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Ultimate Resisting Capacity of Reinforced Concrete Members Damaged under Fire

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ABSTRACT

This paper introduces a numerical method to evaluate ultimate resisting capacity of Reinforced Concrete (RC) members damaged in fire, and the introduced numerical approach is composed of two analysis procedures of the heat transfer analysis and proceeding nonlinear structural analysis. Material nonlinearities including cracking of concrete and yielding of steel are considered on the basis of the layered fiber section approach which can effectively take into account the change in material properties depending on the temperature. In advance, non-mechanical strains such as the thermal, the creep strain in concrete and steel and transient strain in concrete are implemented in defining the strain field. Correlation studies between experimental data and numerical results are performed to verify the performance of the introduced numerical method and, moreover, comparison with design codes is also performed to check the relative conservativeness in design codes.

INTRODUCTION

Generally, Concrete is one of the fire-resisting materials due to low thermal conductivity but, nevertheless, RC members could be damaged by high temperature. Thus, it is important to understand the behavior of RC members under fire conditions. Some researchers conducted full-scale fire experiments to investigate behavior of RC members under fire conditions (Lin et al, 1981). At that time, numerous material models were also developed to represent material properties of concrete & steel in the high temperature such as the thermal, the creep, transient strain and stress-strain relationship in concrete or steel (Poh, K.W, 2001; Youssef, 2007; Kodur, 2010). After then, numerical models were developed for predicting structural behavior of RC members subjected to fire conditions (Terro, 1998; Bratina, 2007; Kodur, 2008) and, they were mostly focused on correlation studies between experimental data and numerical results to verifying their numerical models. In this paper, a numerical method to evaluate RC members subjected to high temperature by using finite element procedure are introduced and it is validated by performing the correlation study between experimental data and numerical results like other researchers. Furthermore, comparison between design codes and numerical results are conducted to check the relative conservativeness for column's dimensions that are given in Eurocode2.

MATERIAL MODEL

Concrete

The total strain in concrete is expressed as the sum of mechanical strain and non-mechanical strains that are given as the sum of the thermal strain, the creep and the transient strain at any fire exposure time. The thermal strain is based on the Eurocode2. The creep strain is based on Harmathy's model using the Eq.(1) where $\beta_1 = 6.28 \times 10^{-6}\text{sec}^{-0.5}$, $d = 2.658 \times 10^{-3} K^{-1}$(Bratina et al., 2007), f_{cT} is compressive strength and σ is stress at elevated temperature. The transient strain is based on the relationship proposed by Anderberg and Thelandersson using the Eq.(2) where, f_c is compressive strength at ambient temperature and k_2 is dimensionless constant whose ranges are from 1.8 to 2.35. (Youssef et al., 2007).

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The stress-strain relationship of concrete, at elevated temperature, is considered in the model based on Lie & Lin's model using the following expression Eq.(3), where $\varepsilon_{co,T}$, whose model is proposed by khennane & baker, is strain at peak stress at elevated temperature. (Youssef et al., 2007)

$$\varepsilon_{cr,c} = \beta_1 \frac{\sigma}{f_{cT}} \cdot \sqrt{t} \cdot e^{d(T \ -293)} \tag{1}$$

$$\Delta\varepsilon_{tr,c} = k_2 \frac{\sigma}{f_c} \cdot \Delta\varepsilon_{th} \tag{2}$$

$$\sigma_{c,T} = f_{cT}\left[1 - (\tfrac{\varepsilon_{co,T}-\varepsilon_{c,T}}{\varepsilon_{co,T}})^2\right],\ (\varepsilon_{c,T} \le \varepsilon_{co,T}) \ , \quad \sigma_{c,T} = f_{cT}\left[1 - (\tfrac{\varepsilon_{co,T}-\varepsilon_{c,T}}{3\varepsilon_{co,T}})^2\right],\ (\varepsilon_{c,T} \ge \varepsilon_{co,T}) \tag{3}$$

Steel

The total strain in steel is also expressed as the sum of mechanical strain and non-mechanical strains like concrete and non-mechanical strains are given as the sum of the thermal strain and the creep strain at any fire exposure time. The thermal strain and stress-strain relationship of steel at elevated temperature are based on Eurocode2 as shown in following Fig. 1.

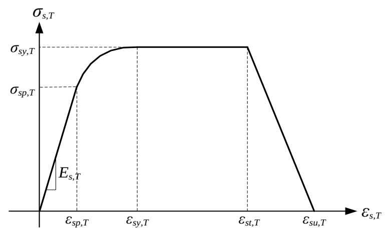

Fig 1. Stress-strain curve of Steel

The thermal creep is not considered explicitly due to the fact that stress-strain relationship in Eurocode 2 partly takes high-temperature creep effect into account. Thermal creep effect in the stress-strain curve is expressed as the flexible strain range between proportional limit ($\varepsilon_{sy,T}$) and yield limit ($\varepsilon_{sp,T}$) (Kodur et al., 2010). More details about stress-strain relationship or reduction factor of the properties of steel at elevated temperature can be found in the refered design code (EC2, 2004)

TRANSIENT HEAT TRANSFER ANALYSIS

The temperature, at any fire exposure time, is determined by using the time-temperature relationship such as ASTM E199 or ISO834 curve and a section of member is divided into a number of fiber layers for the analysis. The heat transfer analysis where a section surface is assumed to have uniform temperature along the length is conducted. The governing equation for transient heat transfer analysis in concrete is written like Eq.(4) where $k(T)$ is thermal conductivity, is density, and $c(T)$ is specific heat and, their expressions are based on Eurocode2. The heat of hydration and are not taken into account.

$$k(T)\left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2}\right) = \rho(T)c(T)\frac{\partial T}{\partial t} \tag{4}$$

$$k(T)\left(\frac{\partial T}{\partial x} + \frac{\partial T}{\partial y}\right) \cdot n_i = h_{eff}(T) \cdot (T_e - T_s) \tag{5}$$

The Boundary conditions due to temperature difference between a heat source and structure member can be expressed as Eq.(5) where, n is normal vector, h is convection coefficient, T_e is ambient temperature and T_s is member temperature. More details about the FE formulation procedures for the heat transfer analysis can be found in elsewhere (Kwak et al., 2011).

NONLINEAR ANALYSIS

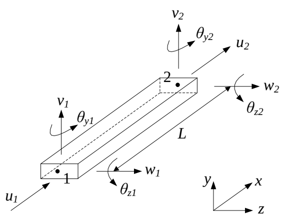

Fig 2. Beam element

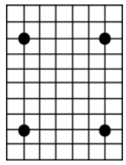
Fig 3. Fiber section

The beam element with 5 DOF at each node in local coordinate system is adopted. The degrees of freedom in the element can be expressed as vector $\mathbf{r} = \{u, v, \theta_z, w, \theta_y\}$ in Fig 2. A typical RC section is divided into virtual fiber layers to describe temperature and stress distribution under fire condition (see Fig 3.) (Kwak et al., 2010). Through the principle of virtual work, the equilibrium equations, at any time j, can be expressed as below Eq.(6). Substituting equivalent forces due to non-mechanical strains in Eq.(7) into Eq.(6), finally, Eq.(8) can be obtained where $K = \int_V \boldsymbol{B}^T E_T \boldsymbol{B}\, dV$, $\mathbf{B}$ is displacement-strain relationship and E_T is the modulus of elasticity at elevated temperature. Additional details about the non-linear analysis procedures can be found in elsewhere (Kwak et al., 2010).

$$d\boldsymbol{R}^j = \left(\int_V \boldsymbol{B}^T E_T \boldsymbol{B}\, dV\right) \cdot d\boldsymbol{r} - \int_V \boldsymbol{B}^T E_T d\varepsilon^{nm}\, dV \tag{6}$$

$$d\boldsymbol{R}^{nm} = \int_V B^T E_T d\varepsilon^{nm}\, dV \tag{7}$$

$$d\boldsymbol{R} = d\boldsymbol{R}^{\boldsymbol{j}} + d\boldsymbol{R}^{nm} = K d\boldsymbol{r} \tag{8}$$

NUMERICAL ANALYSIS

Verification

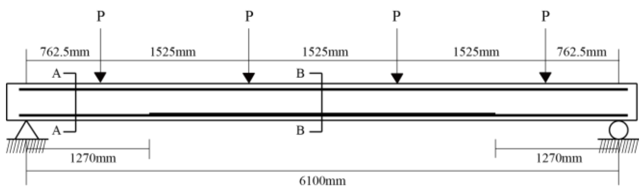

Fig 4. RC Beam (Lin et al)

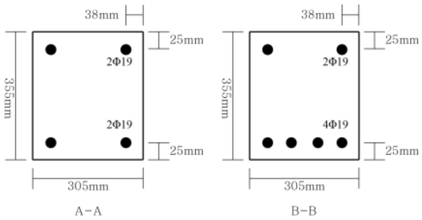

Fig 5. Cross section

The validity of the numerical analysis results is confirmed through comparing them with experimental results of fire test on the RC beam performed by Lin et al(1981). The geometries are given in Fig 4. and Fig 5. The compressive strength of concrete was 30MPa and the yield stress of steel was 435MPa with the value of 2×10^5MPa for the modulus of elascity at normal temperature. The beam was exposed to high temperature from bottom & two sides of section according to ASTM fire curve. Emissivity and the convection coefficient are assumed to be 0.2~0.3 and 30W/mK respectively. As results of the analysis, It is shown that the deflections under fire are properly predicted as compared with data from test (see Fig 6) and the ultimate strength decreases over time as with ultimate strength determined by ACI216 and 500'C isotherm methods (Eurocode2) as shown in see Fig 7.

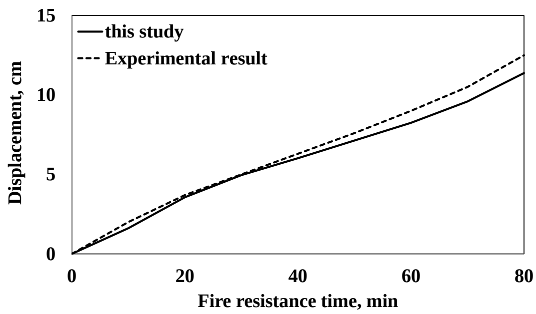

Fig 6. Predicted and measured deflections at center section

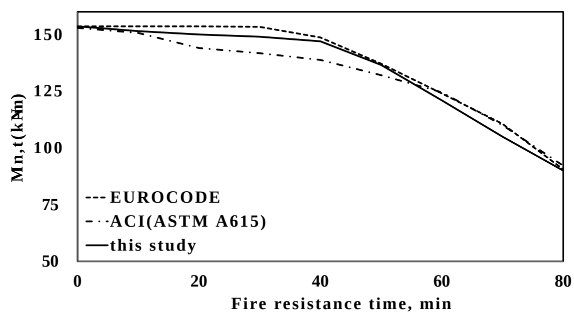

Fig 7. Ultimate flexure capacity at center section

Ultimate strengths of Columns under fire conditions

In Eurocode2, minimum dimensions and axis distances (b_{min}/a) for RC columns are given according to fire resistance time(R), mechanical reinforcement ratio(ω), eccentricity(e), slenderness(λ) and load level $\mathrm{n} = \frac{N_{0Ed,fi}}{0.7(A_c \cdot f_{cd} + A_s \cdot f_{yd})}$ where $N_{0Ed,fi}$ is axial load under fire condition, A_c is area of concrete, A_s is area of steel and, f_{cd} and f_{yd} are strength values of concrete and steel under normal temperature respectively. From given sections in Eurocode2, axial load (P) and first order moment (M) to be compared with numerical results can be determined. The comparison reveals that the Eurocode2 is more conservative than numerical results for short-columns (see Fig 8). On the other hand, some sections given in the Eurocode2 are shown to be unsafe as eccentricity or load level increases for more slender columns (see Fig 9).

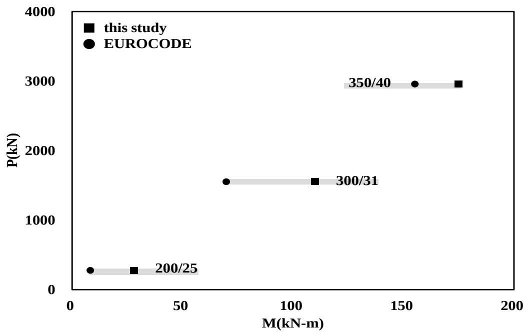

Fig 8. Short Column (λ=10), R60

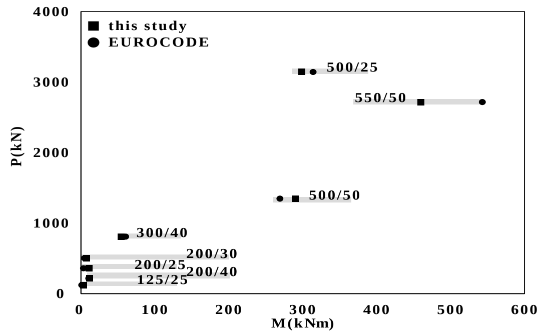

Fig 9. Slenderness Column (λ=30), ω=0.1, R60

CONCLUSIONS

This paper suggests that a numerical method to evaluate ultimate resisting capacity of RC members damaged under fire, and the introduced numerical approach is composed of two analysis procedures of the transient heat transfer analysis and nonlinear structural analysis. The proposed numerical approach was verified by comparing its analysis results with the data from fire test on beam. And the comparison shows that ultimate strength of RC members at elevated temperature by the proposed approach is properly calculated as compared with values determined from design codes. Furthermore, it is shown that, in case of short-column, sections given in Eurocode 2 are conservative and , on the other hand, direct usage of Eurocode 2 can be unsafe for slender columns as eccentricity or load level increases.

ACKNOWLEDGEMENT

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