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Prediction of post fire load deflection response of RC flexural members using simplistic numerical approach

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Abstract. A simplistic approach towards evaluation of complete load deflection response of Reinforced Concrete (RC) flexural members under post fire (residual) scenario is presented in this paper. The cross-section of the RC flexural member is divided into a number of sectors. Thermal analysis is performed to determine the temperature distribution across the section, for given fire duration. Temperature-dependent stress-strain curves for concrete and steel are then utilized to perform a moment-curvature analysis. The moment-curvature relationships are obtained for beams exposed to different fire durations. These are then utilized to obtain the load-deflection plots following pushover analysis. Moreover one of the important issues of modeling the initial stiffness giving due consideration to stiffness degradation due to material degradation and thermal cracking has also been addressed in a rational manner. The approach is straightforward and can be easily programmed in spreadsheets. The presented approach has been validated against the experiments, available in literature, on RC beam subjected to different fire durations viz. 1hr, 1.5hrs and 2hrs. Complete load-deflection curves have been obtained and compared with experimentally reported counterparts. The results also show a good match with the results obtained using more complicated approaches such as those involving Finite element (FE) modeling and conducting a transient thermal stress analysis. Further evaluation of the beams during fire (at elevated temperatures) was performed and a comparison of the mechanical behavior of RC beams under post fire and during fire scenarios is made. Detailed formulations, assumptions and step by step approach are reported in the paper. Due to the simplicity and ease of implementation, this approach can be used for evaluation of global performance of fire affected structures.

Keywords: fire; residual capacity; load deflection; simplistic numerical approach

1. Introduction

Concrete in general is known to have good fire resistance due to its high thermal capacity and low thermal conductivity. However when a reinforced concrete (RC) structure is exposed to fire, it loses its strength and stiffness as a result of increasing temperatures within the member. The extent of strength loss depends on the following conditions (Kodur *et al*. 2010):

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Fig. 1 Al Tayer residential tower, Sharjah (Source: www.constructionweekonline.com)

Fig. 2 Mantralaya building, Mumbai (Source: www.thehindubusinessline.com)

- Type of exposure (duration, no. of faces and percentage of exposure)
- Type of concrete (strength, aggregate type etc)
- Loading (level of initial load before and during exposure to fire).

Fire is one of the severe conditions to which a structure may be exposed to during its design life. Exposure of RC structures to a severe fire generally leaves evidence of significant structural damage such as noticeable spalling, exposed reinforcement and relatively large deformations whereas exposure to mild fire causes minor spalling and unnoticeable deflections. The major reasons for the cause of fire include human error (Al Tayer Residential Tower, Sharjah-April 2012, Fig. 1), electrical malfunction (Mantralaya Building, Mumbai-June 2012, Fig. 2), terror attacks (Hotel Taj, Mumbai, Nov 2008), earthquake (San Francisco, 1906). Recent fires in RC structures as shown in Figs. 1 and 2 have brought out the need to assess the fire affected structures for their post fire structural behavior. This is required to qualify the structure functionally and structurally as well as to estimate the nature and extent of retrofitting required.

The thermal properties of concrete and the dimensions of concrete structural elements lead to the development of high temperature gradients. The principal effects of fire on concrete are: loss of compressive and tensile strength, reduction in modulus of elasticity, cracking and spalling (main concern in high strength concrete). Effect of elevated temperature on concrete strength is

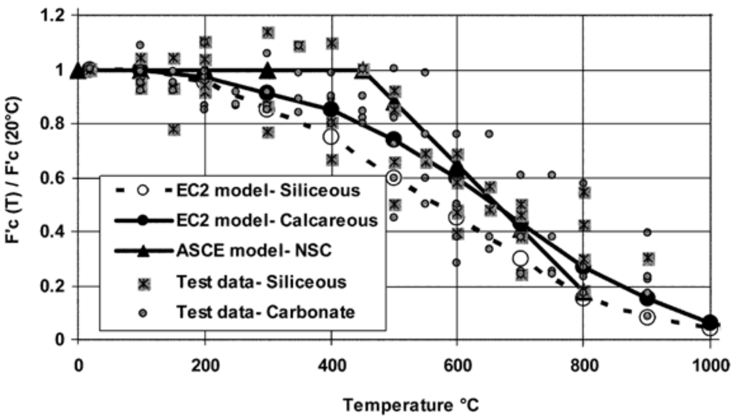

Fig. 3 Variation of compressive strength with temperature for normal strength concrete (Hot condition) (Kodur *et al.* 2008)

studied by conducting different types of tests namely, strength test at elevated temperature and residual state in stressed and unstressed conditions. In general these tests are referred to as testing under 'hot' and 'cold' conditions (Freskakis 1984). In 'hot' condition testing, the specimens are heated gradually to a target temperature, allowed to attain thermal steady state and then tested at that temperature. In 'cold' condition testing, the procedure up to attaining thermal steady state is essentially same. However, the specimens are allowed to cool to ambient temperature prior to testing. Test results of 'hot' condition tests act as input while evaluating the performance of RC structure during fire; whereas the results of 'cold' condition tests are appropriate for assessing the post fire behavior of RC structures.

Being a heterogeneous material, there is an inherent scatter in the properties of concrete. Various studies were carried out during the last few decades to find variation of various mechanical properties of concrete with temperature (Harmathy 1966, Anderberg *et al.* 1976, Freskakis 1979, Malhotra 1982, Schneider 1988, Takeuchi *et al.* 1993, Terro *et al.* 1997, Poon *et al.* 2001, Chang *et al.* 2006, Lee *et al.* 2008, Chen *et al.* 2009, Knaack *et al.* 2010). Based on these studies, factors affecting the mechanical properties were identified, which primarily include mix proportion, porosity, moisture content, rate of heating, mineral composition and structure of the aggregate, type of cement, sustained load during the load test and curing conditions. Fig. 3 displays strength degradation with temperature, for normal strength concrete made with different type of aggregate under hot condition. In addition to above-listed parameters, the residual strength (cold condition) is affected by rate of cooling, type of cooling (air cooled/water quenched) and strength recovery duration after cooling. Tests by Poon *et al.* (2001) suggested that with a post-fire duration of 56 days along with air/water re-curing, the concrete may regain certain amount of strength. Scatter in the residual strength for normal and high strength concrete as reported by different researchers is presented in Fig. 4.

The flexural capacity of RC beams, in general, is significantly influenced by the strength of reinforcing bars as compared to that of concrete. There is limited information available about the variation of mechanical properties of reinforcing bars (hot and cold condition) with temperature (Takeuchi *et al.* 1993, Neves *et al.* 1996). Figs. 5 and 6 show the variation of yield and ultimate strength, Young's modulus and elongation of reinforcing bars with temperature. It is observed that when reinforcing bars exposed to elevated temperature are cooled, they regain most of their

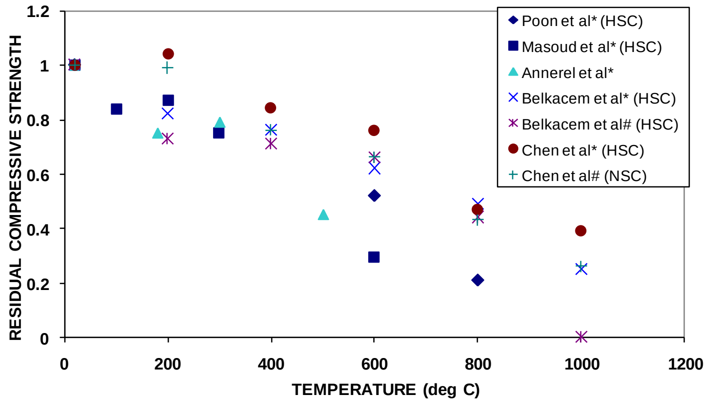

Fig. 4 Variation of residual compressive strength with temperature for concrete (Residual) (*- Air cooled, #- Water cooled)

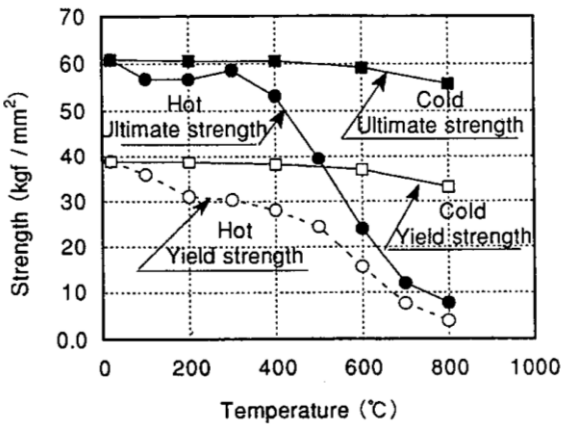

Fig. 5 Variation of Yield and ultimate strength of reinforcing bars as a function of temperature (Takeuchi *et al*. 1993)

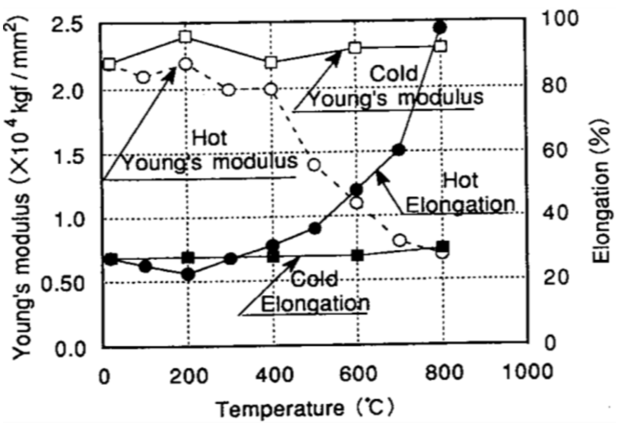

Fig. 6 Variation of Young’s modulus and elongation of reinforcing bars with temperature (Takeuchi *et al*. 1993)

ambient condition properties. Hence, the same flexural member is expected to show drastically different behavior in ‘during fire’ and ‘post fire’ scenarios.

This paper presents a simplistic approach to evaluate the structural performance of RC flexural members exposed to fire. The approach can be used to evaluate the complete load deflection characteristics for the members in during and post fire scenarios. This approach finds direct implementation in structural design offices or retrofitting consultancies as a handy tool for this purpose.

Over the past decade, efforts have been made to develop a rational design methodology/ procedure for evaluating the residual capacity of RC members after exposure to fire (Hus and Lin 2006, Fib Bulletin 46 2008, Kodur *et al*. 2010). However, most of the proposed approaches gave empirical relations that dealt only with the strength/capacity criterion. The approach presented in this paper is capable of estimating not only the load carrying capacity but also the complete load-deflection characteristics of the member. The basic methodology revolves around sequential thermal-stress analysis. The cross-section of the RC flexural member is divided into a number of sectors. Thermal analysis is performed to determine the temperature distribution across the section, for a given fire duration. The mechanical properties of concrete and steel in each sector are modified in accordance with the maximum temperature attained. The moment-curvature relationships are obtained for beams exposed to different fire durations. These moment-curvature characteristics were then utilized to define zero length springs that were used to obtain the load-deflection plots following nonlinear static pushover analysis. One of the major issues in defining the spring characteristics was the definition of initial stiffness for the fire affected beams. In this work a rational approach, accounting for stiffness degradation due to material degradation and thermal cracking has been proposed. The approach is straightforward and can be easily programmed in spreadsheets and can be used in conjunction with commercial software. In this work, the numerical analyses of RC beams tested by Kumar and Kumar (2003), subjected to different fire durations and tested under cold condition, is performed. Complete load-deflection curves have been obtained and compared with experimentally reported counterparts. It has been demonstrated that the results obtained using this simplified approach are in very good agreement with the experimental data. The results also show a good match with the results obtained using more complicated approaches such as those involving Finite element (FE) modeling and conducting a transient thermal stress analysis (Ozbolt *et al*. 2013).

To further evaluate the structural behavior of the beams in hot condition, analyses have been carried out for the same. It has been demonstrated that the load-deflection curves for the RC beams under hot condition are significantly different from the ones tested in cold condition. Due to the simplicity and ease of implementation, this approach can be used for evaluation of global performance of fire affected structures.

2. Proposed approach

The proposed approach for assessing the load-deflection characteristics of flexural member in post fire scenario broadly consists of following steps:

1. Thermal Analysis: Estimation of the maximum temperature profile within the cross-section of RC flexural member for a given fire exposure scenario.

2. Moment-Curvature Analysis: Evaluation of the moment-curvature relationships for the cross-section with temperature profile determined in step 1, using appropriate constitutive relations

for concrete and reinforcing bars.

3. Pushover Analysis: Performing nonlinear static pushover analysis to obtain the complete load deflection curve.

The above stated steps are elaborated in detail below.

The cross section of an RC beam is divided into $m \times n$ segments along its width and depth for analysis. Using lumped system concept, which is common approximation in transient conduction problems (Lienhard and Lienhard 2008), each segment is considered to have a uniform temperature and properties (both thermal and mechanical), lumped at the center of that segment.

2.1 Thermal analysis

To evaluate the behavior of RC flexural members for during or post fire scenario, it is required to know the temperature distribution within the cross-section of the member. Heat transfer from the surrounding hot gases to the surface of the member takes place by means of convection and radiation, while the heat transfer within the section takes place due to conduction. This complete process of heat transfer is transient in nature, since the surrounding gas temperature varies with time. The governing differential equation for two dimensional transient heat conduction problem is given as Eq. (1) (Lienhard and Lienhard 2008), which is solved using Finite difference method to obtain the spatial and temporal distribution of temperature, $T(x, t)$.

$$\rho c \frac{\partial T}{\partial t} = k\left[\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2}\right] \tag{1}$$

The following equation states the boundary condition that needs to be satisfied.

$$-k\frac{\partial T}{\partial n} = h[T_g - T_S] + \varepsilon\sigma[(T_g + 273)^4 - (T_S + 273)^4] \tag{2}$$

Where:

k is the thermal conductivity (W/m °C),
ρ is the mass density (kg/m^3),
c is the specific heat of solid (J/kg. °C),
h is convective heat transfer coefficient (W/m^2 °C),
ε is Stephen Boltzmann constant (5.667 x 10^{-8} W/m^2 °K^4),
σ is surface emissivity,
T_g is gas temperature (°C),
T_S is surface temperature (°C).

2.2 Moment-curvature analysis

The load deflection characteristics of flexural members are mainly dependent on the moment curvature characteristics of the section (Park and Paulay 1975). Moment curvature relationships for RC sections are derived based on assumptions similar to those of theory of bending using strain compatibility and equilibrium of forces within the section. For determining the moment curvature relation, stress strain relationships for concrete and reinforcing bars are required. In the present case, these stress strain relations are required as functions of temperature.

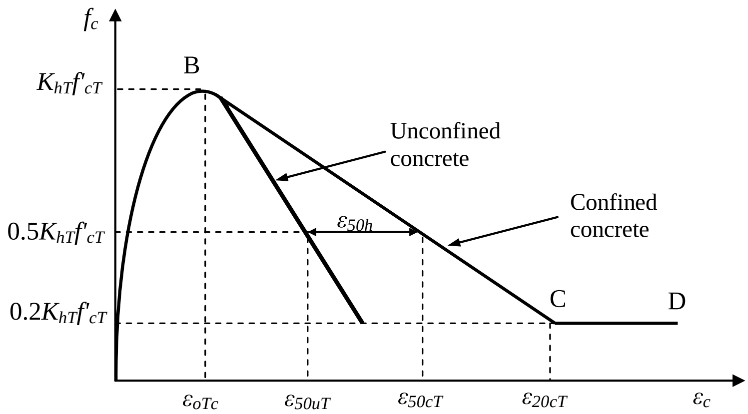

Fig. 7 Stress strain curve for concrete in compression (Youssef and Moftah 2007)

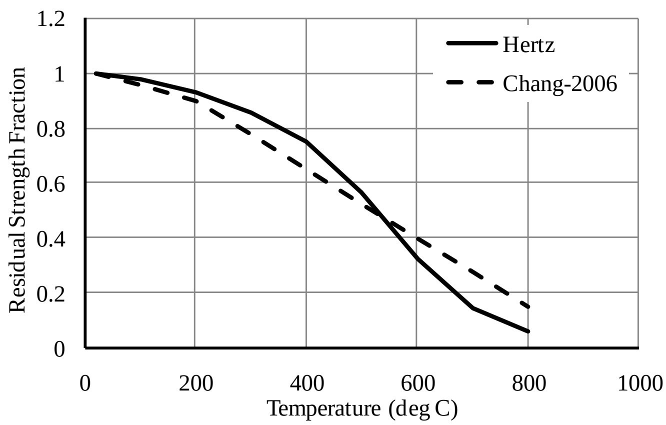

Fig. 8 Comparison of Hertz (2005) model with cold condition experimental results of Chang *et al*. (2006)

Various researchers have suggested constitutive laws of varying accuracy and simplicity for unconfined as well as confined concrete at ambient temperature. Out of these, the stress-strain curve for concrete confined by rectangular stirrups as suggested by Kent and Park (1971) has been shown to provide results in excellent correspondence with experiments (Sharma *et al*. 2011). Furthermore, the model offers a good balance between simplicity of application and accuracy of results. For concrete at elevated temperatures, the strength reduction as a function of temperature for unconfined concrete has been reported by various researchers. Relatively recently, Youssef and Moftah (2007) extended the Kent and Park (1971) model to define the stress-strain curve for confined concrete at elevated temperature (Fig. 7). The basic shape of the stress-strain curve as given by Kent and Park (1971) is maintained and the peak stress as well as the slope of the post peak stress-strain curve is modified by specifying stresses and strains as a function of temperature. In this work, the same model has been employed as the constitutive law for confined concrete in compression.

Youssef and Moftah (2007) used Hertz (2005) model to account for the strength degradation of concrete with temperature, which was proposed for concrete at elevated temperature i.e., hot condition. Chang *et al*. (2006) proposed the strength degradation of concrete with temperature in residual state (cold condition). In Fig. 8, the normalized strength degradation factors for hot and cold condition as proposed by Hertz (2005) and Chang *et al*. (2006) respectively are plotted. It is concluded from Fig. 8, that the Hertz (2005) model gives strength degradation values very close to the cold condition test results reported by Chang *et al*. (2006). In general, concrete strength has little influence on capacity of RC flexural members, since by design the reinforcement shall yield before concrete reaches its maximum strain in compression. Therefore, in this work, Hertz (2005) relation for strength degradation was assumed for cold condition also.

While performing the moment-curvature analysis, the tensile strength of concrete was ignored since the concrete has negligible tensile strength at ambient temperature, which is further degraded due to increase in temperature. The idealized stress-strain plot for reinforcing bars used is depicted in Fig. 9. The plot is a multi-linear approximation of the stress-strain curve for reinforcing bars under tension or compression, which includes hardening portion beyond yield. The complete procedure for calculating Moment-Curvature relationship is explained with a flow chart shown in Fig. 10. For further details of the theory, Park and Paulay (1975) may be referred.

The symbols used in Fig. 10 are described below:

$A_{R,k}$	Area of k^{th} reinforcement bar (mm^2)
A_{sec}	Area of sector (mm^2)
B	Width of the beam (mm)
D	Total depth of the beam (mm)
d'	Clear cover (mm)
$d_{R,k}$	Diameter of k^{th} reinforcement bar (mm)
$F_{comp,c}$	Total compressive force in concrete (N)
$F_{comp,R}$	Total compressive force in reinforcement bars (N)
$F_{Ten,R}$	Total tensile force in reinforcement bars (N)
i, j	Integers for identification of sector (Nos.)
k	Integer for identification of reinforcement bar (Nos.)
m	Number of sectors in X-Direction (along width) (Nos.)
M	Moment of resistance of the section at curvature (N-mm)
n	Number of sectors in Y-Direction (along depth) (Nos.)
N_R	Total number of longitudinal reinforcement bars in beam (Nos.)
$T_{i,j}$	Temperature of sector (*i,j*) (deg C)
tol.	Tolerance (convergence criterion)
$T_{R,k}$	Temperature of k^{th} reinforcement bar (deg C)
$X_{R,k}$	Horizontal distance to the centre of k^{th} rebar for bottom left corner of section (mm)
X_u	Depth of neutral axis from extreme compression fiber of the section (mm)
$Y_{R,k}$	Vertical distance to the centre of k^{th} rebar for bottom left corner of section (mm)
$\varepsilon_{c,ij}$	Strain in concrete of sector (*i,j*) (Unitless)
$\varepsilon_{c,ult}$	Ultimate strain in concrete (from constitutive law) (Unitless)
$\varepsilon_{R,k}$	Strain in k^{th} reinforcement bar (Unitless)
$\varepsilon_{R,ult}$	Ultimate strain in reinforcement (from constitutive law) (Unitless)
ε_u	Strain at extreme compression fiber of the section (Unitless)
ϕ	Curvature of beam (rad/mm)
$\sigma_{c\,T,ij}$	Stress in concrete of sector (*i,j*) at temperature T (MPa)
$\sigma_{PT,\kappa}$	Stress in k^{th} reinforcement bar at temperature T (MPa)

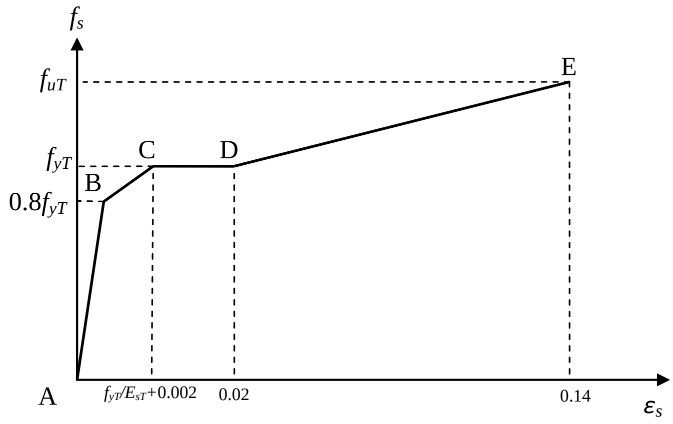

Fig. 9 Idealized stress-strain curve of reinforcing bars in tension or compression

2.3 Pushover analysis

The modelling was performed within the framework of stiffness matrix analysis and lumped plasticity approach using commercial software SAP2000® . The beam was modelled using 3D beam element with six degrees of freedom at each node. The hinge characteristics, once obtained, were assigned to the beam model at critical locations. The hinge is basically a zero length rotational spring with non linear characteristics defined by the moment curvature relationship computed as per section 2.2. One of the major issues in defining the spring characteristics was the definition of initial modulus of elasticity for the fire affected beams. The initial uncracked modulus of elasticity, E_c, for the reference beam (ambient conditions) was considered as $4730(f'_c)^{0.5}$ (ACI 318 2011). In case of reference beam, the first cracking was modelled by evaluating the cracking moment given as modulus of rupture multiplied by section modulus of the gross section.

The stiffness of the fire affected RC beams is much lesser compared to that of the RC beams tested at ambient temperature. The reason for that is two-fold: (i) degradation in modulus of elasticity of materials (concrete and steel) with rise in temperature and (ii) when the beams are subjected to fire loads prior to mechanical loading, they already have pre-cracks due to thermal gradients and restraints and hence have a lower modulus of elasticity compared to that of uncracked beams. Therefore, for the fire affected beams, cracked modulus of elasticity shall be used while performing pushover analysis.

Thus, the modulus of elasticity to be used in the analysis of a fire affected beam is given by

$$E_{ct} = k_1 k_2 E_c \tag{3}$$

Where,

E_{ct} is the effective modulus of elasticity for sections exposed to fire for duration 't' hrs
t is the duration of fire exposure (heating phase only)
E_c is the modulus of elasticity for uncracked concrete given as $4730(f'_c)^{0.5}$ (ACI 318 2011)
k_1 is the factor to account for reduction in stiffness due to material degradation
k_2 is the factor to account for thermal cracking

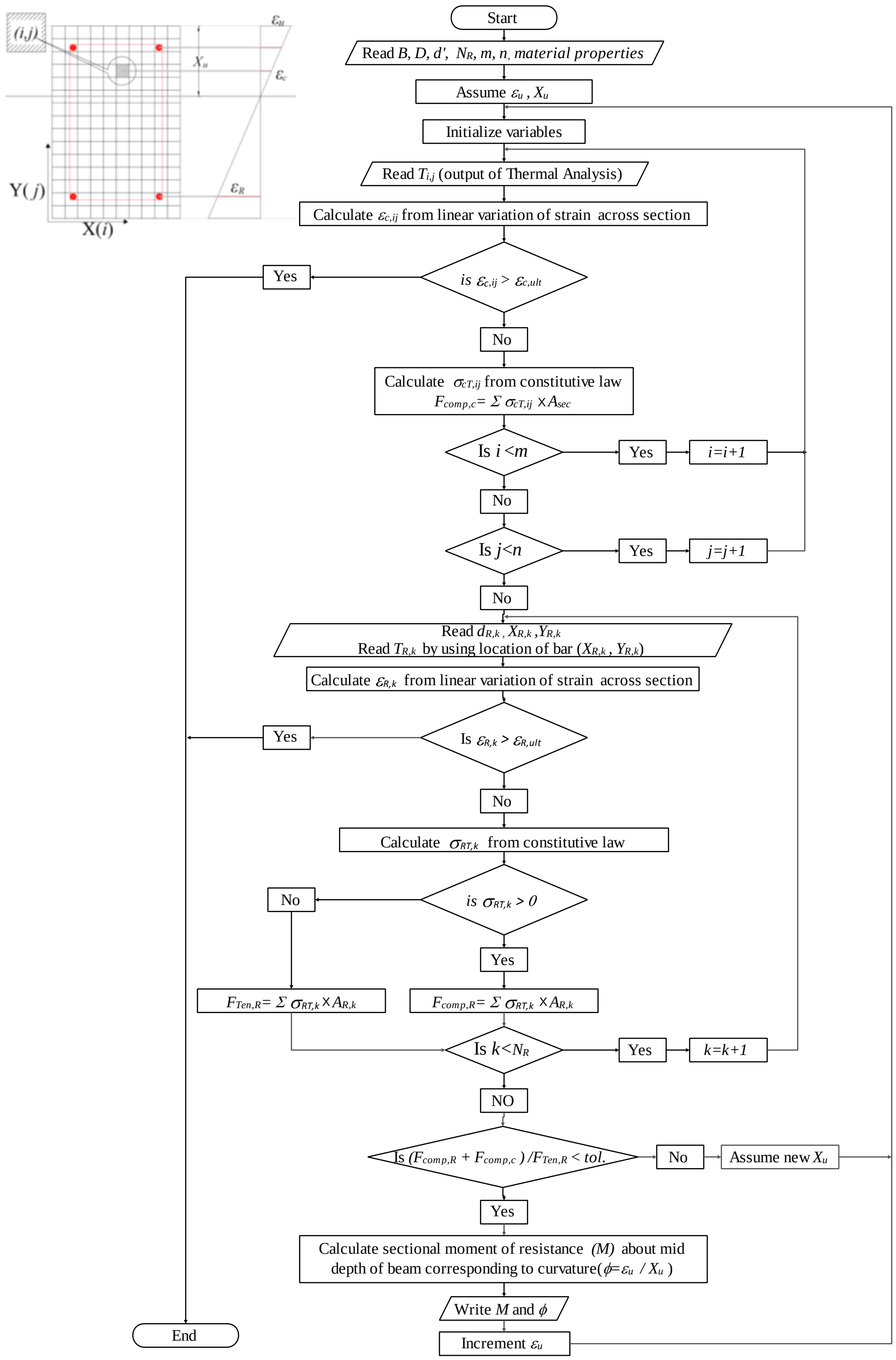

Fig. 10 Flowchart for moment-curvature analysis for the section

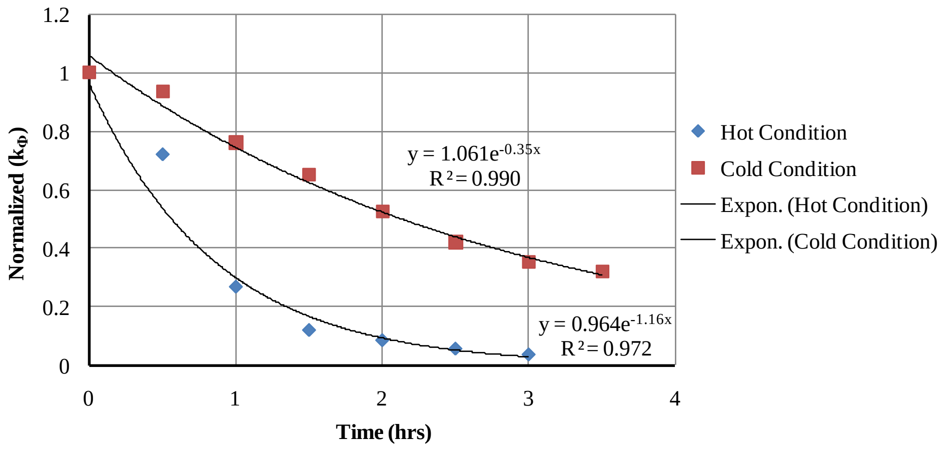

Fig. 11 Normalized stiffness degradation due to fire exposure (Hot and cold conditions)

The factor k_1 for reduction in stiffness of the fire affected beams due to material degradation was evaluated as per following steps:

1. The moment-curvature characteristics for the section at ambient temperature were evaluated (as per procedure explained in section 2.2 and Fig. 10).
2. The moment-curvature characteristics of fire affected sections for various durations of exposure were evaluated (as per procedure explained in section 2.2 and Fig. 10).
3. The slopes of the initial (linear) portion of moment-curvature relationships for all sections moment-curvature relationships obtained in step 1 and 2 were evaluated. The slope is computed as yield moment divided by yield curvature ($k_\Phi = M_y/\Phi_y$).
4. The computed k_Φ for fire affected beams were normalized with respect to that of ambient temperature. Fig. 11 shows the plot of normalized k_Φ values for different exposure durations for hot and cold conditions. A best fit curve was fitted to the normalized data as displayed in Fig. 11.
5. The normalized k_Φ value obtained by the fitted equation was considered as k_1.

At ambient temperature, FEMA 440 (2005) recommends to use a value of $0.5E_c$ as cracked modulus of elasticity for flexural members (i.e., $k_2 = 0.5$). In case of thermal cracks, the cracking is more severe than that of the first cracking in a flexural member at ambient temperature. Moreover, the cracks are spread over the entire section. Considering the same, in this work, the factor k_2 for thermally damaged cracked section was considered as 0.4. Incidentally, this is the same value that is recommended by FEMA 440 (2005) for the cracked modulus of shear dominated sections.

3. Validation against experimental results

Various researchers have performed experiments to evaluate the residual load carrying capacity of fire affected beams (Moetaz *et al*. 1996, Kumar and Kumar 2003, Kodur *et al*. 2010, Jayasree *et al*. 2011). However, in this work, the emphasis is laid not only on the load carrying capacity but the complete load-deflection behavior of beams subjected to elevated temperatures. Therefore, for validating the approach presented in previous section, the experiments on simply supported RC beams performed by Kumar and Kumar (2003) were numerically simulated.

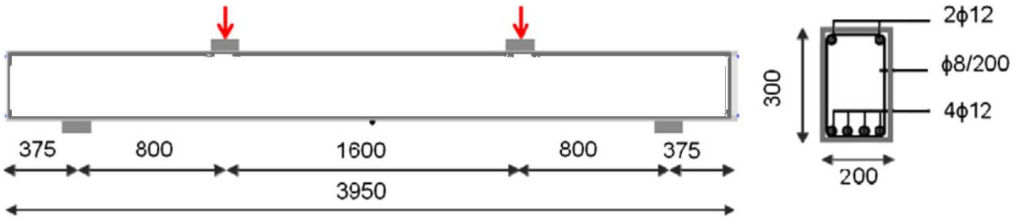

Fig. 12 Test setup for strength test on RC beams (Ozbolt *et al*. 2013)

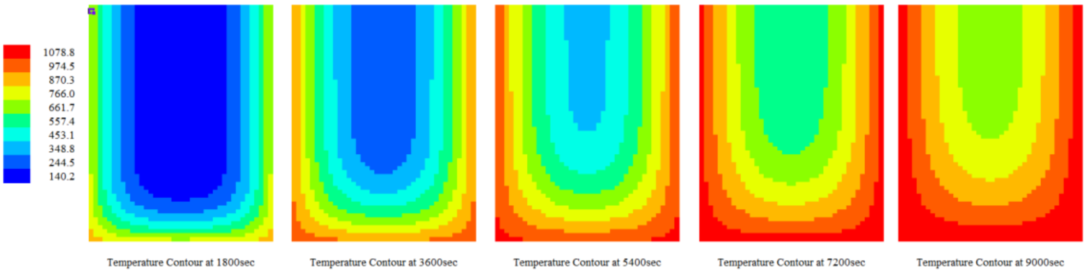

Fig. 13 Temperature contour across beam cross section at 0.5 h, 1.0 h, 1.5 h, 2.0 h and 2.5 h

3.1 Details of test by Kumar and Kumar (2003)

In absence of information on test setup used by Kumar and Kumar (2003), the test setup used by Ozbolt *et al*. (2013) was used as detailed in Fig. 12. The RC beams were 200×300 mm in cross-section. These beams were made of normal strength concrete made of crushed stone aggregate of 20 mm maximum size and grading zone II sand (as per IS) in proportion of 1:3.4:8 by weight. The average cube strength, f_{ck} was 20.89 MPa after 28 days. Clear cover to main reinforcing bars was 25 mm. High yield strength deformation bars of grade Fe-415, conforming IS 1786-1985 were used as reinforcing bars.

All the beams were exposed to standard temperature-time curve as per ISO 834 from three sides. During exposure no superimposed load was applied on the beams. Four beams were tested with exposure durations of 1 hr, 1.5 hr, 2 hr and 2.5 hrs respectively. One beam was tested without exposure to fire, this served as the control or reference specimen. The heated beams were allowed to cool naturally to ambient temperature and subsequently tested under four point bending (ref. Fig. 12) in load control mode. The load-deflection curves were reported by Kumar and Kumar (2003) for the reference specimen and 1 hr, 1.5 hr and 2 hrs case. It was reported that the specimen heated up to 2.5 hrs showed excessive deflections during heating phase and hence was not tested for residual load-displacement test. A plot of normalized flexural strength against fire duration was given by the authors.

3.2 Thermal analysis

The thermal analysis of the beams was performed by numerically solving equations (1) and (2). A visual basic program using the macros feature within Microsoft® Excel was developed for the same. The analysis was performed for ISO834 fire exposure durations from 0.5 hrs to 2.5 hrs at an interval of 30 min. The case of 0.5 hrs was added to (i) obtain the data for stiffness degradation

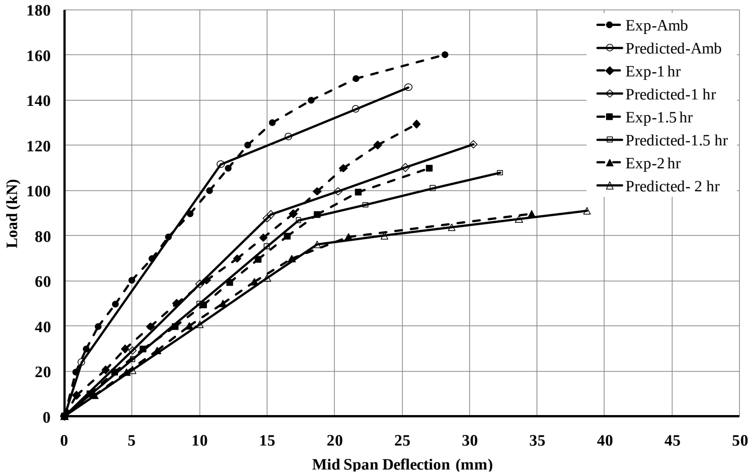

Fig. 14 Comparison of predicted load deflections with experimental values

i.e., to generate the plot shown in Fig. 11, and (ii) to perform the analysis of an additional case for hot condition. The temperature profiles within the section due to different fire exposure durations are shown in Fig. 13. It was noted that the thermal profiles computed using the program were in good agreement with those available in Eurocode2 (2004). These temperature profiles were utilized to derive the temperature affected properties of each sector, which were thereby utilized to perform stress analysis.

3.3 Stress analysis

The results of the thermal analysis were used to evaluate the moment curvature relationships for the sections following the procedure explained in section 2.2. Additionally, to evaluate the pre-yield stiffness, the effective modulus of elasticity was calculated based on the procedure given in section 2.3. For example, for the case of the beam exposed to 2 hrs fire (cold condition), the effective modulus of elasticity, E_{ct} was calculated using Eq. (3) and chart shown in Fig. 11 as:

$$E_{ct} = (1.06e^{-0.352\times 2}) \times (0.4) \times (4730\sqrt{0.8 \times 20.89}) = 4055\text{MPa}$$

for t = 2 hrs and f_{ck} = 20.89MPa (assuming $f_c' = 0.8f_{ck}$)

The computed moment-curvature relationship along with the effective young's modulus for fire affected section was used to compute the load-deflection characteristics of the member by performing pushover analysis. Fig. 14 compares the predicted total load v/s mid span deflections with the experimental values reported by Kumar and Kumar (2003). It is noted that the complete load-deflection curve including the initial stiffness, yield and peak loads as well as corresponding displacements are in good agreement with the experimental values.

The experimental and numerical values of peak loads for each case are compared in Fig. 15. It is observed that the numerical predictions are in good agreement with the experimental values and also on the conservative side i.e., slightly lower than the experimental values. It is noted that the difference between the predicted peak load and experimental peak load decreases with higher exposure duration. Two points observed close to equality line are those corresponding to 1.5 and 2 hr exposure duration. This is attributed to the fact that during experiment, reduction is cross-

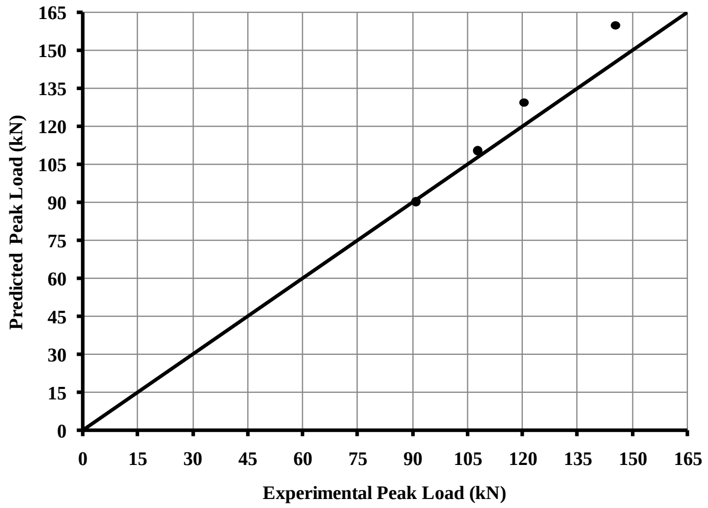

Fig. 15 Experimental v/s predicted Peak load comparison

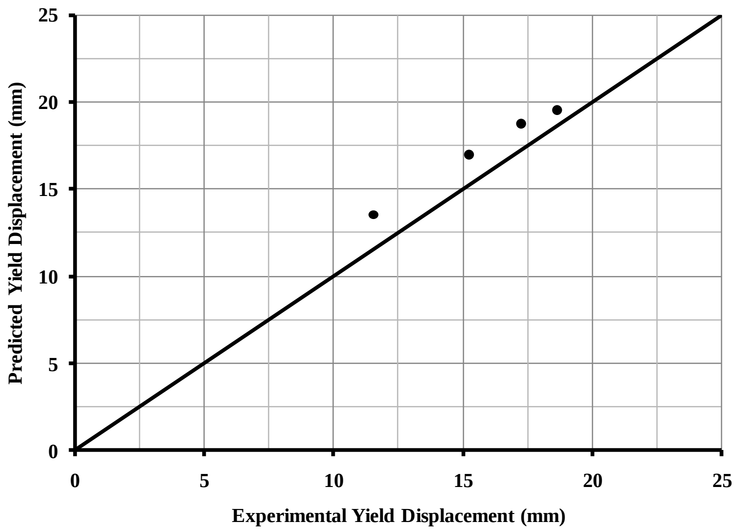

Fig. 16 Experimental v/s predicted yield deflections

section due to spalling was observed, which has not been accounted for explicitly in this formulation. The method assumes that no severe spalling and consequent reduction in cross section has occurred.

The predicted displacements at yield are consistently on the conservative side with a reasonably good match with the experimental observations, as displayed in Fig. 16. Same is the case with the predicted displacements at peak loads. The comparison between predicted and experimental displacement at peak loads is shown in Fig. 17. The difference in experimentally reported and numerically predicted values may be attributed to the fact that the experiment was performed under load controlled conditions, which is considered to not provide accurate displacement values in near and post-peak region. It is noted that the simplistic numerical approach presented is well

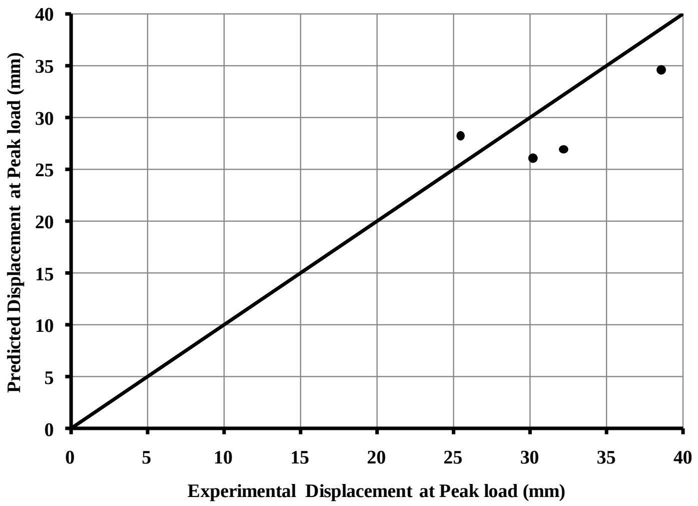

Fig. 17 Experimental v/s predicted deflections at peak load

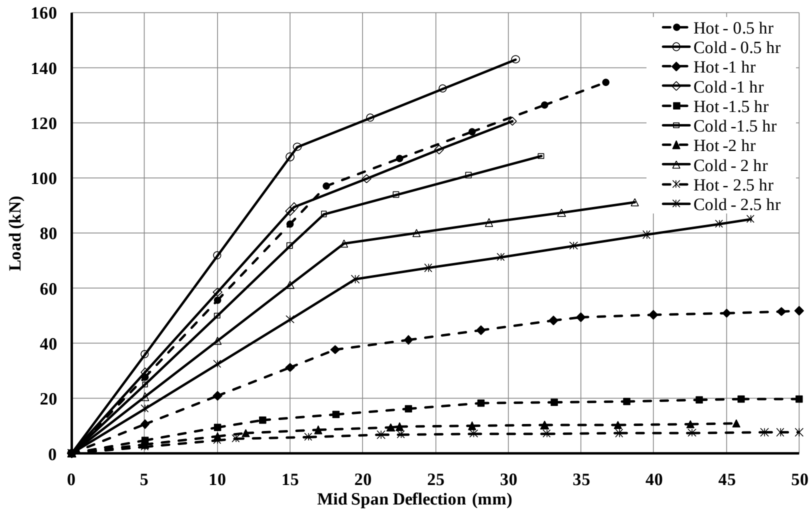

Fig. 18 Comparison of load deflection in 'Hot' and 'Cold' conditions

capable of predicting the complete load-deflection curve including the initial stiffness, yield and peak loads as well as corresponding displacements with reasonable accuracy.

4. Further evaluation: RC beams at high temperature (Hot condition)

Once the numerical approach explained above, was established against the experimental results on RC flexural members for post fire scenario, the same was used to obtain the load deflection characteristics of the same under "during fire" (hot condition) scenario. For extending this simplistic numerical approach to during fire scenario, the basic procedure remains the same except the material properties to be used in this case would correspond to those obtained by testing concrete and steel in hot condition. The properties of concrete at elevated temperature were the same as explained above and those for reinforcing steel were from Eurocode2 (2004).

Fig. 18 presents the results obtained in the form of load-deflection plots for the beams in hot condition. To appreciate the difference in behavior, the results are superimposed with those obtained for cold condition. It can be observed that the behavior of the same RC beam in hot condition is drastically different from that in cold condition (Fig. 18). For flexural members, this difference in behavior is primarily attributed to the difference in properties of reinforcing steel in hot and cold condition. It is observed that in hot condition the load carrying capacity of RC beam dropped drastically with exposure duration of 1 hr onwards. For the exposure duration of 2.5 hrs, the load carrying capacity of the beam drops down to an extent that it can no longer support even its self weight. This observation is also in correspondence with the observations reported by Kumar and Kumar (2003), where it is reported for this case the beams showed large deflections during heating phase and could not be tested to obtain residual load-deflection curves.

5. Conclusions

Recent accidents world over have emphasized the importance of structural assessment of fire affected reinforced concrete structures. Currently, there is a lack of simplistic approaches for evaluating the behavior of fire affected RC members and a few available lack scientific basis. This paper attempted to present one such rational yet simplistic approach utilizing well established principles of mechanics and heat transfer to evaluate the complete structural performance in form of load deflection characteristics for RC flexural members exposed to fire. The approach can be used for evaluation in during and post fire scenarios.

The basic methodology consists of performing a sequential thermal-stress analysis for the RC section. Using lumped system concept, the cross-section of the RC flexural member is divided into a number of sectors and thermal analysis is performed to determine the temperature distribution across the section, for a given fire duration. The governing differential equation for two dimensional transient heat conduction problem is solved using Finite difference method to obtain the spatial and temporal distribution of temperature.

The mechanical properties of concrete and steel in each sector are modified in accordance with the maximum temperature attained. Using these material properties the moment-curvature relationships are obtained for beams exposed to different fire durations following basic principles of mechanics. These moment-curvature characteristics were then utilized to define zero length springs that were used to obtain the load-deflection plots following nonlinear static pushover analysis. One of the major issues in defining the spring characteristics was the definition of initial stiffness for the fire affected beams. In this work a rational approach, accounting for stiffness degradation due to material degradation and thermal cracking has been proposed.

The presented approach has been validated against the experiments, available in literature performed by Kumar and Kumar (2003), on RC beam subjected to different fire durations. Complete load-deflection curves have been obtained and compared with experimentally reported counterparts. It was observed that for all the cases, the presented approach predicted the complete load-deflection curve including the initial stiffness, yield and peak loads as well as corresponding displacements with reasonable accuracy.

The simplistic numerical approach presented in this paper can be used to estimate not only the residual capacity of RC flexural members but can also predict the complete load displacement characteristics for the member, which is an important parameter for performance based design philosophy. The same approach has been demonstrated to evaluate load deflection characteristics for RC members in hot condition as well. The approach can be effectively used to study the behavior of fire affected structures at element level, sub assembly level and also at global level.

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