Residual Post-Fire Behaviour of Pre-Damaged Confined Concrete

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LISBOA 2012

SUMMARY: The effects of pre-damage on residual strength and deformability after sequential loadings i.e. mechanical as well as thermal loadings are to be quantified for performance assessment of the structures after post earthquake fires. The present paper describes the concentric compression testing on pre-damaged confined concrete specimens exposed to elevated temperature ranging from room temperature to 900 ºC. The test variables were initial damage levels, temperature of exposure and tie as confining reinforcement. In this experimental program, the cylindrical concrete specimens of size 150 mm x 450 mm were firstly introduced initial mechanical damage by applying pre-determined uni-axial compressive loads for a desired displacement at ambient temperature. After this, these pre-damaged confined concrete specimens were exposed to a single cycle of heating and cooling. After cooling, the compression tests were conducted to assess the residual mechanical properties. An increased deformability and decreased residual strength were obtained.
KEYWORDS: Confined concrete, post earthquake fires, residual mechanical properties.

1. INTRODUCTION

The response in ductile behaviour of the structural components during a major earthquake has been reported during several seismic events. Although, it is important to dissipate seismic energy by post-elastic deformation in column hinging cannot be avoided entirely in most buildings during severe earthquakes. To sustain a large deformation without a dramatic loss of strength in columns, their potential plastic hinge regions should be reinforced with appropriately designed and detailed longitudinal and lateral confining steel. This philosophy is obviously based on strength enhancement due to confinement. It is reported earlier that the lateral steel would enhance section and member ductility. Hence, the ability of a structure to withstand a severe earthquake depends mainly on the formation of plastic hinges and their capacities to absorb and dissipate energy without significant loss of strength. The required ductility of reinforced concrete components is generally attained with the confinement of the concrete at core section (Sheikh and Uzumeri, 1980, Mander et al.1988). The parametric study of confinement on the strength and ductility of confined concrete at ambient temperatures are readily available (Cussion and Paultre, 1994, Sharma et al., 2005). With the ever-increasing use of confined concrete in the potential plastic hinge regions of earthquake resistant structures, significance has developed regarding the post-fire behaviour of such pre-damaged confined concrete zones.

The effects of high temperatures on the mechanical properties of concrete have been investigated by many researchers in the past (Abrams, 1971, Mohamedbhai, 1986, Phan and Carino, 1998). These studies were carried on unconfined concrete only and few studies related to the behaviour of confined concrete exposed to elevated temperatures. Little research has been undertaken to investigate the significance of confined end regions of reinforced concrete columns during fire (Kodur and MaGrath, 2006, Zaidi, et al. 2012). The need of ductility and strength in RC structures after earthquake has been recognized in the past. But, the required ductility is difficult to achieve due to the brittle nature of plain concrete, mainly in members subjected to high temperature after some initial damage. The present research paper deals with an experimental data to determine the residual strength and ductility in a pre-damaged confined concrete columns after exposure to high temperatures subsequent to mechanical pre-damage. Therefore, it has a potential application to high rise RC structures subjected to post-earthquake fire scenarios.

2. EXPERIMENTAL PROGRAM

2.1 Test Specimens

A total of 24 small-scale column specimens were prepared in three sets and tested under concentric axial compression. All columns contained six longitudinal rebars of 8 mm diameter which were equally spaced. The longitudinal rebars at the top and bottom surface of the specimens have a cover of 15 mm to prevent direct loading of the bar. The ties were 6 mm diameter with two spacing 42 mm and 68 mm. A concrete cover of 12.5 mm was provided in the confined concrete specimens of size 150 mm x 450 mm in cylindrical shape. Two thermocouples i.e. one at centre and one at cover-core interface were also embedded into the concrete before casting. The designation used for two different series (CB and CC) in the present study were as follows: C stands for concrete column specimens, second letter B and C indicated 42 mm and 68 spacing of confining tie reinforcements. The third digit shows the temperature of exposure i.e. 3 for 300 ºC, 6 for 600 ºC and 9 for 900 ºC. The fourth letter (W, X, Y and Z) specifies the level of initial damage. Three initial damage levels (X, Y and Z) along with an undamaged case (W) as explained in further section were selected in this study.

The damage levels for confined concrete specimens were derived from the load deformation curve at room temperature. The maximum allowable deformation was considered from the post-peak displacement when the load carrying capacity has undergone 15 % reduction (Park, 1988). The load-deformation relation does not have a well defined yield point due to nonlinear behaviour of the materials. Hence, the yield displacement of the equivalent elasto-plastic system with reduced stiffness was found as secant stiffness at 75 % of the peak load. Accordingly, the initial damage levels were derived from the load displacement curve in which the displacement corresponding to yield and 85 % post-peak load were marked as shown in Figure 1. The first level of initial damage, X was considered corresponding to the yield displacement (δ_y). The initial damage level, Y was defined as yield displacement (δ_y) plus 25 % of the difference between the displacement at 85 % post peak load (δ_{c85c}) and yield displacement (δ_y). The initial damage level, Z was taken as yield displacement plus 50 % of the difference between the displacement at 85 % post peak load (δ_{c85c}) and yield displacement (δ_y). These three levels of initial damage were induced as mechanical pre-damage in the confined concrete column specimens.

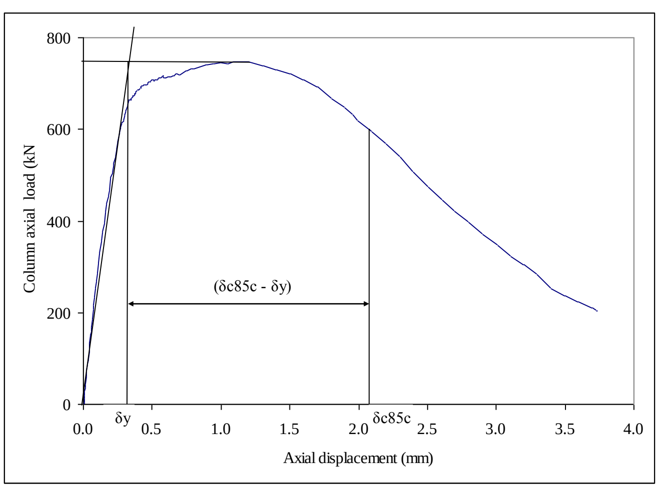

Figure 1. Initial damage determination for confined concrete

The concrete was made with ordinary Portland cement, natural river sand, crushed limestone aggregate of maximum size 12.5 mm and tap water. The concrete mixes of specified 28 days cylinder compressive strength (36.13 MPa) was employed. The longitudinal reinforcement of column consisted of 6 numbers of 8 mm diameter bars of 622.5 MPa yield strength, while 510 MPa of reinforcing steel with a diameter of 6 mm were used as lateral hoop reinforcement. The spacing of lateral ties used was 42 mm and 68 mm resulted to volumetric ratio 2.26 % and 1.40 % for CB and CC series of the specimens. The water curing period lasted for 28 days after which the specimens were kept in laboratory at room temperature and humidity conditions for another 62 days until they reached equilibrium moisture content. After 90 days, the specimens were exposed to various heating regimes subsequent to the application of mechanical pre-damages.

2.2 Instrumentation, Test Set up and Procedure

A programmable electrical furnace designed for a maximum temperature of 1200 ^{0}C was used to heat the specimens. The thermocouples were fixed in the specimens during casting stage at three different locations i.e. at their mid-height levels of surface, cover-core interface and centre of the specimen to record temperatures. The concrete specimens were heated in the furnace to different target temperatures ranging from room temperature to 900 ^{0}C. A maximum target temperature of 900 ^{0}C was considered to be reasonable for investigation. The lower limit of the maximum exposure temperature was taken as 300 ^{0}C, because no significant effects were found on the properties of heated concrete below this temperature. The heating rate was set at 10 ^{0}C /min, which has been shown to be realistic for structures exposed to fires. Each target temperature was maintained for three hours and then the samples were left in the furnace to allow natural cooling to room temperature. The mechanical testing of the specimens was carried out after a single cycle of heating and cooling. The test specimens were loaded using a 2500 kN UTM having displacement controlled capabilities as shown in Figure 2(a). The monotonic concentric compression was applied at a very slow rate (0.1 mm/minute) to capture the complete post peak behaviour of the measured load deformation curve. Failure of the specimens during uni-axial loading was forced in the test region, which was equal to 300 mm in the middle of the specimen height, by providing external confinement in the 75 mm on both end regions. The external confinement was obtained by fixing 20 mm thick steel collars in the top and bottom end regions to prevent premature end failure. The mean axial displacement of the central zone of the specimens (gauge length 200 mm) was measured and converted into an average strain over the measured base of the LVDTs. Average stress-strain relationships were found out by performing at least three tests for each type of the concrete specimens.

3. TEST RESULTS AND DISCUSSION

3.1 Thermal Behaviour

No thermal spalling of concrete was noticed in the confined concrete specimens during heating and cooling phase. The pre-damaged confined concrete specimens did not even show any distinct sign of thermal cracking when heated up to a temperature of 600 °C. However, some thermal cracks were seen in the specimens at 900 °C temperature. These cracks were probably caused by the physical changes and chemical decomposition of major concrete constituents at high temperatures. The temperature histories as recorded by the various thermocouples were closely monitored during the testing. The target temperatures and a steady state condition were obtained in the specimens subjected to higher temperatures as shown in Figure 3(b) for the specimens exposed to 600 °C. Furthermore, the Figure 3(b) shows a kink in the rate of temperature rise at the center of the specimen cross section compared to the surface temperature. This is the point when first free water and later chemically bound water is released from the specimens due to heating [Phan, L T 2001].

(a) (b)

Figure 2. (a). Test set up with specimen in place (b) Appearance of CB tested specimens

(a) (b)

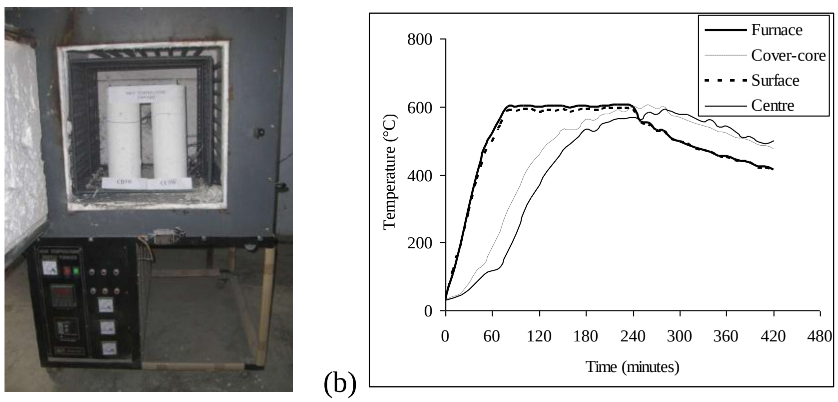

Figure 3. Temperature histories of CC6 series (a) Electric furnace (b) Time-temperature curves

3.2 Residual Stress-Strain Behaviour

The response of the pre-damaged confined specimens between, P, applied axial load and the average axial strain measured by the LVDTs are given in Table 1. To facilitate the comparison of behaviour of different specimens, the ordinates in the curves have been non-dimensionalised with respect to the (P_o) theoretical capacity of the specimens. Figure 2 (b) shows the appearance of specimen from each batch exposed to 600 °C after thermo-mechanical testing. Figure 3(b) illustrates the appearance of typical specimens of the CB series after testing. Each curve in Figures 4(a)-(m) and each result in Tables 1 represents the average result of three specimens.

Table 1. Test results of residual properties

Specimens	Exposure Temperature (°C)	P_{max} (kN)	P_{max} / P_o	ε_{cc} (micron)	ε_{cc}/ ε_{co}
CB	Room	792.16	1.06	13000	6.047
CB3W	300	841.44	1.07	12200	5.674
CB3X	300	866.74	1.10	11500	5.349
CB3Y	300	840.00	1.07	11700	5.442
CB3Z	300	822.36	1.04	9600	4.465
CB6W	600	726.60	0.98	20500	9.535
CB6X	600	716.94	0.97	22400	10.419
CB6Y	600	688.60	0.93	22100	10.279
CB6Z	600	713.25	0.96	24800	11.535
CB9W	900	334.62	0.51	31500	14.651
CB9X	900	365.58	0.56	29400	13.674
CB9Y	900	364.43	0.55	32700	15.209
CB9Z	900	371.01	0.56	29000	13.488
CC	Room	747.20	1.01	5500	2.558
CC3W	300	710.04	0.92	7400	3.442
CC3X	300	724.21	0.93	8100	3.767
CC3Y	300	707.56	0.91	5800	2.698
CC3Z	300	710.74	0.92	7300	3.395
CC6W	600	599.33	0.82	16300	7.581
CC6X	600	603.15	0.83	14300	6.651
CC6Y	600	583.32	0.80	15000	6.977
CC6Z	600	565.56	0.78	13500	6.279
CC9W	900	249.04	0.39	20700	9.628
CC9X	900	221.26	0.34	18800	8.744
CC9Y	900	222.13	0.34	22000	10.233
CC9Z	900	229.80	0.36	16500	7.674

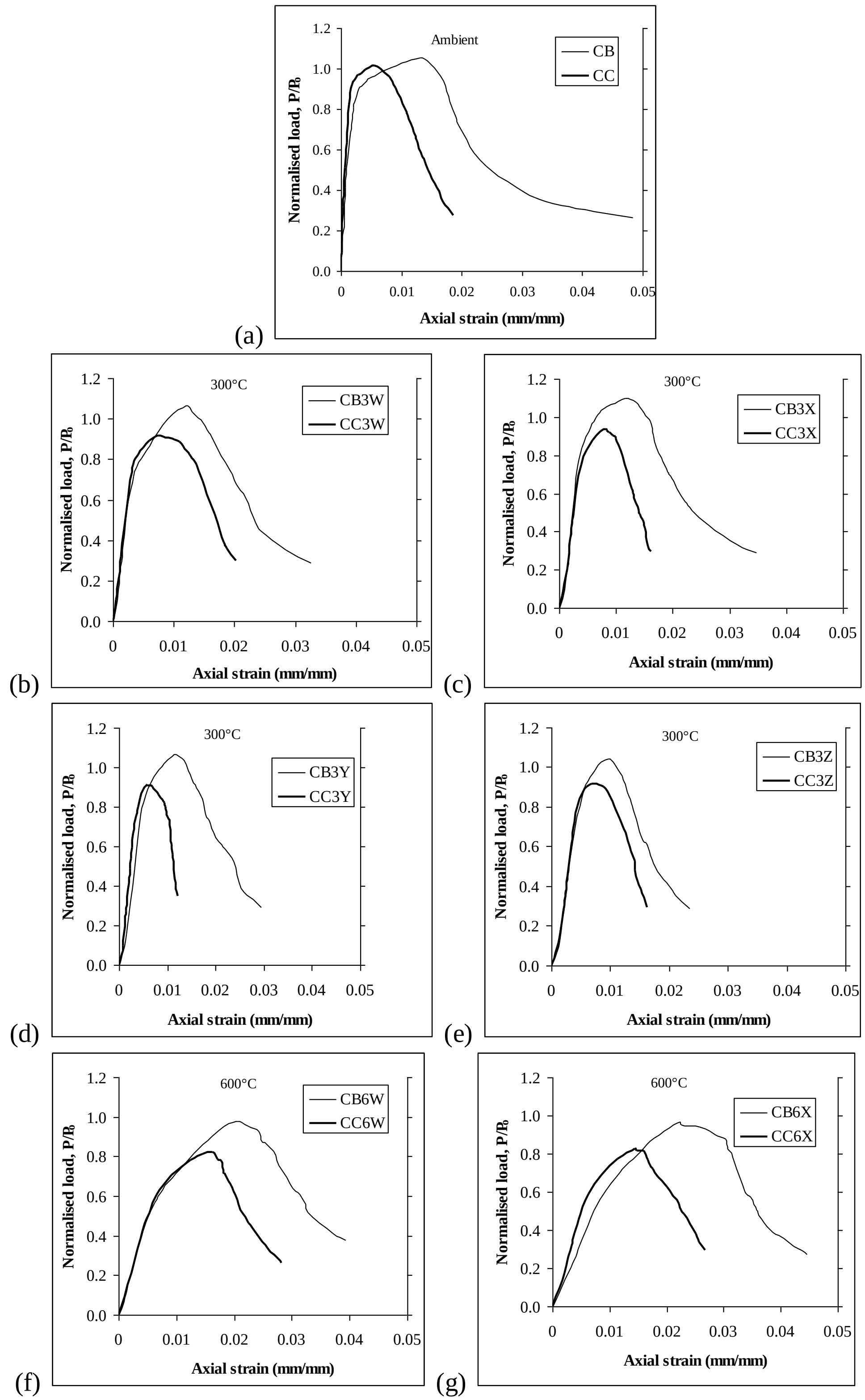

Figure 4. (a-g). Normalized applied load vs axial-strain for pre-damaged confined column (continues)

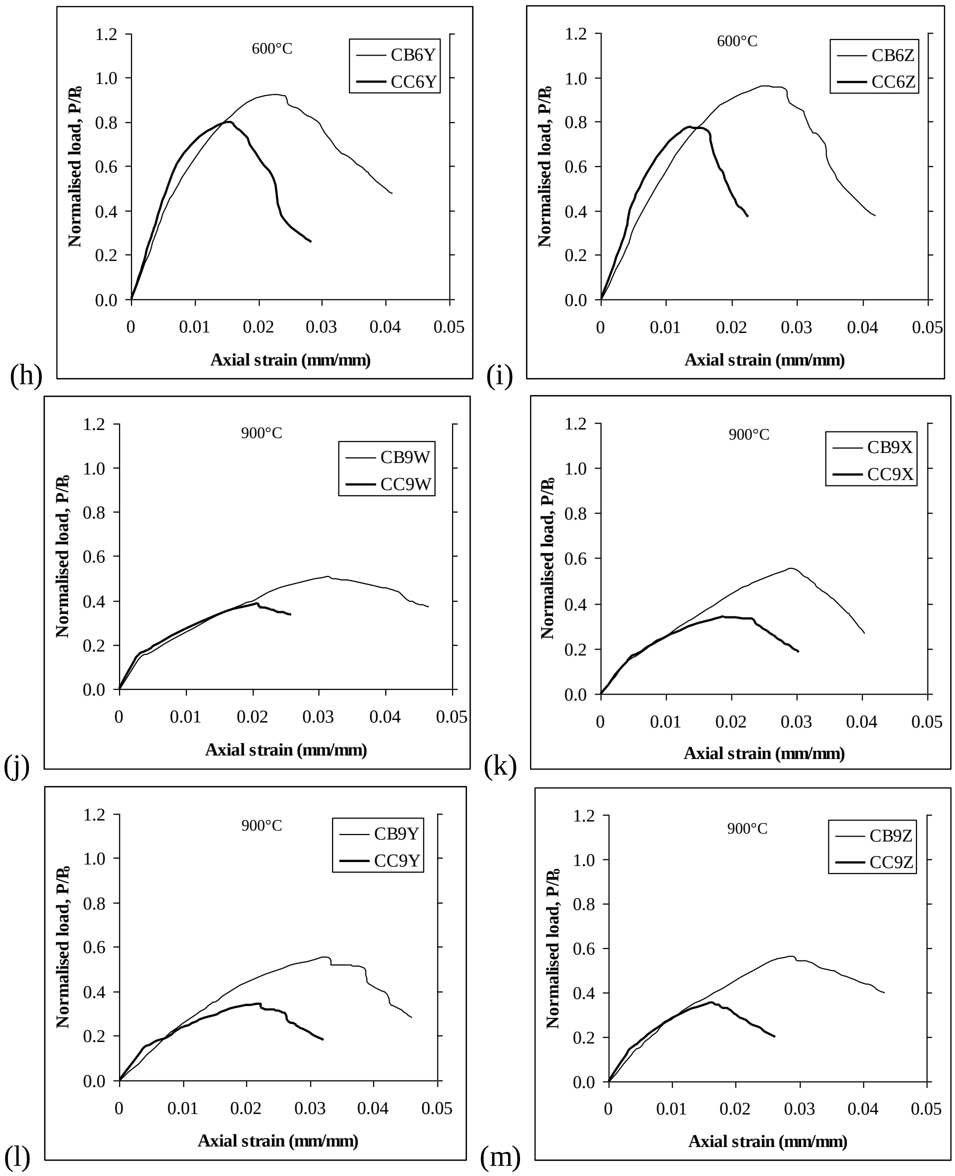

Figure 4. (h-m). Normalized applied load vs axial-strain for pre-damaged confined column (continued)

3.3 Residual Strength and Strain of Confined Concrete

All the pre-damaged confined concrete specimens initially behaved in a similar manner and exhibited a relatively linear load deformation response in the ascending part. The linear ascending portion was longer in the specimen's heated at 300 ºC temperatures than the specimens exposed to higher temperatures. The cracking in the concrete cover often resulted in a non-linear response with axial deformation increasing at a faster rate compared to the applied load. Post-peak behaviour of specimens depended solely upon their confinement levels and temperature of exposure. Higher the degree of confinement and temperature of exposure, more ductile was the post-peak response. The failure of the pre-damaged confined concrete specimens under compressive loads was observed to be of shear type for specimens exposed to temperature at 300 ºC. Above 600 ºC, the failure of the confined concrete specimens was marked by significant lateral dilation of concrete and softening of strain. This gradually led to the fracture of hoops and buckling of longitudinal steel in many cases. The residual load ratio P_{max}/P_o ranges from a maximum values of 1.10 at 300 ºC temperature to a minimum value of 0.34 at 900 ºC temperature for the pre-damaged confined concrete specimens tested under this program. It can be observed that the residual load capacity does not get affected in the temperature at 300 ºC. In fact, the residual peak load and hence the load ratio P_{max}/P_o increase slightly at temperature of exposure 300 ºC in most of the specimens. In case of specimens of CC series, which were relatively poorly confined, the residual load ratio, P_{max}/P_o began to decrease at 300 ºC temperature and further a considerable reduction in load capacity at 600 ºC temperature. It drops markedly in the specimens exposed to temperatures at 900 ºC. The results indicate that the peak load of the confined concrete specimens drops to 78 to 80 % of the theoretical concentric capacity at 600 ºC temperature. It is only beyond 600 ºC temperature that the carrying capacity of the pre-damaged confined concrete specimens falls to 34 to 40 % of the corresponding theoretical capacity. The strains ε_{cc} corresponding to the observed computed peak confined load was computed for all the specimens. To characterize the residual deformability of confined concrete specimens, these strain values were then normalized with respect to the corresponding unconfined concrete strain 2150 micron (ε_{co}), measured at ambient temperature (Zaidi, et al. 2012). It can be observed that the strain ratios $\varepsilon_{cc}/\ \varepsilon_{co}$ vary from 2.56 to 15.21 for the pre-damaged confined concrete specimens as the temperature of exposure increased from room temperature to 900 ºC. These strain ratios are several times larger in case of heated pre-damaged specimens than in comparable unheated specimens. However, the various strain values and the corresponding strain ratios do not vary significantly at the initial damage X, Y and Z following a temperature exposure of 300 ºC. It is only in the temperature range of 600 to 900 ºC that the peak strain ratios increase considerably irrespective of initial damage level. Nevertheless, the influence of temperature was more pronounced above 300 ºC regardless of the initial damage level and the degree of confinement.

3.4 Effect of Test Variables

Figures 5 and 6 demonstrate the effect of volumetric ratio, initial damage level, exposure temperature and spacing of confining reinforcement on the normalised axial load-deformation behaviour of confined concrete. The results indicate that the larger the volumetric ratio or closer the spacing of lateral ties, the more ductile is the behaviour irrespective of the pre-damage level as well as temperature of exposure. The pre-damaged confined concrete specimens with reduced volumetric ratio or increased spacing of lateral steel exhibited a faster rate of strength decay in the post-peak region. In case of the specimens tested at ambient temperature, the residual confined strength ratio P_{max}/P_o increased from 1.01 to 1.06 and the strain ratio $\varepsilon_{cc}/\ \varepsilon_{co}$ increased from 2.56 to 6.05 as the volumetric ratio of lateral hoops was increased from 1.40 to 2.26. The corresponding enhancements were from 0.36 to 0.56 in the load ratio P_{max}/P_o and from 7.67 to 13.49 in the strain ratio $\varepsilon_{cc}/\ \varepsilon_{co}$ as the volumetric ratio of hoops were increased in case of specimens exposed to 900 ºC. The results indicate that an increase in the amount of lateral confinement leads to more limited thermally induced losses in terms of confined concrete strength, especially in the temperature range of 300 to 600 ºC irrespective of the initial damage levels. Further, in the temperature range of 600 to 900 ºC, increasing the amount of confinement result in an even greater peak strains indicating still more deformability.

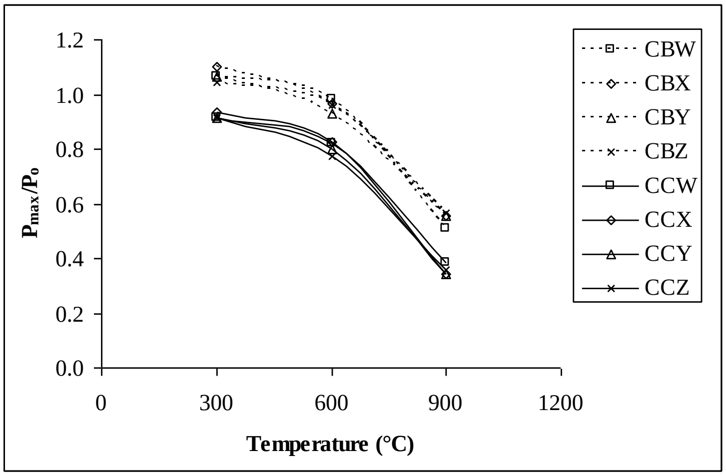

Figure 5. Variation of load ratio P_{max}/P_o with temperature of pre-damaged confined column

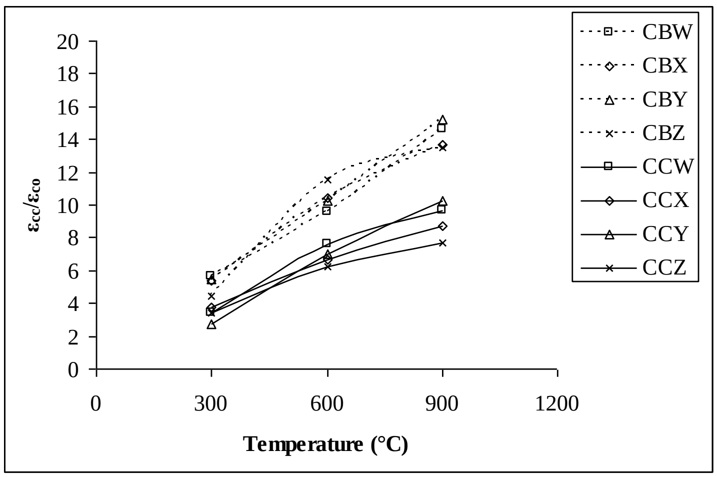

Figure 6. Variation of strain ratio $\varepsilon_{cc}/\varepsilon_{co}$ with temperature of pre-damaged confined column

5. CONCLUSIONS

This study reports the residual properties of 24 pre-damaged confined normal strength short columns concrete specimens subjected to elevated temperatures subsequent to initial damage. Within the scope of the present investigation, the following conclusions may be drawn: 1) The thermal performance of pre-damaged confined concrete is better at lower temperature of exposure than that at the higher temperature in terms of reduced thermal cracking, spalling and other symptoms of distress. 2) The combined detrimental effects of temperature and initial damage on the residual load deformation behaviour of confined concrete do not effect much at temperature of 300 ºC. Further, the load carrying capacity of pre-damaged confined concrete specimens drops only to 80 to 90 % of the corresponding room temperature theoretical concentric capacity up to a temperature exposure of 600 ºC with an enhancement in deformability. 3) The effect of volumetric ratio of confining hoops on the behaviour of the confined concrete appeared to be similar for both pre-damaged and undamaged specimens subjected to high temperature i.e. residual strength and ductility improved with an increase in the confinement.

ACKNOWLEDGMENTS

The present research work reported in the paper was sponsored by the U.K.-India Education and Research Initiative (UKIERI) project. The experimental program was conducted in Structural Engineering laboratory of the department of civil engineering at Indian Institute of Technology Roorkee, India.

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