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Seismic Resistance of Fire-Damaged Reinforced Concrete Columns

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ABSTRACT

A study was carried out on the assessment of the residual strength and lateral/seismic load capacity of reinforced concrete columns after fire exposure. The mechanical properties of concrete after fire exposure, such as residual tension and compression strength, stiffness and constitutive responses, were reviewed. A nonlinear finite element analysis program, developed for three-dimensional reinforced concrete structures, was employed to estimate the post-fire axial and lateral performance of two full-scale reinforced concrete columns, tested previously at the National Research Council Canada. The results of the axial load-deformation response obtained from the analysis were compared and verified successfully with the test data. The lateral load and deformation responses of the specimens were then estimated using the numerical program. The results showed that both the lateral/seismic load capacity and ductility of the two reinforced concrete column specimens decreased noticeably due to the fire exposure. Studies are suggested on fire damage mitigation and post-fire shear strength recovery of reinforced concrete columns.

INTRODUCTION

Statistics report and studies show that the average annual fire occurrence in moderate and high-rise buildings exceeds 10,000 incidents only in the United States (Hall, 2001). In most of these fire incidents, building structures have experienced minor to major damage due to fire exposure, such as degradation of material properties due to elevated temperatures and damage to structural elements due to thermal expansion. After any building fire incident, an inspection is required for the assessment of the structural loss and damage and evaluation of the building residual capacity. As the result of such a survey, the decision is made to either repair or demolish and rebuild the structure (CIB W14, 1990). Therefore, application of a reliable approach for damage evaluation plays an important role in the safety and subsequent cost of the repair or reconstruction of the fire-exposed structures. This would include assessment of both the short-term and long-term response of the structures after a fire.

At elevated temperatures, the compressive strength and stiffness properties of concrete decline substantially (CIB, 1990; Schneider, 1988; Concrete Industry, 2008; Harmathy, 1986; Lie et al., 1986). Maximum temperature during the fire exposure and cooling method are among the most important parameters responsible for the degree of damage to concrete (Lee et al., 2008). The faster the cooling rate, the higher temperature gradient and therefore the higher damage induced to the concrete. Even with natural cooling, the interior temperature gradient within the concrete can be higher than that during the heating process (Lee et al., 2008). Due to large creep strains, higher degradations have been observed in the elastic stiffness than in the strength of concrete after the fire exposure. A study on post-fire response of reinforced concrete columns under axial load combined with uniaxial or biaxial bendings, concludes that this higher degradation in stiffness will result in large deformations and that should be considered for seismic response evaluation of a building subjected to earthquake after a fire (Chen et al., 2009).

The tensile strength of concrete also reduces at elevated temperatures (EI-Hawary et al., 1997; Chang et al., 2006; Nechnech et al., 2002). Since the shear strength of concrete is dependent on its tensile strength, the loss of tensile strength capacity could increase the risk of shear failure for concrete elements in fire and therefore decrease their lateral/seismic load capacity. Studies on flexure and shear behavior of six concrete beams during fire showed that although shear cracks on the beams appeared relatively early in the test, shear strength at elevated temperatures was not significantly affected (Ellingwood and Lin, 1991). One of the reasons for the flexural response governing in these tests could be due to the reduction of the beams' flexural capacity in fire, which resulted from a reduction in the yield stress of the reinforcing bars at elevated temperature. After cooling, steel recovers its original strength and stiffness (CIB, 1990) and therefore the beams recover most of their flexural capacity. However, the residual tensile strength of concrete has been reduced due to fire exposure, resulting in a permanent reduction of the shear capacity. If a fire reduces the beam shear capacity to less than the flexure capacity, then the beam is at risk of shear failure or a brittle failure. Loss of shear strength would be more important for deep beams and seismic resistant elements such as shear walls, in which shear mechanisms play the principal role in the element response. Research on the lateral capacities of reinforced concrete columns reveals that axial, shear and flexure mechanisms all influence the response of a reinforced concrete column under axial and shear forces at ambient temperatures (Mostafaei and Kabeyasawa, 2007). A three dimensional numerical model should capture these three main response mechanisms for columns. Hence, in this study, VecTor3, a three-dimensional nonlinear finite element analysis program, developed at the University of Toronto, is employed to evaluate the lateral load capacity of two fire-damaged reinforced concrete column specimens.

MECHANICAL PROPERTIES OF FIRE-DAMAGED CONCRETE

Degradation in the mechanical properties of concrete during fire was investigated by several researchers (Abrams, 1971; Malhotra, 1956; Lie et al., 1992).

Concrete modulus of elasticity. Concrete experiences a rapid loss of stiffness at elevated temperature. At 200^{o}C, the modulus of elasticity of concrete reduces to 70-80% of that at ambient temperature and at 400^{o}C, it diminishes to 40-50% of its original value (Lie et al., 1992). The original modulus of elasticity of concrete can not be recovered upon cooling (Harada, 1961). The fire-damaged concrete stiffness can recover somewhat, depending on the temperature exposure and the curing time after the exposure. However, recovery is never complete (Lie et al., 1992). In numerical modeling, and in this study, the initial modulus of elasticity of concrete can be estimated according to the concrete strength and peak compressive strain, Eq. (1).

$$E_{oT} = \frac{f'_{cT}}{2\varepsilon_{oT}} \tag{1}$$

where, E_{oT} is the residual initial stiffness after exposure to temperature T, f'_{cT} is the residual compressive strength of concrete after exposure to temperature T, and ε_{oT} is residual peak compressive strain after exposure to temperature T. Alternative models for peak compressive strain of unstressed/stressed concrete at elevated temperatures have also been proposed (Anderberg and Thelandersson, 1976; Youssef and Moftah, 2007). Models are also available for post-fire residual peak strain of concrete. Chang et al. (2006) proposed Eq. (2) for the residual peak compressive strain of unstressed concrete after fire.

$$\frac{\varepsilon_{oT}}{\varepsilon_o} = \begin{cases} 1.0 & 20^o C < T \le 200^o C \\ (-0.1 f'_c + 7.7)\left[\dfrac{\exp(-5.8+0.01T)}{1+\exp(-5.8+0.01T)} - 0.0219\right] + 1.0 & 200^o C < T \le 800^o C \end{cases} \tag{2}$$

where, ε_o is the concrete compressive peak strain at ambient temperature (20^{o}C), f'_c is the concrete compressive strength at ambient temperature (20^{o}C), and T is the maximum temperature to which the concrete has been exposed before cooling. Eq. (2) was verified for two groups of concrete specimens with siliceous aggregate: the first group having with an original compressive strength of 40 MPa, similar to that of the columns investigated in this study, and the second group with an original compressive strength of 27 MPa, for a temperature up to 800^{o}C. Eurocode4 (2005) considers the same peak strain for concrete during heating and during the cooling down. However, a study on the residual mechanical properties of normal strength concrete with siliceous aggregate shows that Eurocode4 (2005) provides higher values for the residual peak strain than that of the test data by Chang et al. (2006). This indicates that higher peak strains are estimated for concrete at the maximum temperature T than that after having cooled down to 20^{o}C. For stressed concrete, which is commonly the condition of concrete in columns, temperatures appear to have less effect on the peak compressive strain than it does for the unstressed concrete (Khennane and Baker, 1992). Youssef and Moftah (2007) studied different models for peak strain of concrete at elevated temperatures and concluded that Khennane and Baker (1992), Eq. (3), provided good agreement with test results. Eq. (3) was derived based on the tests of normal strength concrete under three level of stress: 10%, 17-22.5% and 45% of the original concrete compressive strength. Although, Eq. (3) is proposed for peak strain of concrete at elevated temperature, this study shows that using the same equation, in the analysis of the two column specimens, results in good agreement between analysis and test data.

$$\varepsilon_{oT} = \varepsilon_o + 0.00000167 \tag{3}$$

where, ε_{oT} is residual peak compressive strain of stressed concrete at temperature T during and after exposure to temperature, and ε_o is concrete compressive peak strain at 20^{o}C. Suggested value for ε_o is 0.00267 (Youssef and Moftah, 2007).

Concrete compressive response. The compression strength of concrete at elevated temperatures is dependent mainly on temperature, type of the aggregate, cement to aggregate ratio, and level of the applied stress (Lie et al., 1992). Increases in temperature result in degradations of concrete compression strength. Loss of compressive strength of concrete with siliceous aggregate due to increased temperature is faster than that of the concrete with carbonate aggregate (Eurocode2, 2004). The aggregate-cement ratio has a significant effect on the compressive strength of concrete at elevated temperatures. The reduction is proportionally smaller for lean mixes than for rich mixes (Schneider, 1988). For the same elevated temperature, stressed concrete typically shows less loss of compressive strength than that of the unstressed concrete (Abrams, 1971; Malhotra, 1956). Typically, the concrete residual strength after cooling is less than that at the maximum exposed temperature (Malhotra, 1956). The cooling rate plays a highly important role in such strength losses and is identified to be responsible for the fire-induced damage to the concrete (Lee et al., 2008). Both the strength and stiffness of concrete decrease faster when the cooling rate is increased. This may be due to increased temperature gradients resulting in increased cracking of the concrete and therefore losses in its mechanical properties. Chang et al. (2006) proposed a temperature-dependant residual compressive strength model, Eq. (4), from compression tests of 108 specimens. This model has been chosen for this study.

$$f'_{cT}\Big/f'_c = \begin{cases} 1.01 - 0.00055T & 20^o C < T \le 200^o C \\ 1.15 - 0.00125T & 200^o C < T \le 800^o C \end{cases} \tag{4}$$

where, f'_{cT} is the residual compressive strength of concrete after exposure to temperature T and f'_c is concrete compressive strength at ambient temperature (20^{o}C), and T is the maximum temperature that concrete has been exposed to, before cooling. Studies show that the compressive strength ratios determined by Eq. (4) are close to the experimental results by Abrams (1971) and results of the equation provided by Eurocode4 (2005). However, relatively lower strength is determined by the Abrams' model compared to that of the Chang et al.'s model. The compressive strength determined by Eq. (4) is for unstressed concrete. Studies on residual strength of reinforced concrete columns reveals that the residual strength was not affected by the stress (Lie et al., 1986). Hence, in this study, the residual compressive strength is considered unchanged for unstressed and stressed concrete, using the same model, Eq. (4).

Several stress-strain models have been proposed for concrete in compression at elevated temperatures and at cooled temperatures such as Chang et al. (2006), Nechnech et al. (2002), Youssef and Moftah (2007), Eurocode2 (2004), Eurocode4 (2005), and Terro (1998), among others. The effects of concrete confinement on the peak and post-peak compressive response of concrete have been considered using available models which were modified for high temperature (Youssef and Moftah, 2007). In this study, the model by Popovics (1973) is used by VecTor3 as the residual compressive stress-strain relation of concrete given in Eq. (5).

$$f_c = -(\frac{\varepsilon_c}{\varepsilon_{oT}}) f'_{cT} \frac{n}{n-1+(\varepsilon_c / \varepsilon_{oT})^n} \quad where \quad n = \frac{E_{oT}}{E_{oT} - E_{\sec}} \tag{5}$$

where, f_c is the compressive stress, ε_{oT} is the residual peak compressive strain from Eq. (3), ε_c is the compressive strain, E_{cT} is the initial tangent stiffness, and E_{sec} is the secant stiffness. For reinforced concrete columns and beams with high ratios of transverse reinforcement ratio, f_c and ε_{oT} can be modified to include confinement effects. A modified Park-Kent model (Park et al., 1982) is employed by VecTor3 for post-peak and confinement effect of concrete in compression.

Concrete Tensile Response. Compared to the compressive mechanical properties, fewer studies have been carried out on tensile strength and stiffness of concrete at or after fire exposure. At elevated temperatures, Terro (1998) proposed a linear stress-strain relation for the pre- and post-peak response of concrete in tension. The tensile strength of concrete in this model reduces zero at the tensile stain of 0.004, which was selected based on the results from independent temperature studies. Reductions in the concrete tensile strength in fire are greater than those in the compressive strength, especially for temperatures less than 400°C (Chang et al. 2006). For example, the residual compression strength of normal strength concrete, after exposure to 200^{o}C, is about 90% of its original strength; however, the residual tensile strength of the concrete at the same temperature is about 80% of that at the ambient temperature (Chang et al., 2006). Several models are available for tensile mechanical properties of concrete such as strength, stiffness and the stress-strain relations (Chang et al., 2006; Nechnech et al., 2002; Youssef and Moftah, 2007; Eurocode4, 2005; Eurocode2, 2004; Papayianni and Valiasis, 1991; Felicetti and Gambarova, 1999; Terro, 1998). Eurocode2 (2004) provides a relatively simple model for tensile strength of concrete at different temperatures. Chang et al. (2006) compared the results of this equation with the results they obtained from tension splitting tests of 54 normal strength concrete specimens with siliceous aggregate and showed that the residual tensile strength determined by the Eurocode2 equation is relatively larger than that of the test data for temperatures lower than 200^{o}C and smaller for temperatures higher than 200^{o}C. Chang et al. thus proposed an alternative residual tensile strength model for concrete: Eq. (6). This model is employed for the calculation of the residual tensile strength of column concrete in this study.

$$f'_{tT} \big/ f'_t = \begin{cases} 1.05 - 0.0025T & 20^o C < T \le 100^o C \\ 0.8 & 100^o C < T \le 200^o C \\ 1.02 - 0.0011T \ge 0.0 & 200^o C < T \le 800^o C \end{cases} \tag{6}$$

where f'_{tT} is the residual tensile strength of concrete after exposure to temperature T, f'_t is the concrete tensile strength at 20^{o}C, and T is the maximum temperature to which concrete has been exposed before cooling.

A stress-strain relation for concrete in tension at ambient temperature, allowing for tension stiffening effects, was proposed by Bentz (2000). The same model, Eq. (7) is implemented in this study to estimate the residual stress-strain response of fire-damaged concrete in tension.

$$f_{tT} = \frac{f'_{tT}}{1+\sqrt{c_t \varepsilon_t}} \tag{7}$$

where, f_t is the tensile stress, f'_{tT} is the residual concrete tensile strength from Eq. (6), ε_t is the tensile strain, and c_t is a factor that incorporating the influence of reinforcement bond characteristics.

Residual Response of Reinforcing Bars. Although at elevated temperatures both the stiffness and strength of steel drop substantially (Lie et al., 1992), recovery of the yield strength after cooling is generally complete for temperatures up to 450^{o}C for cold work steel and 600^{o}C for hot rolled steel. Above these temperatures, a simple approach is suggested by CIB (1990) in which for every 100^{o}C increase in temperature 7.5% of the yield strength is reduced.

Deformation Properties. At elevated temperature, the total concrete strain ε is determined based on the following four components (Anderberg and Forsen, 1982).

$$\varepsilon = \varepsilon_{th} + \varepsilon_{\sigma} + \varepsilon_{cr} + \varepsilon_{tr} \tag{8}$$

where $\varepsilon_{th,}$ a function of temperature, is the thermal strain (Eurocode2 2004), including shrinkage, measured on specimens under variable temperature; ε_{σ},, a function of stress, stress history, and temperature, is the instantaneous stress-related strain, determined based on stress-strain relations obtained under constant, stabilized temperature; ε_{cr}, a function of stress, temperature and time, is the creep strain (Gross 1975) or time-dependent strain measured under constant, stabilized temperature; and ε_{tr}, a function of stress and temperature, is the transient strain (Anderberg and Thelandersson, 1976), which is the result of temperature increase under constant stress. Studies show that for a short period of heating, the value of the creep strain is relatively small and the transient strain is accounted for in the main portion (Terro 1998). After temperatures have cooled, the main strains for calculating the total deformation are the stress-related strain, and the creep strain. Test results show that the creep strain can be significantly large for stressed concrete after fire. For example, test results by Lie et al. (1986) show that for a reinforced concrete column specimen exposed to a two-hour standard fire, the creep strain recorded was about five times higher than that of a similar column with a one-hour fire exposure, one day after the fire.

MODEL VERIFICATION FOR RESIDUAL COMPRESSIVE RESPONSE

Test Specifications. Two 305 × 305 mm reinforced concrete columns (named Column A and Column B in this study) made with siliceous aggregate, previously tested at the National Research council Canada (Lie et al., 1986), were selected for the purpose of this study. The clear height of the columns was 3760 mm; however, only 3150 mm of the height was contained within the furnace. Each column had four longitudinal bars with 25 mm diameter and 13 ties with 10 mm diameter (at 305 mm spacing). The cover concrete to the longitudinal bars was 48 mm. The yield strength of the longitudinal bars was 444 MPa, and that of the ties was 427 MPa. The ultimate tensile strength was 730 MPa for the main bars, and 671 MPa for the ties. The concrete compressive strength, measured on the day of the fire test, was 38.9 MPa for Column A and 41.8 MPa for Column B. The moisture condition at the center of the Column A was about equivalent to that in equilibrium with air of 87% relative humidity (RH) at ambient temperature, and of Column B with air of 83% RH. The axial load applied on Column A was 992 kN and that on Column B was 1022 kN. Column A was exposed to a standard fire ASTM E-119 for one hour; Column B for two hours. Axial load was kept constant and axial deformations were measured until the temperature reached values close to ambient temperatures (about one day after the start of the cooling period). At this stage, the axial loads on the columns were increased at a rate of 12.5 kN per minute until failure of the columns was achieved. Column A failed at axial load of 2671 kN; Column B failed at 1987 kN. Temperatures were measured at different locations on the cross section of the columns during the fire tests. Further details are provided by Lie et al. (1986).

Numerical Modeling. VecTor3 (2008), a nonlinear finite element analysis program for three-dimensional reinforced concrete solid structures subjected to quasi-static load conditions developed at the University of Toronto, was implemented for the purpose of analysis in this study. The mechanical models in VecTor3 are based on the Modified Compression Field Theory (Vecchio and Collins, 1986), a rotating crack approach, and modified according to the Disturbed Stress Field Model (Vecchio, 2000). VecTor3 includes concrete tension stiffening and softening models, concrete crack models, advanced formulations for shear in concrete, and models for compression softening effects, confinement effects, and bond, to name a few. These are among the main parameters that contribute to the lateral response of reinforced concrete columns (Mostafaei and Kabeyasawa, 2007). In order to validate the capability of VecTor3 for the residual response estimation of reinforced concrete columns exposed to fire, analyses were first carried out for the column specimens under axial load and the results were compared and verified with the test data. The maximum temperatures reached at various depths, in the cross section, during the heating and cooling period were estimated from the test results (Lie and Woollerton, 1988) and illustrated in Fig. 1.

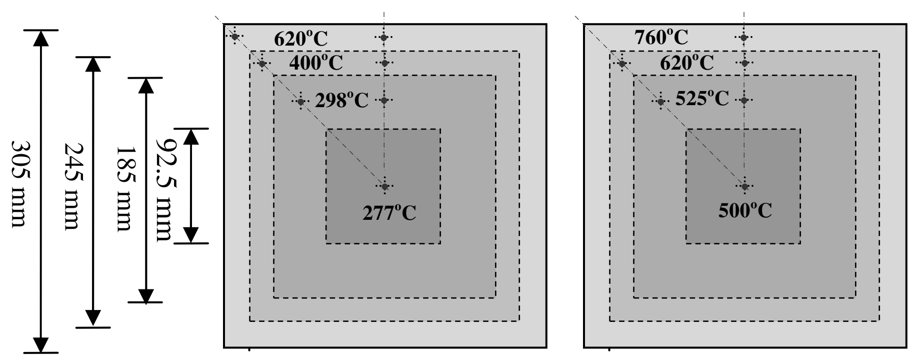


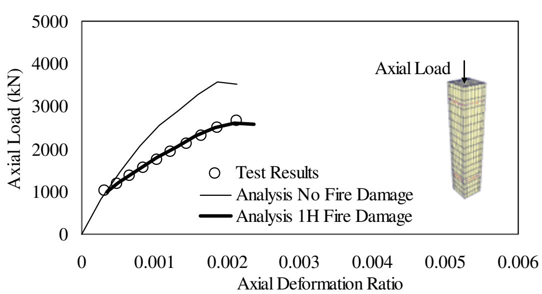
Fig. 1 Maximum averaged temperatures attained during the fire exposure in test

The average temperatures were interpolated and calculated based on the temperatures measured along both the diagonal section and axis of the section, as shown in Fig. 1. The residual compressive strength of concrete was then determined according to Eq. (4). Eq. (3) was used to determine the residual peak compressive strain. It was assumed that the reinforcing bars retained the original strength after the fire (Lie et al., 1986). The residual tensile strength of concrete was determined using Eq. (6). The sesidual mechanical properties of the material are provided in Table 1. Considering the column symmetry, half of the column height (1880 mm) was modeled by VecTor3. For the part of the column that was inside the furnace (1575 mm), the residual properties of concrete, shown in Table 1, were employed; for the non-exposed part of the column height, the original material properties were assigned. VecTor3 was implemented to estimate axial load–deformation ratio response of the column specimens.

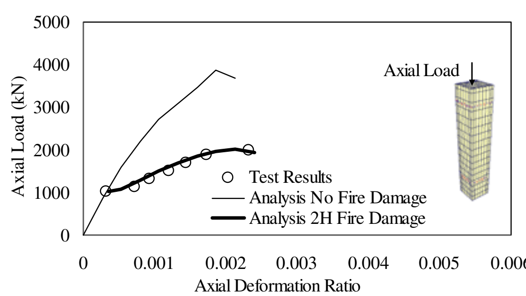
Table 1. Residual mechanical properties of concrete of the column specimens

Spec. No.	Depth from the edge (mm)	Max. Temperature Exposed (^oC)	f'_{cT} (MPa)	f'_{tT} (MPa)	E_{oT} (MPa)	ε_{oT}
Column A	15	620	14.6	0.7	7874	0.0037
	45	400	25.3	1.2	15150	0.0033
	83	298	30.2	1.4	19096	0.0032
	152	277	31.3	1.5	19962	0.0032
Column B	15	760	8.4	0.4	4245	0.0039
	45	620	15.7	0.7	8461	0.0037
	83	525	20.6	0.9	11638	0.0035
	152	500	21.9	1.0	12522	0.0035

The results were then compared with the load–deformation ratio measured from the last stage of the test; that is, one day after the fire exposure, from the onset of increasing axial load until column failure, reported by Lie and Woollerton (1988). Hence, in the comparison of the numerical and experimental results, the creep strain of the column measured during the one day cooling period was not included. Fig. 2 illustrates the comparison between the test and numerical results for the two columns, indicating consistent agreement. Furthermore, for the sake of comparison, the axial response of the two columns with no fire exposure, using the original properties, is estimated and provided in the same figure. The results show that residual axial capacity for Column A, one day after the fire exposure, was reduced to 73%; for Column B, the axial load capacity was reduced to 52% of the original.



(a) Column A



(b) Column B

Fig. 2 Axial load and deformation ratio relations: numerical and test results

The numerical results show that using Eq. (3), which gives less residual peak strain than that for unstressed concrete by Eq. (2), results in good agreement between the test and analysis. Fig. 3 illustrates the comparison between the two models for peak compression strain of concrete. Furthermore, the test data show that failure occurred for Column A after 7.3 mm axial deformation, or 0.0019 average compressive strain, and for Column B after 7.5 mm, or 0.0020 average compressive strain, from the onset of final loading. These deformations were calculated by deducting the axial deformation at the failure from that just before increasing the axial load in the last stage of the test. These relatively small axial deformations could indicate that concrete peak strains, of the two columns, were not affected significantly by the fire exposure, compared to the original values. Fig. 4 illustrates the history of the axial deformation for both columns, extracted from the test data. According to the results, for both specimens, even after ceasation of the fire exposure, thermal expansion continued to elongate the column. Furthermore, although they were exposed to different fire exposure times, both columns experienced almost identical maximum elongations. It appears that the first hour of the fire exposure, for both specimens, determined the magnitude of the maximum elongation due to thermal expansion. A comparison of the axial deformation response history for Column A and B shows substantially large creep deformation during the first day after the fire. The creep deformation for Column B was more than 30 mm. The effects of such a large deformation on the post-fire performance of reinforced concrete buildings need further study.

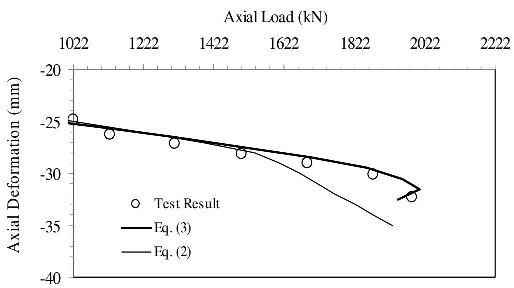


Fig. 3 Concrete peak compression strain models: Eq. (2) and Eq. (3) (Column B)

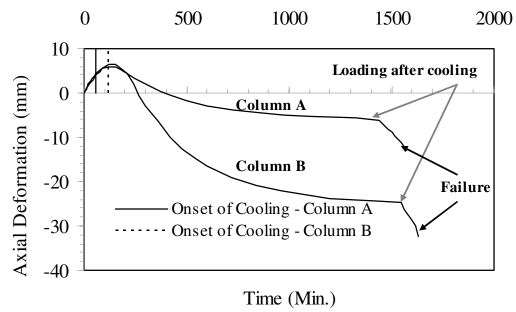
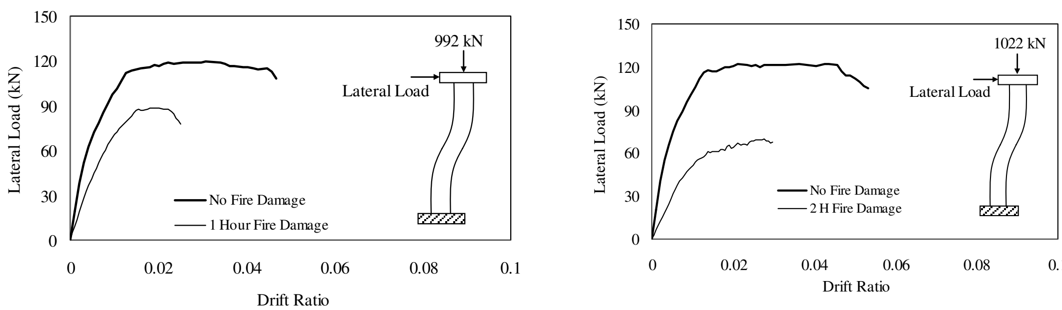


Fig. 4 History of axial deformations during the test and the onset of cooling

ASSESSMENT OF RESIDUAL SEISMIC/LATERAL RESPONSE

Numerical Modeling. VecTor3 was utilized to investigate the residual seismic/lateral load response of the two column specimens. The material properties and column specifications used were those from the previous analysis under axial load. The axial load for Column A was 992 kN, and for Column B, 1022 kN, held constant throughout the analysis. Half of the column was modeled based on the symmetric conditions. The lateral load was applied horizontally at the top of the columns, in deformation controlled mode, and incremented from zero up to the column failure. Fig. 5 shows the numerical results for the columns before and after fire exposure. The results indicate that the residual lateral load capacity for Column A is 73% and for Column B, 58% of the original lateral load capacity. With respect to the column ductility, the ultimate lateral deformation for Column A reduced to 48% of its original value with no fire damage and for Column B, to 61%. Therefore, according to this outcome, both the ductility and lateral/seismic load capacity of the columns were substantially reduced as a result of the fire exposure.

Strength Recovery. The strength loss of concrete due to fire exposure is largely recoverable in the long-term using a proper curing method (Weigier and Fisher, 1968; Poon et al. 2001). For instance, concrete that is exposed to a elevated temperature of 500^{o}C, recovers 90% of its original strength in one year (Lie et al., 1992). One might say that the likelihood of earthquake in one year, during which the concrete recovers, is low and therefore risk of failure of fire-damaged reinforced concrete buildings due to earthquake in the first year is negligible. However, as indicated previously, a survey of high-rise building fires shows that in the United States alone the annual fire occurrences in high-rise buildings exceed 10,000 incidents (Hall 2001). This means that in fact the likelihood of having a large number of fire-damaged buildings with less than one year recovery period experiencing an earthquake can be high. Further studies are required to investigate this issue both in short-term and long-term performance of structures.



(a) Column A: 1 hour fire damage (b) Column B: 2 hour fire exposure

Fig. 5 Lateral load and lateral deformation ratio response analysis

CONCLUSIONS

As a result of this study, the following remarks can be made:

- Analytical results show that the main seismic resistance properties of two reinforced concrete columns, namely the lateral load capacity and ductility, decreased substantially due to fire exposure.
- Application of the peak compressive strain model, available for stressed concrete at elevated temperatures, resulted in good agreement for the response estimation of the two fire-damaged column specimens.
- Large creep strains occurred during the cooling period of the reinforced concrete columns. The effects of such large deformations on post-fire performance of structures require further study.
- The first hour of the fire exposure was the most critical in determining the magnitude of the maximum elongation due to thermal expansion of the two columns.

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