C$
Li & Purkiss [2005]	$f'_{cT} = f'_c \left[0.00165 \left(\frac{T}{100} \right)^3 - 0.03 \left(\frac{T}{100} \right)^2 + 0.025 \left(\frac{T}{100} \right) + 1.002 \right]$
Hertz [2005]	$f'_{cT} = f'_c \left[\frac{1}{1 + \frac{T}{T_1} + \left(\frac{T}{T_2} \right)^2 + \left(\frac{T}{T_8} \right)^8 + \left(\frac{T}{T_{64}} \right)^{64}} \right]$ <p>For :</p> <p>Siliceous aggregate: T1=15000, T2=800, T8=570, T64=100000</p> <p>Lightweight aggregate: T1=100000, T2=1100, T8=800, T64=940</p> <p>Other aggregates: T1=100000, T2=1080, T8=690, T64=1000</p>
Kodur et al. [2008]	$f'_{cT} = \begin{cases} f'_c [1.0 - 0.003125(T - 20)]; & T < 100^\circ C \\ 0.75 f'_c & ; 100^\circ C \leq T \leq 400^\circ C \\ f'_c [1.33 - 0.00145T] & ; 400^\circ C < T \end{cases}$
Aslani & Bastami [2011]	<p>For NSC:</p> $f'_{cT} = f'_c \begin{cases} [1.012 - 0.0005T \leq 1.0] & 20^\circ C \leq T \leq 100^\circ C \\ 0.985 + 0.0002T - 2.235 \times 10^{-6} T^2 + 8 \times 10^{-10} T^3 & 100^\circ C < T \leq 800^\circ C \\ 0.44 - 0.0004T & 900^\circ C \leq T \leq 1000^\circ C \\ 0 & T > 1000^\circ C \end{cases}$ <p>for HSC (siliceous aggregate), 55.2 MPa $\leq f'_c \leq$ 80 MPa :</p> $f'_{cT} = f'_c \begin{cases} [1.01 - 0.00068T \leq 1.0] & 20^\circ C \leq T \leq 200^\circ C \\ 0.935 + 0.00026T - 2.13 \times 10^{-6} T^2 + 8 \times 10^{-10} T^3 & 200^\circ C < T \leq 400^\circ C \\ 0.90 + 0.0002T - 2.13 \times 10^{-6} T^2 + 8 \times 10^{-10} T^3 & 400^\circ C < T \leq 800^\circ C \\ 0.44 - 0.0004T & 900^\circ C \leq T \leq 1000^\circ C \\ 0 & T > 1000^\circ C \end{cases}$ <p>80 MPa $\leq f'_c \leq$ 110 MPa :</p> $f'_{cT} = f'_c \begin{cases} [0.8 - 0.0005T \leq 1.0] & 20^\circ C \leq T \leq 500^\circ C \\ 0.96 - 0.0008T - 5.17 \times 10^{-7} T^2 + 4 \times 10^{-10} T^3 & 500^\circ C < T \leq 800^\circ C \\ 0.44 - 0.0004T & 800^\circ C \leq T \leq 1000^\circ C \\ 0 & T > 1000^\circ C \end{cases}$

The constitutive equations based on British Standard (BS) [8] divide the compressive strength of concrete degradation, the temperature of the three categories. BS assumes that the concrete does not degrade the strength up to a temperature of 100°C. The decrease in

compressive strength of concrete occurs linearly at temperatures above 100°C. Unlike the above constitutive equation, the model proposed by Li & Purkiss [11] only consists of a single equation, in which polynomial equations are used and generally used to different temperatures. Equation proposed by Hertz [10] is also similar to the equation by Li & Purkiss, namely the equation, but by giving a certain constant factors (T_1 , T_2 , T_8 , T_{64}) based on the type of aggregate used (see Table 1). The equation proposed by Kodur et al. [13], concrete compressive strength degradation split into three as BS models above. The first category is considered that up to a temperature of 100°C the concrete compressive strength decreased linearly although not significant, and the second category is a temperature of 100°C to 400°C the compressive strength of concrete is considered lost by 25%, and the third category ($T > 400^\circ\text{C}$) concrete compressive strength decreased linearly to the compressive strength of concrete control.

Another model is proposed by Aslani & Bastami [14] and it is proposed to be applied to normal and high-strength concrete. Constitutive equation is divided from the normal temperature category to temperature of 1000°C, and the temperature at the top of the concrete is considered not having residual compressive strength again. Constitutive equations in Table 1 will then be compared with experimental results of steel fibre concrete in this study.

3. EXPERIMENTAL PROGRAM

A comprehensive research on the experiment has described fully in the research report [16]. Experimental program includes the manufacture of test specimens of steel fibre concrete cylinder diameter of 100 mm and height 200 mm. The parameters reviewed are concrete compressive strength and temperature. Percentage of fibre that is used is 0.5% of the volume of concrete, because based on previous fibre concrete research shows that optimum conditions obtained on the percentage value [6]. The used steel fibre has a ratio of length to diameter of between 40 and 50.

Concrete mix design with the composition shown in Table 2. The specimens are designed into three categories of water cement ratio (w/c), namely 0.53, 0.38 and 0.3, with a target to achieve concrete quality at normal strength, intermediate (transition) and high strength. Aggregate used is from the city of Semarang, Fly Ash is from waste of Paton steam power plant. To maintain the concrete workability in order to remain well, especially for mixed w/c at 0.38 and 0.3, viscocrete a doses of 0.5% by weight of cement is used.

Table 2: Concrete mix design

Materials	Mixture I w/c=0.53	Mixture II w/c=0.38	Mixture III w/c=0.30
Cement (kg/m^3)	350	419.98	485
Fly Ash (kg/m^3)	-	74.11	82.83
Water (Lt/m^3)	200	160	140
Viscocrete 0,5% (lt/m^3)	-	6.228	9.28
Fine Aggregate (kg/m^3)	722.9	696.62	662.07
Coarse Aggregate (kg/m^3)	886.8	1044.93	1080.22

The casted specimens are subsequently grouped into control (temperature of 30°C/not burned), and the specimens are burned at a temperature of 300°C, 600°C and 900°C. Chamber incinerator with a size of 1.3×1.2×3.2 meter structure made of flint SK-32 is coated with heat resistant asbestos and then iron on the outside. In addition to adjust the amount according to the desired temperature, we mount thermometer on the bottom. In the combustion chamber there is a section for providers and the air vacuum, so that the results can be good combustion, not sooty and the color's change of concrete due to fire can be seen clearly. Figure 1 shows the combustion chamber arrangement of the test object.

Burning specimen are implemented after the concrete specimen reaches the age of 120 days. The tests to determine the compressive strength of concrete and the stress-strain behavior using UTM testing machine press with a capacity of 1000 kN. The testing system is Strain Control and loading speed is 0.01/sec.

Based on ASTM standard, standard cylinder compressive strength of diameter 150 mm, height 300 mm is obtained by making a correction factor of stress test results press cylinder 100/200 experiments at 0.95. The evaluation of the strength of concrete degradation is conducted by normalizing compressive strength specimen at a given temperature (f_{cr}) to the compressive strength of concrete at normal temperature (f_c).

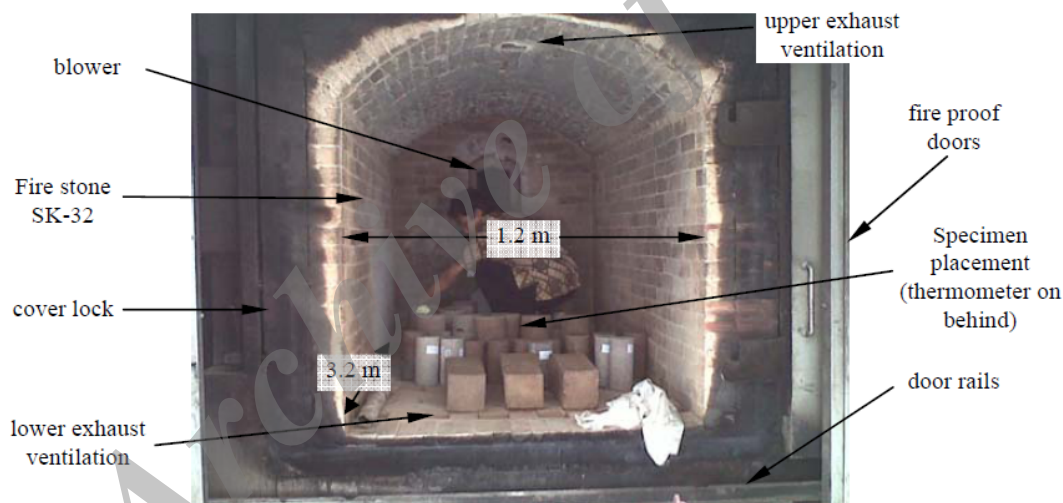


Figure 1. Detail of chamber incinerator and arrangement of the test object

4. EXPERIMENTAL RESULTS AND DISCUSSION

4.1 Compressive Strength at Elevated Temperatures

The test results shown in Table 3. Based on the Table, it has resulted three categories of fibre concrete compressive strength on average which is the implementation of a variety of water-cement ratio (w/c) on the mix design, the concrete compressive strength of 30.4 MPa, 51.1 MPa and 72.5 MPa. The definition based on Antonius & Imran [17], low or normal strength concrete is concrete with strength up to 41 MPa, concrete with strength from 41 to

55 MPa as medium strength concrete, and above 55 MPa as high-strength concrete. Furthermore, the third strength of concrete above is considered to represent the specimen as Normal-Strength Fibre Concrete (NSFC), Medium-Strength Fibre Concrete (MSFC) and High-Strength Fibre Concrete (HSFC).

The nature of the workability of concrete slump indicated from the value of these experimental results can be well preserved because of the addition of viscocrete. Although the HSC specimen produce a relatively small slump value, but that value can be deemed to be within the limits for the properties of concrete workmanship, and does not occur in the porous concrete produced.

Table 3: Concrete compressive strength test results of steel fibres

f_c' 28 days (MPa)	Specimens	Average slump (mm)	Peak strain and compressive strength of 120 days							
			T=30°C		T=300°C		T=600°C		T=900°C	
			ϵ_c'	f_c (MPa)	ϵ_{ct}'	f_{ct}' (MPa)	ϵ_{ct}'	f_{ct}' (MPa)	ϵ_{ct}'	f_{ct}' (MPa)
30.4	1	135	0.0032	30.86	0.0029	27.01	0.0024	14.01	0.0018	12.10
	2		0.0028	29.65	0.0032	26.75	0.0021	15.41	0.0023	10.83
	3		0.003	33.89	0.0031	28.03	0.0025	14.52	0.0016	12.10
	4		0.0027	31.46	0.0028	26.11	0.0024	13.50	0.0019	10.83
	5		0.0031	27.23	0.0029	26.75	0.0022	14.14	0.0027	12.36
	6		0.003	32.07	0.0028	28.03	0.0023	15.03	0.0027	11.21
	Average		0.0030	30.86	0.0030	27.11	0.0023	14.44	0.0022	11.57
51.1	1	130	0.0031	49.01	0.0029	42.68	0.0017	21.02	0.0019	17.83
	2		0.003	52.04	0.0032	49.04	0.0019	19.11	0.0018	17.20
	3		0.0033	54.46	0.0031	47.77	0.0022	20.00	0.0018	16.56
	4		0.0026	52.04	0.0032	48.28	0.0025	20.38	0.0017	17.58
	5		0.003	53.85	0.0028	48.66	0.0025	21.66	0.0019	17.32
	6		0.0032	47.02	0.0025	46.88	0.0024	19.75	0.002	16.94
	Average		0.0030	51.40	0.0030	47.22	0.0022	20.32	0.0019	17.24
72.5	1	60	0.0028	75.03	0.0027	58.60	0.0016	23.57	0.0016	19.11
	2		0.0031	71.40	0.0028	63.31	0.0016	24.84	0.0018	15.92
	3		0.0027	70.80	0.0028	57.32	0.0018	25.48	0.0017	18.85
	4		0.0028	76.85	0.0027	56.69	0.0017	25.10	0.0016	19.36
	5		0.0032	77.45	0.003	56.69	0.0016	24.84	0.0016	17.07
	6		0.003	69.59	0.0029	54.27	0.0017	26.11	0.0017	18.06
	Average		0.0029	73.52	0.0028	57.81	0.0017	24.99	0.0017	18.15

Figure 2 shows the effect of temperature changes on the compressive strength of concrete specimens with normal strength to high strength. As shown in the figure, a decrease in NSFC and MSFC (f_c' =30.4 MPa and f_c' =51.1 MPa) is burned at a temperature of 300°C approximately 10% of the compressive strength of the control specimens. However on specimens of HSFC compressive strength (f_c' =72.5 MPa) and the same temperature, the higher the compressive strength reduction is about 20%.

The degradation of concrete compressive strength is significant post-burn combustion 600°C, where the decline in the average compressive strength of concrete for compressive strength range is approximately 60-70% of the control specimens. The highest compressive

strength degradation occurs in high strength concrete with post-combustion temperature of 900°C, where the residual compressive strength that occurs is approximately 25% of the compressive strength of the control specimens.

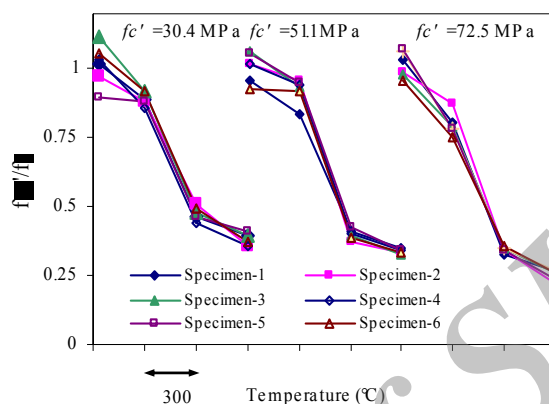


Figure 2. Compressive strength of steel fibre concrete to the temperatures

Figure 3, 4 and 5 show the comparison between models of degradation of concrete compressive strength at various temperatures with experimental results. The prediction of compressive strength degradation based on Kodur et al model is the lowest compared with predictions by other models, especially at a temperature of 300°C and 600°C, but at normal temperature and the temperature of 900°C, the estimation of the compressive strength of concrete based on all models is relatively the same. The experimental results on all steel-fibre concrete compressive strength are relatively close to the models predictions based Li & Purkis, Hertz, Aslani & Bastami and BS if they are at a normal temperature of 300°C and 600°C. At a temperature of 900°C predicted by all the models is obviously underestimating the experimental results. This phenomenon indicates that residual strength of steel fibre concrete is higher to the residual strength of normal concrete at very high temperature. It is also an indication that the steel fibres play a role in inhibiting the collapse of the concrete material totally.

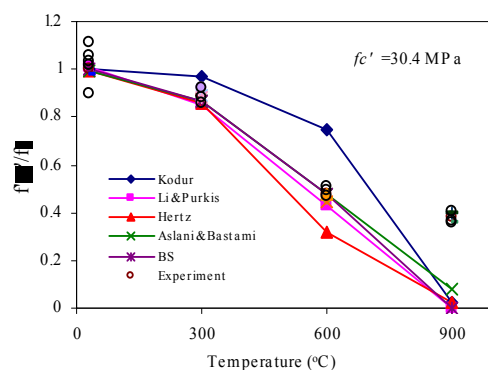


Figure 3. Comparison of compressive strength models at elevated temperatures with experimental results for $f'_c = 30.4 \text{ MPa}$

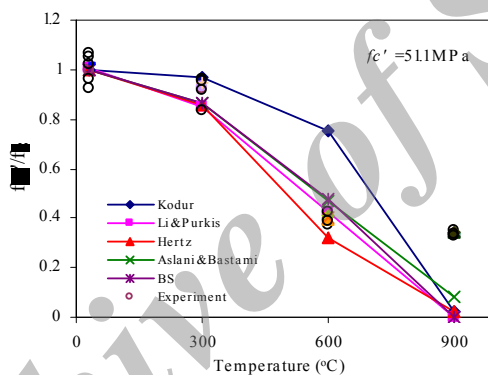


Figure 4. Comparison of compressive strength models at elevated temperatures with experimental results for $f'_c = 51.1 \text{ MPa}$

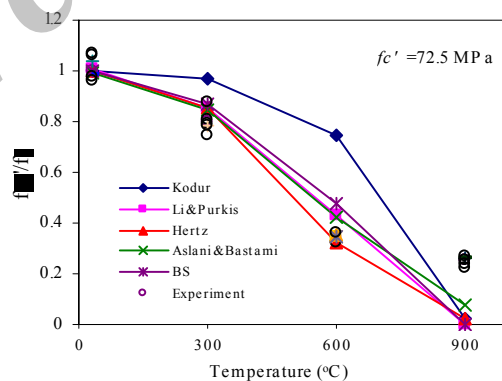


Figure 5. Comparison of compressive strength models at elevated temperatures with experimental results for $f'_c = 72.5 \text{ MPa}$

4.2 Stress-Strain Behaviour

4.2.1 Effect of temperature changes

Effect of temperature changes with the compressive strength of concrete remains on the stress-strain behavior shown in Figure 6. At a temperature of 300°C, initial specimen stiffness is generally equal to the stiffness of the specimens to normal temperature. As well as the degradation of concrete compressive strength of the control specimens in the previous discussion, the concrete stiffness is significantly decreased when the specimen happen to be at a temperature of 600°C and 900°C. The peak strain of NSFC specimens is relatively unchanged when the temperature increases, even though strain is not likely to increase significantly (see Table 3). The post-peak behavior indicates that the NSFC is more gentle curve when the temperature increases. This behavior indicates that the steel fibres are still working to improve the ductility in spite of a very high temperature. The concrete post-peak response in MSFC shows a significant decrease in ductility when the specimen is burned at a temperature of 600°C and 900°C (also decrease of peak strain). The same ductility behavior occurs in HSFC specimens. Sharp decrease in ductility at MSFC and HSFC ($T=600^{\circ}\text{C}$ and 900°C) shows that steel fibres are not working, so it can be said that the specimen behave like normal concrete (without fibre).

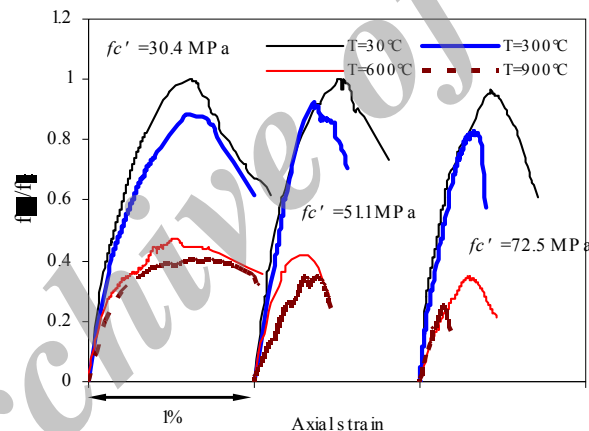


Figure 6. Stress-strain curves of steel fibre concrete with influence of temperatures

4.2.2 Effects of changes in compressive strength of concrete

Figure 7 is a comparison of the stress-strain behavior of steel fibre concrete at various temperatures by reviewing changes in compressive strength of concrete. In general there is the degradation of concrete compressive strength and ductility of fibre along with increasing the compressive strength of concrete being used. However, seen from the figure that the proficiency level of ductility behavior of normal concrete is still the highest quality and relatively increased despite the increased temperature. This is indicated by of post-peak curve flat/sloping of normal strength of concrete specimen compared with the curve on the compressive strength of concrete to another.

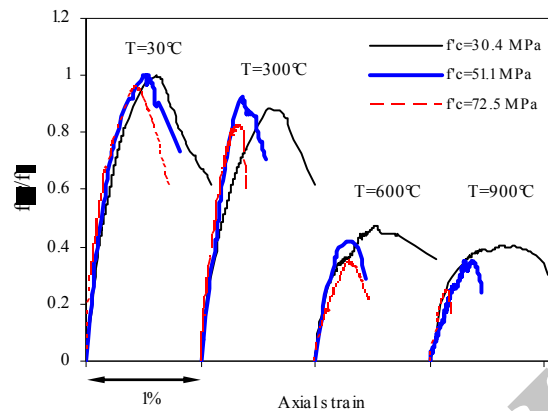


Figure 7. Stress-strain curves of steel fibre concrete with influence of compressive strength

5. STRESS-STRAIN MODEL OF STEEL FIBRE CONCRETE AT ELEVATED TEMPERATURES

Stress-strain model of steel fibre concrete at various temperatures is necessary in order to predict stiffness properties, the ability of fishermen and failure mechanisms with good material, and improves the understanding of the behaviour of the structures during fire events.

5.1 Modeling of the stress-strain

The experimental results indicate that there are significant differences in the behavior of compressive strength and ductility at normal strength and high strength specimens. So that in the stress-strain modeling of steel fiber concrete at elevated temperatures is divided into two concrete strength, which is the model for normal and medium concrete ($f'_c \leq 55$ MPa), and the model for high strength concrete ($f'_c > 55$ MPa). The stress-strain model for normal and medium strength fibre concrete adopted from equation proposed by Kodur et al. [13] is in Equation (1).

$$\left. \begin{aligned} f_c &= f_{cT}' \left[1 - \left(\frac{\epsilon_{cT}' - \epsilon_c}{\epsilon_{cT}'} \right)^H \right] \\ f_c &= f_{cT}' \left[1 - \left(\frac{30(\epsilon_c - \epsilon_{cT}')}{(130 - f_c')\epsilon_{cT}'} \right) \right] \end{aligned} \right\} \begin{array}{l} \vdots \\ \epsilon_c \leq \epsilon_{cT}' \\ \vdots \\ \epsilon_c > \epsilon_{cT}' \end{array} \quad H = 2.28 - 0.012f_c' \quad (1)$$

For high-strength concrete, the equation based on BS [8] is adopted, that is:

$$f_c = f_{cT}' \left[\frac{3\varepsilon_c f_{cT}'}{\varepsilon_{cT}' \left(2 + \left(\frac{\varepsilon_c}{\varepsilon_{cT}'} \right)^3 \right)} \right] \quad (2)$$

5.2 Peak Stress and Peak Strain at Elevated Temperatures

f_{cT}' value is derived by using data of concrete compressive strength test results in Table 3. The derived equations are divided into three categories of concrete compressive strength, among them are below 300°C, between 300°C and 600°C, above 600°C. The resulted equation based on the results of the linear regression equation is below.

- For normal and medium strength fibre concrete:

$$\left. \begin{aligned} \text{For } T < 300^\circ C &: \frac{f_{cT}'}{f_c'} = 1.0219 - 0.0004T \\ 300^\circ C \leq T < 600^\circ C &: \frac{f_{cT}'}{f_c'} = 1.3795 - 0.0016T \\ T \geq 600^\circ C &: \frac{f_{cT}'}{f_c'} = 0.591 - 0.0003T \end{aligned} \right\} \quad (3)$$

- For high-strength fibre concrete ($f_c' > 55$ MPa):

$$\left. \begin{aligned} \text{For } T < 300^\circ C &: \frac{f_{cT}'}{f_c'} = 1.0266 - 0.0006T \\ 300^\circ C \leq T < 600^\circ C &: \frac{f_{cT}'}{f_c'} = 1.3503 - 0.0016T \\ T \geq 600^\circ C &: \frac{f_{cT}'}{f_c'} = 0.5208 - 0.0003T \end{aligned} \right\} \quad (4)$$

Fibre concrete peak strain at various temperatures (ε_{cT}') was formulated based on the results of the linear regression.

- For normal and medium strength fibre concrete:

$$\left. \begin{aligned} \text{For } T < 300^\circ C &: \frac{\varepsilon_{cT}'}{\varepsilon_c'} = 1.0074 - 0.0002T \\ 300^\circ C \leq T < 600^\circ C &: \frac{\varepsilon_{cT}'}{\varepsilon_c'} = 1.225 - 0.001T \\ T \geq 600^\circ C &: \frac{\varepsilon_{cT}'}{\varepsilon_c'} = 0.8467 - 0.0003T \end{aligned} \right\} \quad (5)$$

- For high-strength fibre concrete ($f_c' > 55$ MPa):

$$\left. \begin{aligned}
 \text{For } T < 300^{\circ}C &: \frac{\varepsilon_{cT}}{\varepsilon_c'} = 1.089 - 0.0003T \\
 300^{\circ}C \leq T < 600^{\circ}C &: \frac{\varepsilon_{cT}}{\varepsilon_c'} = 1.41 - 0.0013T \\
 T \geq 600^{\circ}C &: \frac{\varepsilon_{cT}}{\varepsilon_c'} = 0.79 - 0.0003T
 \end{aligned} \right\} \quad (6)$$

The value of ε_c' for normal and medium-strength concrete assumed equal 0.005, and for high-strength concrete is 0.004.

5.3 Corroboration of the Model with Experimental Results

The validation of the model that have been developed on top is to do a comparison of the results of stress-strain experimental specimen at various temperatures. Figure 8, 9 and 10 show the comparison between the model with experimental results. The comparison shows that the curve before the peak relatively overlaps at various temperatures. In the post-peak response, stress-strain curves show a similar trend between the model with experimental results.

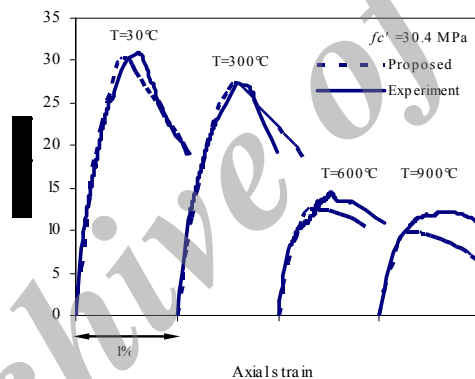


Figure 8. Comparison between proposed stress-strain model with experimental results for $f'_c = 30.4$ MPa

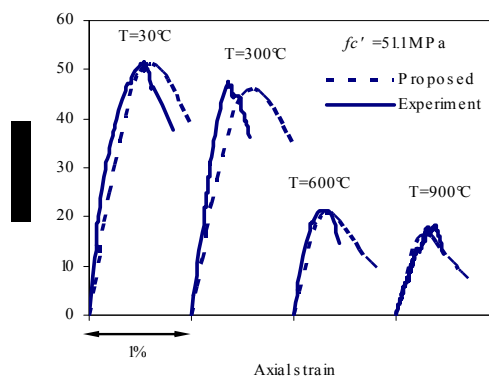


Figure 9. Comparison between proposed stress-strain model with experimental results for $f'_c = 51.1$ MPa

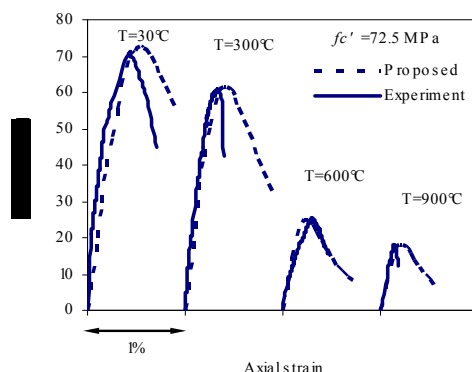


Figure 10. Comparison between proposed stress-strain model with experimental results for $f'_c = 72.5$ MPa

6. CONCLUSION

Steel fibre concrete tested in this study underwent relatively small compressive strength degradation when burned to a temperature of 300°C, but the significant degradation of compressive strength only occurred in the combustion temperature of 600°C and even 900°C. Generally on all combustion temperature, compressive strength degradation would be higher if the used compressive strength of the concrete increased, especially for high strength concrete where there was a strong decrease in the maximum at about 75% of the control specimens. At a temperature of 300°C and 600°C, the experimental results of concrete compressive strength of steel fibre had a value that does not vary much with the predictions based on the models by Li & Purkis, Aslani & Bastami and BS, but at a temperature of 900°C a significant deviation occurred. At the same compressive strength, stiffness of steel fibre concrete decreased when the temperature was going higher. Ductility of normal strength concrete with contains of steel fibre is relatively stable despite the increase of the temperature, but the ductility of medium and high strength of steel fibre concrete tends to decrease with increasing temperature. The stress-strain model of concrete with contains of steel fibre at various temperature levels were proposed to predict the behavior before and after the peak as well.

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NOTATIONS

ε_c = strain corresponding to the compressive stress
 ε'_c = peak strain corresponding to the compressive stress

ε_{cT}' = peak strain corresponding to the peak compressive strength both at elevated temperatures

f_c = concrete compressive stress at normal temperature

f_c' = concrete compressive strength at 28 days

f_{cT} = concrete compressive strength at elevated temperatures

f_{cT}' = peak stress of concrete at elevated temperatures

T = temperature

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