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CONSTITUTIVE MODELS OF REINFORCED CONCRETE FOR FIRE-DAMAGED SEISMIC EVALUATION

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ABSTRACT

The constitutive models for concrete and steel under fire environments were proposed previously. Based on the reinforced concrete panel tests performed at the University of Houston, the constitutive models for concrete and steel under seismic loading were developed recently. To evaluate the seismic performance of fire-damaged reinforced concrete structures, the constitutive models for fire-damaged reinforced concrete structures subjected to seismic loading are proposed in this paper and the algorithm for seismic response analysis of fire-damaged reinforced concrete structures is developed.

Introduction

Fire is one of the most frequently happen and severe destructive disasters in the world. After fire, especially in the seismic zones, such as California, Tokyo, Beijing, etc., to evaluate the seismic performance of the surviving structures is still a thorny problem. Currently, the most wildly used method of performing seismic analysis of structures is the finite element method, in which, determining the properties of material is the key basis. Degradation of mechanical behavior of structural materials due to exposure to high temperature has been studied since as early as the 1950s (Schneider 1988, Phan *et al* 1998). The properties of materials, including compressive strength, peak strain, elastic modulus, spalling of concrete (Lie *et al* 1986, Lu 1989, Yao 1991, Niu *et al* 1990, Li *et al* 1993, Yuan *et al* 2001, Wu *et al* 1999, Concrete Society 1990), yield strength, elastic modulus of steel (Lu 1989, Concrete Society 1990, Holmes *et al* 1982, Stevens 1966, Lv 1996, Shen *et al* 1991, Hua 2000) and the bond behavior between concrete and steel (Zhu *et al* 1991), have been investigated. Some relationships of concrete and steel after fire were proposed (Lie *et al* 1986, Lu 1989, Yao 1991, Niu *et al* 1990, Li *et al* 1993, Yuan *et al* 2001, Wu *et al* 1999).

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However, there still exist two problems for the evaluation of seismic performance of structures after fire. Firstly, most of the researches focused on the monotonic behavior of the fire damaged materials, which is not sufficient for seismic evaluation. Secondly, most proposed stress-strain relationships of concrete and steel were uniaxial models, which only can be used for frame type (1D) structures. Based on a large amount of test data (Mansour *et al* 2005), and following the rotating-angle softened truss model (RA-STM) (Hsu 1993), the fixed-angle softened truss model (FA-STM) (Pang *et al* 1996), the softened membrane model (SMM) (Hsu *et al* 2002), the University of Houston (UH) group reported the cyclic softened membrane model (CSMM) (Mansour 2001). This model can be used for either monotonic or cyclic analysis of wall type structures (2D). In the monotonic analysis, compared to other models, CSMM can satisfactorily predict the entire response curves, including both the ascending and descending branches. Additionally, in the cyclic analysis, the model can not only rationally predict the cyclic shear force–displacement behavior of walls where shear deformation is significant, but also can accurately predict the pinching effect, the shear ductility and the energy dissipation capacities of the membrane element (Mo *et al* 2008). Combining the research results of fire damaged materials and CSMM, the Fire Damaged Cyclic Softened Model (FD-CSMM) is proposed in this paper. Furthermore, utilizing the software named "*OpenSees*" as a platform, FD-CSMM can be successfully employed for the seismic evaluation of frame and wall structures.

Effect of fire on concrete

The properties of concrete at room temperature (normally, 20^{o}C) and after fire are totally different; which depend on the concrete grades, type of aggregate, mix proportions, curing conditions, heating parameters and cool regimes. Numerous experimental studies to investigate the properties of concrete under fire or after fire have been undertaken by many researchers worldwide, focusing mainly on the variations of compressive and tensile strength, peak strain and elastic modulus.

Strength of concrete

The experiments of Lu (1998) and Yao (1991) indicate that the concrete with different aggregate does not behave the same with regard to its strength degradation under high temperature. However, below 500 ^{o}C the difference between concrete with siliceous aggregate and that with calcium aggregate is negligible (Niu *et al* 1990, Li *et al* 1993). According to Yao (1991) and Zhu *et al* (1991), the effects of cooling regime have great influence to the strength of concrete after fire. When the elevated temperature is below 400 ^{o}C, the strength degradation of concrete cooled with water is greater than that when cooled in air, whereas above 600 ^{o}C the influence of cooling regime becomes indistinct. Furthermore, Yuan *et al* (2001) and Wu *et al* (1999) investigated the influence of concrete types on the relative axial compressive strength at elevated temperature. It is found that the mechanical behavior of steel fiber concrete (SFC) at elevated temperature is better than that of plain concrete, and the compressive strength of high strength concrete (HSC) with and without polypropylene fiber is almost the same after high temperature, regardless of the elevated temperature. Besides, due to its dense microstructure, HSC tends to spall in fire and the strength loss is more obvious after a regime of higher temperature. In general, as the temperature increases from room temperature to 400°C, the compressive strength of concrete initially drops slightly and then increases to some extent. For the sake of simplicity, this small variation of strength with temperature can be ignored and considered as a constant. The compressive strength of concrete starts to degrade drastically as the temperature reaches above 400°C. At 800°C, it drops to less than 20% of its value at room temperature. Hence, the following expression (proposed model) can be employed to describe the compressive strength of concrete after fire.

$$f_0(T) = \begin{cases} f_0 & \left(0^0 C \le T \le 400^0 C\right) \\ (\alpha - \beta T) f_0 & \left(400^0 C < T \le 800^0 C\right) \end{cases} \tag{1}$$

In which, $f_0(T)$ and f_0 are the concrete compressive strengths at elevated and room temperature, respectively, α, β are the material parameters, for normal strength concrete $\alpha = 1.76, \beta = 0.0019$, for high strength concrete $\alpha = 2.2, \beta = 0.003$.

Peak strain in concrete

The peak strain in concrete increases linearly with temperature (Lu 1989, Yao 1991). Moreover, the relative compressive peak strain in concrete is greater for concrete cooled rapidly with water than that cooled down slowly in the air. In the case of high performance concrete with blast-furnace-slag, Xiao *et al* (2003) reported that the compressive peak strain in concrete was approximately equal to that of NSC below 400 °C, while beyond 400 °C, it was clearly greater than that of NSC. In this paper, the following expression is proposed to express the compressive peak strain of concrete after fire.

$$\varepsilon_0(T) = (1 + 0.0018T)\varepsilon_0 \tag{2}$$

In which, $\varepsilon_0(T)$ and ε_0 are the compressive peak strain at elevated and room temperature, respectively.

Effect of fire on steel reinforcement

The strength (including yield strength and ultimate strength) and the deformation performance (including stress-stain relationship and elastic modulus) are the two main topics highlighted in the research of mechanical behavior of steel reinforcement (i.e. rebar), both under and post high temperature.

Yield strength

According to fire tests (Lu 1989, Li *et al* 1993, Shen *et al* 1991, Hua 2000), it was found that the strength of low-carbon, low-alloy and cold-worked rebar decreases with the increase in temperature. The yield plateau of low-carbon rebar is gradually shortened as the temperature rises and it almost disappears when the temperature is near 300 ^oC. As the temperature increases from room temperature to 400 ^oC, the strength of low-carbon (plain) and low-alloy (deformed) rebar increases a little, but the ductility decreases because of the *blue brittleness* ranging from 200 ^oC to 350 ^oC. While above 400 ^oC, the strength decreases monotonically up to only 20% of strength at room temperature remains when the temperature reaches 700 ^oC. The possible reason could be that it recovers some strength when the temperature drops to room temperature. Compared with the concrete degrading distinctively above 400 ^oC, it seems that the threshold of high temperature influence on rebar is at 300 ^oC. Zhu *et al* (1991) indicated that the relationship between yield and ultimate strength of rebar below 600 ^oC was almost the same as that at room temperature. To describe the above phenomenon of steel rebar, the following expression can be employed (proposed model).

$$f_y(T) = \begin{cases} f_y & \left(0^0C \le T \le 350^0C\right) \\ (1.574 - 0.00164T)\,f_y & \left(350^0C < T \le 800^0C\right) \end{cases} \tag{3}$$

Where, $f_y(T)$ and f_y are the yield strength of steel rebar at elevated and room temperature.

Modulus of elasticity

Previous research (Lu 1989, Lv 1996, Hua 2000) demonstrates that the elastic modulus of steel rebar gradually decreases as the temperature increases under fire, which is much similar to the behavior of yield strength degradation of steel rebar under fire. Experimental results (Lv 1996) indicate that the elastic modulus of rebar under high temperature is independent of the various types of steel rebars and could generally be expressed (proposed model) as shown in Eq.(4) where, $E_s(T)$ and E_s denote the elastic modulus of steel rebar at elevated and room temperature, respectively.

$$E_s(T) = \begin{cases} (1 - 0.000486T)E_s & \left(0^0C \le T \le 370^0C\right) \\ (1.515 - 0.001879T)E_s & \left(370^0C < T \le 700^0C\right) \\ 0.2E_s & \left(T > 700^0C\right) \end{cases} \tag{4}$$

Temperature gradient along the cross-section

The first step in evaluation of structure is to determine the temperature at all points in a specimen cross-section, i.e. the temperature gradient along the cross-section, based on the foundational theory of differential equation for heat conduction (Zang *et al* 1994). To solve the equation, the relationship between temperature and material parameters including density ρ_c, specific heat C_c, and coefficient of heat conduction λ_c must be determined. Variation in the density of concrete is negligible with temperature, normally, it can be taken as a constant of $\rho_c = 2200 \sim 2400 kg/m^3$. The specific heat of concrete gradually increases with an increase in temperature, and the effect of the type of aggregates on this property is very small. Eurocode (1990) proposed a uniform expression for the specific heat of different concretes as follows:

$$C_c = 900 + 80\left(\frac{T}{120}\right) - 4\left(\frac{T}{120}\right)^2 \qquad 20^0C \le T \le 1200^0C \tag{5}$$

The coefficient of heat conductivity decreases with an increase in temperature, and the type of aggregate has obvious effect to this property. Eurocode (1990) proposed three different expression for three different aggregates as shown below:

$$\text{Siliceous aggregate} \quad \lambda_c = 2 - 0.24\left(\frac{T}{120}\right) + 0.012\left(\frac{T}{120}\right)^2 \qquad 20^0C \le T \le 1200^0C \tag{6a}$$

$$\text{Calcareous aggregate} \quad \lambda_c = 1.6 - 0.16\left(\frac{T}{120}\right) + 0.008\left(\frac{T}{120}\right)^2 \qquad 20^0C \le T \le 1200^0C \tag{6b}$$

$$\text{Lightweight aggregate} \quad \lambda_c = \begin{cases} 1.0 - \dfrac{T}{1000} & 20^0C \le T \le 800^0C \\ 0.5 & 800^0C < T \le 1200^0C \end{cases} \tag{6c}$$

The equation for heat conduction for a specimen under fire can be expressed as:

$$\frac{\partial T}{\partial \tau} = \alpha\left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2}\right) \tag{7}$$

In which, $T\left({}^0C\right)$ is temperature, $\tau\left(hour\right)$ is the time the specimen remains under fire, x, y, z are the coordinate. $\alpha = \lambda_c / C_c\rho_c$. In a 3D problem, the temperature field in a specimen cross-section can be solved through numerical method. The commonly used method includes finite difference method and finite element-finite difference hybrid method. In this paper, only the case of one and two surfaces of specimen under fire are considered, so the equation of heat conduction can be simplified as:

$$\frac{\partial T}{\partial \tau} = \alpha \frac{\partial^2 T}{\partial x^2} \tag{8}$$

The boundary conditions are: when $\tau = 0$, $T\left(x,0\right) = T_0$; where $x = 0$, $T\left(0,\tau\right) = T_s$; where $x = h$, $T\left(h,\tau\right) = T_0$. Then the Eq.Then the Eq.(8) can be solved directly as shown below:

$$T\left(x,\tau\right) = T_s - \left(T_s - T_0\right) erf \frac{x}{2\sqrt{\alpha\tau}} \tag{9}$$

In which, $T(x,\tau)$ is the temperature along the coordinate direction-x (The origin of coordinate is set at the face that is under fire), T_s is the surface temperature (fire temperature), T_0 is the room temperature. The error function, *erf(x)* can be expressed by Taylor's series as:

$$erf\,(x)=\frac{2}{\sqrt{\pi}}\sum_{n=0}^{\infty}\frac{(-1)^n x^{2n+1}}{(2n+1)n!}=\frac{2}{\sqrt{\pi}}\left(x-\frac{x^3}{3}+\frac{x^5}{10}-\frac{x^7}{42}+\frac{x^9}{216}-\cdots\right) \tag{10}$$

In the practical application, five terms of the series can be considered yielding sufficient accuracy. When the distance from the point in the cross section to the surface where fire occurs is less, the theoretical result are more consistent to the experiment. The reason is that, as mentioned before, the solution of the equation of heat is based on a lot of assumed conditions. However, since the divergence only happens in the inner section, the effect on the accuracy of evaluation will be very small.

Fire damaged cyclic softened membrane model

Similar to the Cyclic Softened Membrane Model (CSMM), in the fire damaged CSMM (FD-CSMM) , the cracked reinforced concrete is assumed to be a continuum material in a smeared crack model. The material properties are characterized by a set of smeared (or average) stress–strain relationships for concrete and steel, established directly from full-scale element tests subjected to biaxial loading. An important advantage of using the averaging concept in tension stiffening is that the tensile stress–strain relationship of concrete becomes mesh independent (Mo *et al* 2008). A material stiffness matrix relates the state of stresses and strains for an element. When the structure is fire damaged, it needs to consider the effect of temperature on the properties of concrete and steel as discussed in the above sections. For the implementation procedure in this paper, a tangent material stiffness matrix *[D^T]* for a fire damaged reinforced concrete element is formulated as:

$$\left[D^T\right]=d\left\{\boldsymbol{\sigma}^T\right\}/d\left\{\boldsymbol{\varepsilon}^T\right\} \tag{11}$$

$\left[D^T\right]$ is evaluated by:

$$\left[D^T\right]=\left[T(-\theta_1)\right]\left[D_c^T\right][V]\left[T(\theta_1)\right]+\sum_i\left[T(-\theta_{si})\right]\left[D_{si}^T\right]\left[T(\theta_{si}-\theta_1)\right][V]\left[T(\theta_1)\right] \tag{12}$$

where $\left[D_c^T\right]$ is the uniaxial tangential constitutive matrix of fire damaged concrete, and $\left[D_{si}^T\right]$ is the uniaxial tangential constitutive matrix of fire damaged steel; Both of these matrices can be obtained from their uniaxial constitutive relationships.

Uniaxial constitutive relationship of concrete

The cyclic smeared uniaxial constitutive relationships of concrete in compression and tension are given in Fig. 1 (Mo *et al* 2008). The characteristics of these concrete constitutive laws include: (1) the softening effect on the concrete in compression due to the tensile strain in the perpendicular direction; (2) the softening effect on the concrete in compression under reversed cyclic loading; (3) the opening and closing of cracks, which are taken into account in the unloading and reloading stages:

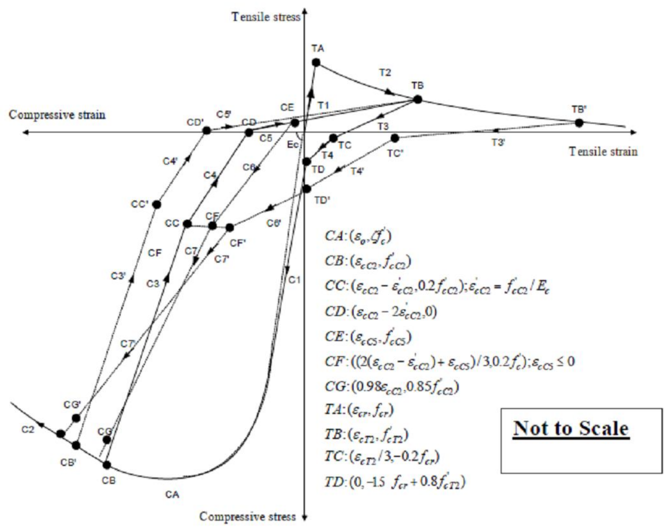

Figure 1. Cyclic smeared stress–strain curve of concrete

(Stage C1) $\sigma_c(T)=\left(D(T)\xi(T)f_c^{'}(T)-f_{cT4}^{'}\right)\left[2\left(\frac{\bar{\varepsilon}(T)}{\xi(T)\varepsilon_0(T)}\right)-\left(\frac{\bar{\varepsilon}(T)}{\xi(T)\varepsilon_0(T)}\right)^2\right]+f_{CT4}^{'}$

$0\le\left|\bar{\varepsilon}(T)\right|\le\left|\xi(T)\varepsilon_0(T)\right|$ (13a)

(Stage C2)

$\sigma_c(T)=D(T)\xi(T)f_c^{'}(T)\left[1-\left(\frac{\bar{\varepsilon}(T)/\varepsilon_0(T)-1}{4/\xi(T)-1}\right)^2\right]$ $\left|\bar{\varepsilon}(T)\right|>\left|\xi(T)\varepsilon_0(T)\right|$ (13b)

(Stage T1) $\sigma_c(T)=E_c(T)\bar{\varepsilon}(T)$ $0\le\bar{\varepsilon}(T)\le\varepsilon_{cr}(T)$ (13c)

(Stage T2) $\sigma_c(T)=f_{cr}(T)\left(\frac{\varepsilon_{cr}(T)}{\bar{\varepsilon}(T)}\right)^{0.4}$ $\bar{\varepsilon}(T)>\varepsilon_{cr}(T)$ (13d)

Unloading and reloading curves

$$\sigma_c^T = \sigma_i^c + E_{cc}\left(\bar{\varepsilon}_i - \bar{\varepsilon}\right) \qquad E_{cc} = \frac{\sigma_i^c - \sigma_{i+1}^c}{\bar{\varepsilon}_i - \bar{\varepsilon}_{ii}} \tag{13e}$$

Where, $f_c^{'}(T)$ and $\varepsilon_0(T)$ are the cylinder compressive strength and peak strain of concrete after fire damage with the temperature of $T\left({}^0C\right)$, which can be obtained through Eq.(1) and Eq.(2). $f_{CT4}^{'}$ is the stress at point TD. Additionally,

$$\xi(T) = \frac{5.8}{\sqrt{f_c^{'}(T)(MPa)}}\frac{1}{\sqrt{1+400\bar{\varepsilon}_T^{'}/\eta^{'}(T)}} \le 0.9,\quad D = 1-0.4\frac{\varepsilon_c^{'}}{\varepsilon_0(T)} \le 1.0 \text{ and}$$

$$\eta^{'}(T) = \eta(T) = \frac{\rho_t f_{ty}(T) - \sigma_t}{\rho_t f_{ly}(T) - \sigma_l} \le 1.0$$

In this paper, the effect of high temperature to the elastic modulus and the cracking strength of concrete are considered through $E_c(T) = 3875\sqrt{f_c^{'}(T)(MPa)}$ and $f_{cr}(T) = 0.31\sqrt{f_c^{'}(T)(MPa)}$ respectively, $\varepsilon_{cr} = \varepsilon_{cr}(T) = 0.00008$.

Uniaxial constitutive relationship of mild steel rebar embedded in concrete

The cyclic smeared uniaxial constitutive relationships of fire damaged embedded mild steel rebar was developed by (Mansour *et al* 2001). The constitutive relationships are given in Fig. 2. The smeared stress of embedded mild steel rebar is lower than the yield stress of bare steel rebar and the hardening ratio of steel rebar after yielding is calculated from the steel ratio, steel strength and concrete strength. The unloading and reloading stress–strain curves of embedded steel rebar take into account the *Bauschinger* effect.

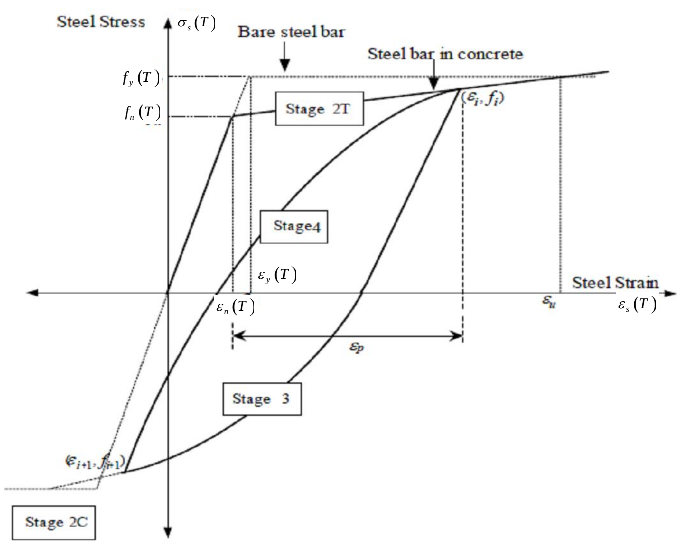

Figure 2. Cyclic smeared stress–strain curve of mild steel rebar

$$\text{(Stage 1)} \quad \sigma_s(T) = E_s(T)\bar{\varepsilon}_s(T) \qquad \left(\bar{\varepsilon}_s(T) \le \bar{\varepsilon}_n(T)\right) \tag{14a}$$

$$\text{(Stage 2T)} \quad \sigma_s(T) = f_y(T)\left[(0.91-2B)+\left(0.02+0.25B\frac{\bar{\varepsilon}_s(T)}{\varepsilon_y(T)}\right)\right] \quad \left(\bar{\varepsilon}_s(T) > \bar{\varepsilon}_n(T)\right) \tag{14b}$$

$$\text{(Stage 2C)} \quad \sigma_s(T) = -f_y(T) \qquad \left(\sigma_s(T) \le -f_y(T)\right) \tag{14c}$$

$$\text{(Stage 3 \& Stage 4)} \quad \bar{\varepsilon}_s(T) - \bar{\varepsilon}_{si}(T) = \frac{\sigma_s(T)-\sigma_i(T)}{E_s(T)}\left[1 + A^{-R}\left|\frac{\sigma_s(T)-\sigma_i(T)}{f_y(T)}\right|^{R-1}\right] \tag{14d}$$

Where, $f_y(T)$ and $E_s(T)$ are the yield strength and elastic modulus of steel rebar after fire damage with the temperature of $T\left({}^0C\right)$, respectively, which can be obtained through Eq.(3) and Eq.(4). Also,

$$B = \frac{1}{\rho}\left(\frac{f_{cr}(T)}{f_y(T)}\right)^{1.5} \text{ and } \rho \ge 0.23\%, \ \bar{\varepsilon}_n(T) = \varepsilon_y(T)(0.93-2B), \ A = 1.9k_p^{-0.1} \quad R = 10k_p^{-0.2},$$

and $$k_p = \frac{\bar{\varepsilon}_p}{\bar{\varepsilon}_n(T)} = \frac{\varepsilon_i - \varepsilon_n}{\varepsilon_n(T)}$$

Constitutive relationship of concrete in shear

The relationship between the shear stress ($\tau_{12}^c(T)$) and shear strain ($\gamma_{12}^c(T)$) of fire damaged concrete in the 1-2 coordinate system can be expressed as:

$$\tau_{12}^c(T) = G_{12}^c(T)\frac{\gamma_{12}(T)}{2} \tag{15a}$$

Where, $$G_{12}^c(T) = \frac{\sigma_1^c(T)-\sigma_2^c(T)}{\varepsilon_1(T)-\varepsilon_2(T)} \tag{15b}$$

Implementation of FD-CSMM

In the Eq.(11), the uniaxial constitutive matrix of concrete $\left[D_c^T\right]$ is given by

$$\left[D_c^T\right] = \begin{bmatrix} \bar{\bar{E}}_1^c(T) & \frac{\partial\sigma_1^c(T)}{\partial\bar{\varepsilon}_2(T)} & 0 \\ \frac{\partial\sigma_2^c(T)}{\partial\bar{\varepsilon}_1(T)} & \bar{\bar{E}}_2^c(T) & 0 \\ 0 & 0 & G_{12}^c(T) \end{bmatrix} \tag{16}$$

Where $\overline{\overline{E}}_1^c(T)$ and $\overline{\overline{E}}_2^c(T)$ are the tangential stiffness of uniaxial moduli of fire damaged concrete in the 1 and 2 directions evaluated at a stress/stain state. Off-diagonal terms $\frac{\partial \sigma_1^c(T)}{\partial \bar{\varepsilon}_2(T)}$ and $\frac{\partial \sigma_2^c(T)}{\partial \bar{\varepsilon}_1(T)}$ can be obtained by using the uniaxial constitutive relationships and taking into account the states of the concrete stresses and uniaxial strains in the 1-2 directions. $G_{12}^c(T)$ is the shear modulus of concrete give by Eq.(15b).

The uniaxial constitutive matrix of steel rebars $\left[D_{si}(T)\right]$ is evaluated as;

$$\left[D_{si}(T)\right] = \begin{bmatrix} \rho_{si}\overline{\overline{E}}_{si}(T) & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{17}$$

Where $\overline{\overline{E}}_{si}(T)$ is the uniaxial tangential modulus of the fire damaged rebars, which is determined for a particular stress-strain state. For FD-CSMM, the algorithm is quite the same as that for the CSMM (Mo *et al* 2008). In each interaction, the fire damaged material stiffness matrix *[D(T)]*, the element stiffness matrix *[k_e]* and the global stiffness matrix *[K]* are iteratively refined until convergence is achieved. Furthermore, in order to implement the FD-CSMM, a finite element analysis software - "*OpenSees*", is used as a platform. Three new material modules, namely *SteelZ01T*, *ConcreteZ01T*, *RCPlaneStressT* and two element modules, namely, *nonlinearBeamColumnT* and *quadT* have been developed through this research work. *SteelZ01T* and *ConcreteZ01T* are the uniaxial material modules, in which the uniaxial constitutive relationships of fire damaged steel and concrete specified in the FD-CSMM are defined. The *nonlinearBeamColumnT* element is the fiber section beam type element. Compared to the normal fiber section beam element, *nonlinearBeamColumnT* has one additional step that is in determining the temperatures of all fibers in section using Eq.(9). The *RCPlaneStressT* is implemented with the quadrilateral element *quadT* to represent the four-node fire damaged reinforced concrete membrane elements.

To carry out the seismic analysis of frame type structures, the materials properties of steel and concrete can be determined directly by *steelZ01T* and *concreteZ01T*. In wall type structures, uniaxial materials of *steelZ01T* and *concreteZ01T* are related with material *RCPlaneStressT* to determine the fire damaged material stiffness matrix of membrane reinforced concrete in *RCPlaneStressT*. Using the *OpenSees* as the finite element framework, a nonlinear finite element program titled Simulation of Fire Damaged Concrete Structures (SFDCS) was developed for the simulation of reinforced concrete structures subjected to monotonic and reversed cyclic loading.

Conclusions

In this paper, combining the previous research on fire damaged concrete and steel, and the cyclic softened membrane model, the fire damaged cyclic softened membrane model is proposed, which can be used not only for the seismic evaluation of frame structures after fire, but also for the shear wall structures. In the model, the effect of high temperature on the compressive strength and peak strain of concrete; yield strength and elastic modulus of steel are taken into account. Based on the platform of *OpenSees*, three materials and two element modules were developed, which will be very convenient for the seismic analysis of RC structures after fire. Since the temperature gradient is directly considered in the element modules, it is no longer needed to simulate the temperature field before the seismic analysis.

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