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RESIDUAL STRENGTH AND DEFORMATION CHARACTERISTICS OF CONFINED CONCRETE SUBJECTED TO ELEVATED TEMPERATURE

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ABSTRACT

The objective of this study is to gain the insight of the strength and deformability of confined concrete after exposure to a thermal cycle at high temperature. In this context, an experimental program was designed and carried out to study the residual compressive stress-strain behavior of confined concrete at elevated temperatures ranging from room temperature to 800^0 C. The experimental variables were exposure temperature, concrete strength, amount of confining reinforcement and yield strength of transverse confining steel. A total of 108 unconfined and confined cylindrical specimens of size 150 mm x 450 mm were cast and tested. The effects of various key variables of confinement were studied and quantified with respect to strength and ductility gains. The residual strength, strain corresponding to the peak load and post-peak strains of confined concrete are not affected significantly up to an exposure temperature of 300^0 C. However, the peak load falls and corresponding strains increase considerably in the temperature range of 400 to 800^0 C. It was observed that increasing the exposure temperature makes the stress-strain curve of confined concrete flatter. Furthermore, the larger the degree of confinement reinforcement, the greater the residual strength and deformability of confined concrete.

Introduction

In the modern day construction, the structures are designed to behave in a ductile manner to resist natural and man- made hazards like earthquake, fire and blast loading. Thus inelastic deformability of reinforced concrete elements is essential for overall stability of structures in order to sustain these hazards. Deformability of reinforced concrete structural components is generally achieved through proper confinement of the core concrete. The increase in strength and ductility of concrete confined by well-detailed lateral confinement reinforcement is well-documented now at ambient conditions (Sheikh and Uzumeri 1980, Mander et al. 1988, Razvi and Saaticioglu 1994, Sharma et al. 2005). Although most of the concrete structures are

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subjected to a range of heat no more severe than that caused by the weather, there are important cases in which structures are exposed to much higher temperatures. Examples include building fires and some industrial applications where reinforced concrete structural elements are close to furnaces and reactors. Such fires or elevated temperatures result, in most cases, in considerable damage to structures.

The effects of high temperatures on the mechanical properties of unconfined concrete have been investigated by many researchers in the past (Mohamedbhai 1986, Phan and Carino 1998, Kodur and Phan 2007). Therefore, it becomes important to evaluate the effectiveness of confinement reinforcement in confining core concrete after exposure to elevated temperatures, especially in seismic resistant structures. It also remains to be seen that how the various parameters of confinement would affect the behavior of concrete after having exposed to elevated temperatures. Thus the main aim of the present investigation is to provide experimental data on the residual behaviour of confined concrete subjected to high temperature.

Experimental Program

A total of 108 concrete short column specimens were cast and tested under this investigation. They included 84 numbers of hoop confined specimens and 24 unconfined specimens. All the specimens were of cylindrical shape with a size of 150 mm x 450 mm. The details of all the confined specimens are illustrated in Table 1. The unconfined specimens (Table 2) were of the same size and shape as confined specimens but without reinforcement. The specimens were cast and tested in triplicate in order to get the average of three results thus making 28 confined and 8 unconfined independent column designs. The experimental variables included spacing of hoop reinforcement, yield strength of confining reinforcement, concrete strength and temperature of exposure. The confined and unconfined concrete specimens were cast in five different series (A, B, C, D and H). Each series of the confined specimens consist of specimens with the same confinement parameters and concrete strength but different exposure temperatures. The longitudinal reinforcement consisted of 8 mm diameter bars of 650 MPa yield strength and two different grades (510 MPa and 726 MPa) of reinforcing steel with a diameter of 6 mm were used as lateral hoop reinforcement. The standard plain concrete cylinders (100 x 200 mm) were also cast and tested to determine the nominal strength of concrete on the day of testing of test specimens.

A concrete cover of 12.5 was provided in all the confined column specimens. A cover of 15 mm was also provided between the ends of the longitudinal bars at the top and bottom surface of the specimens to prevent direct loading of the bar. Failure of the specimens 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 end regions of 75 mm by 20 mm thick steel collars in order to prevent premature end failure. The specimens were cast in PVC moulds in the laboratory. After 24 hours, the specimens were removed from the moulds and submerged in a water tank for curing. The water curing period lasted for 28 days after which the specimens were kept in the laboratory at ambient temperature and humidity conditions for another 62 days until they reached equilibrium moisture content. After 90 days of ageing, the specimens were exposed to various heating regimes. Subsequent to a single heating and cooling cycle, the specimens were tested under monotonic compression.

A programmable electrical furnace design for a maximum temperature of 1200^0C was used to heat the specimens. The temperature inside the furnace was measured and recorded with specially installed thermocouples. The thermocouples were also fixed during casting at mid height of the specimens at three different locations inside the cylinder i.e. at the surface, at cover core interface and at the centre of the specimen to record temperature histories. At an age of 90 days the specimens were heated in the furnace to different target temperatures ranging from ambient temperature to 800^0C. Generally, during fires a maximum temperature in the range of 1000^0C to 1200^0C is reached. However, such high temperatures occur only at the surface of concrete and that too for a short duration. Therefore, a maximum temperature of 800^0 C was considered to be reasonable. The lower limit of the maximum exposure temperature was taken as 200^0C (Mohamedbhai 1986) because no significant effect were found on the properties of heated concrete below this temperature. Heating rate was set at 5^0C /min and the each target temperature was maintained for 4.5 hrs to achieve a thermal steady state. After reaching the maximum desired temperature the furnace was switched off, and samples were left in the furnace to allow natural cooling. The rate of cooling was not controlled but was measured during complete test. The data from thermocouples were recorded in a PC by data logger.

The mechanical testing of specimens was carried out after a complete cycle of heating and cooling. The test specimens were loaded using 2500 kN capacity (INSTRON) UTM with displacement controlled capabilities. The monotonic concentric compression was applied at a very slow rate (0.1mm/min) to capture the complete post peak behavior of the measured load deformation curve. The axial contraction of the cylindrical specimens was monitored by the average of the data of two linear variable displacement transducers (LVDT) placed on the circumferential opposite to each other of the specimens. The mean axial displacement of the central zone of the specimens (gauge length 200mm) was measured and converted into an average strain over the measured base of the LVDTs. Loads were recorded through a load cell inbuilt in the UTM. The recorded data from the LVDTs and load cell were fed into a data acquisition system and stored on a computer.

Observations

The confined and unconfined concrete cylinders did not show any distinct sign of cracking when heated up to a temperature of 600^0C. However beyond this temperature, cracks were noticed on the heated surface along the members heated at 800^0C in both the concretes. These cracks were probably caused by thermal effects such as internal resisting stress, drying shrinkage of the concrete or the difference between the thermal expansion coefficient of the reinforcement and concrete during heating and cooling of the specimens. No observable symptoms of spalling were noticed during heating and cooling cycle in any of the unconfined and confined specimens. The temperature histories as recoded by the various thermocouples were closely monitored during the testing. A remarkable difference was observed in the heating trends. The target temperatures could not be achieved in the specimens exposed to 200^0 C and 300^0 C temperatures. Therefore, a true steady state condition could not be achieved in these specimens. However, the target temperatures and a steady state condition were obtained in the specimens subjected to higher temperatures.

The residual compressive behavior of unconfined concrete cylinders exposed to temperatures ranging from room temperature to 500^0 C exhibited brittle longitudinal splitting. While the unconfined specimens subjected to 600^0 C temperature showed comparatively a soft failure mode, the one exposed to 800^0 C temperature could not bear any load. The failure of confined concrete specimens under compressive loads was observed to be of shear type for temperatures up to 400^0 C. Above 500^0 C, the failure of confined concrete specimens was marked by significant lateral dilation of concrete and softening of strain mainly at their mid height. This gradually led to the fracture of hoops and bucking of longitudinal steel in many cases. The failure of specimens cast with higher strength concrete mix was comparatively less ductile compared to the specimens constructed with lower strength concrete mix. Fig. 1 shows the appearance of some of the specimens after testing.

Test Results

The test results are given in Table 2 for unconfined specimens and in Table 3 for confined concrete specimens. Each result represents the average result of three specimens. To facilitate the comparison of behavior of different confined concrete specimens, the maximum observed load, $P_{max,}$ has been nondimensioalized with respect to the concentric theoretical capacity of specimens, P_o, at ambient temperature. Similarly, the strain corresponding to the peak load, $\varepsilon^{/}$, and the post-peak strain, $\varepsilon^{/}_{80,}$ have been normalized with respect to the peak strain of the corresponding unconfined concrete, ε_{co}, at ambient temperature.

The residual load ratio $P_{max}/\ P_o$ ranges from a maximum value of 1.12 at ambient temperature to a minimum value of 0.40 at 800^0 C temperature for the confined concrete specimens tested under this program. It can be observed that the residual peak load of confined specimens does not get affected in the temperature range of 100 to 400^0 C. Infact, the peak load and hence the load ratio $P_{max}/\ P_o$ increases slightly up to a temperature of 300^0 C in most of the specimens. Similar trends were noticed in unconfined specimens as well. In the temperature range of 500^0 C to 800^0 C, the peak load in each case drops markedly. However, the results indicate that the peak load of confined specimens drops only to 76 to 81% of the theoretical concentric capacity up to a temperature of 600^0 C. It is only beyond 600^0 C temperature i.e. at 800^0 C temperature that the carrying capacity of confined concrete specimens falls to 40 to 49% of the theoretical capacity. However in unconfined concrete specimens, the residual maximum load drops sharply even after 400^0 C.

The strain corresponding to peak load, $\varepsilon^{/}$, and the post peak strain, $\varepsilon^{/}_{80}$ (axial strain at which the load drops to 80% of the peak load) were computed for all the confined concrete specimens as reported in Table 3. To characterize the deformability of confined concrete specimens, these strain values were then normalized with respect to the unconfined concrete strain, ε_{co}, measured at ambient temperature. It can be observed that the strain ratio $\varepsilon^{/}/\varepsilon_{co}$ increases from a minimum value of 1.52 at ambient temperature to a maximum value of 13.35 at 800^0 C temperature in the confined concrete specimens. The strain ratio $\varepsilon^{/}_{80}/\varepsilon_{co}$ ranges from a minimum value of 3.08 at ambient temperature to a maximum value of 18.98 at 800^0 C temperature. Both these strains and hence the corresponding strain ratios do not vary significantly within the temperature range of 100 to 300^0 C. However, in the temperature range of 400^0 C to 800^0 C, both peak and post-peak strains increase considerably.

Effect of Test Variables

Fig. 2 demonstrates the effect of the volumetric ratio and spacing of the confining reinforcement on the axial load-deformation behavior of confined concrete specimens exposed to varying temperatures. The load-strain responses of unconfined concrete specimens are also shown for comparison purpose. The results indicate that the larger the volumetric ratio or closer the spacing of lateral steel, the more ductile is the behavior irrespective of the temperature of exposure. The column specimens with reduced volumetric ratio or increased spacing of lateral steel exhibit a faster rate of strength decay after the peak. The residual peak load ratio P_{max}/ P_o increased from 1.005 to 1.10, peak strain ratio $\varepsilon^{/}/\varepsilon_{co}$ increased from 2.23 to 4.70 and the post-peak strain ratio $\varepsilon^{/}_{80}/\varepsilon_{co}$ increased from 6 to 8 as the volumetric ratio of lateral hoops increased from 1.40 in to 2.26 at ambient temperature. The corresponding enhancements were from 0.40 to 0.49 in load ratio P_{max}/ P_o, from 9.30 to 12.05 in the strain ratio $\varepsilon^{/}/\varepsilon_{co}$ and from 12.79 to 14.84 in the post-peak strain ratio $\varepsilon^{/}_{80}/\varepsilon_{co,}$ as the volumetric ratio of hoops was increased in the specimens exposed to 800^0 C. It can be observed that increasing the amount of lateral confinement leads to more limited thermally induced losses in terms of load carrying capacity, especially in the temperature range of 400^0 C to 800^0 C. Further, in the temperature range of 400^0 C to 800^0 C, increasing the amount of confinement results into even greater peak and post-peak strains indicating still more deformability.

The significance of varying the yield strength of lateral hoops is shown in Fig 3, where different pairs of specimens have been compared. The compared specimens of each pair had the same concrete strength as well as the same volumetric ratio of lateral steel and similar temperature of exposure but different yield strengths. It can be noticed that while the load ratio P_{max}/ P_o did not vary much with an increase in the yield strength of transverse confining reinforcement up to 300^0 C temperature, a reduction in P_{max}/ P_o was noticed in the temperature range of 400^0 C to 800^0 C when higher yield strength hoops were used. No clear trends could be noticed in terms of peak and post-peak strains with regard to this test parameter. Further research is needed to establish the effect of temperature on the possible advantages of using higher yield strength confining reinforcement.

In this study it was possible to observe the effect of varying the concrete strength by comparing the residual behaviour of specimens of CBN series with CBH series (Fig. 4). The residual load-strain curves of the specimens with the same volumetric ratio, spacing and yield strength of lateral steel and longitudinal steel but with different concrete strengths were compared to quantify the effects of this parameter. The post peak curves of the higher strength concrete specimens are distinctly steeper indicating a faster rate of strength decay as compared to the lower strength concrete specimens. Similar trends were noticed for the specimens tested at ambient temperature and those exposed to higher temperatures. A considerably higher peak ($\varepsilon^{/}$) and post-peak strains ($\varepsilon^{/}_{80}$) were noticed in lower strength concrete mix compared to higher strength concrete mix. Enhancement in the load carrying capacity was also observed to be less in the specimens constructed with higher strength concrete except in the specimens exposed to 300^0 C and 400^0 C temperatures, where a better load ratio (P_{max}/ P_o) was noticed in higher strength concrete specimens.

Conclusions

This study reports the results of 108 hoop confined concrete specimens exposed to varying temperatures and subsequently tested under concentric compression. The effect of testing conditions such as frictional restraint between the loading platens and the specimen, the gauge length, the stiffness of the testing machine, the loading rate and the shape and the size of the specimen are also equally important, however, the same could not be considered in the present study. Within the scope of the present investigation, the following conclusions may be drawn:

1. For the range of values considered in the present study, the effect of volumetric ratio of confining hoops on the behaviour of confined concrete appeared to be similar for both at ambient temperature and at elevated temperatures i.e. residual strength and ductility improved with the increase in confinement.
2. No specific trends could be observed with respect to yield strength of confining steel. Further research is underway to fully investigate the influence of this parameter on confined concrete exposed to elevated temperatures.
3. An increase in the concrete strength results into lower post-peak deformability both for specimens tested at ambient temperature and those exposed to elevated temperatures.
4. The effect of temperature on residual behavior of confined concrete and the influence of parameters of confinement do not matter much up to a temperature of 300^0 C. Further, the load carrying capacity of confined concrete specimens drops only to 76 to 81% of the corresponding ambient temperature theoretical concentric capacity up to a temperature exposure of 600^0 C against a drop of more than 50% in case of unconfined concrete specimens. Nevertheless, this nominal drop in load carrying capacity of confined concrete specimens up to 600^0 C is associated with a considerable enhancement in deformability.

Notations

f'_c = cylinder compressive strength of concrete
f_y = yield strength of steel
P_{co} = unconfined strength of concrete specimen
P_{max} = maximum load capacity of the confined specimen
P_o = theoretical concentric capacity of specimen
ε_{co} = strain at peak load of unconfined concrete specimen
ε' = axial strain corresponding to the peak load of confined concrete specimen
ε'_{80} = axial strain at which the load drops to 80% of the peak load
ρ_s = volumetric ratio of hoops
ρ_t = ratio of longitudinal reinforcement

Table 1 Properties of confined concrete specimens

Specimens	f'_c (MPa)	Longitudinal steel			Transverse steel			
		Number & Diameter (mm)	ρ_t %	f_y MPa	Diameter (mm)	Spacing (mm)	ρ_s %	f_y (MPa)
CBN CB2N CB3N CB4N CB5N CB6N CB8N	41.92	6Nos 8	1.70	650	6	42	2.26	510
CCN CC2N CC3N CC4N CC5N CC6N CC8N	41.88	6Nos 8	1.70	650	6	68	1.40	510
CDN CD2N CD3N CD4N CD5N CD6N CD8N	41.86	6Nos 8	1.70	650	6	42	2.26	726
CBH CB2H CB3H CB4H CB5H CB6H CB8H	71.36	6Nos 8	1.70	650	6	42	2.26	510

Table 2 Properties and results of unconfined specimens

Specimens	$f'c$ (MPa)	Temperature of exposure (^{0}C)	Pco (KN)	ε_{co}
CAN	41.78	Ambient	694	0.00215
CA2N	41.78	200	721	0.00203
CA3N	41.78	300	723	0.00253
CA4N	41.78	400	689	0.0033
CA5N	41.78	500	599	0.0049
CA6N	41.78	600	323	0.0080
CA8N	41.78	800	–	–
CAH	71.63	Ambient	1238	0.00208

Table 3 Results of confined concrete specimens

Specimen	Exposure Temperature (^{0}C)	P_{max}	P_{max}/P_o	$\varepsilon^{/}$	$\varepsilon^{/} / \varepsilon_{co}$	$\varepsilon^{/}_{80}$	$\varepsilon^{/}_{80} / \varepsilon_{co}$
CBN	AMBIENT	906	1.10	0.0101	4.70	0.0172	8.00
CB2N	200	929	1.13	0.0101	4.70	0.0139	6.47
CB3N	300	912	1.11	0.0102	4.74	0.0163	7.58
CB4N	400	865	1.05	0.0101	4.70	0.0179	8.33
CB5N	500	821	1.04	0.0138	6.42	0.0196	9.12
CB6N	600	668	0.81	0.0210	9.77	0.0248	11.53
CB8N	800	401	0.49	0.0259	12.05	0.0319	14.84
CCN	AMBIENT	821	1.005	0.0048	2.23	0.0129	6.00
CC2N	200	880	1.07	0.0092	4.28	0.0148	6.88
CC3N	300	781	0.95	0.0070	3.26	0.0119	5.53
CC4N	400	749	0.91	0.0100	4.65	0.0172	8.00
CC5N	500	702	0.86	0.0112	5.21	0.0147	6.84
CC6N	600	589	0.72	0.0160	7.44	0.0224	10.42
CC8N	800	327	0.40	0.0200	9.30	0.0275	12.79
CDN	AMBIENT	916	1.12	0.0110	5.12	0.0154	7.16
CD2N	200	919	1.12	0.0110	5.12	0.0143	6.65
CD3N	300	935	1.14	0.0100	4.65	0.0136	6.33
CD4N	400	824	1.01	0.0105	4.88	0.0164	7.63
CD5N	500	794	0.97	0.0150	6.98	0.0185	8.60
CD6N	600	620	0.76	0.0160	7.44	0.0210	9.77
CD8N	800	380	0.46	0.0287	13.35	0.0408	18.98
CBH	AMBIENT	1348	1.07	0.00316	1.52	0.0064	3.08
CB2H	200	1358	1.08	0.00371	1.79	0.0069	3.32
CB3H	300	1449	1.15	0.00377	1.81	0.0074	3.56
CB4H	400	1385	1.10	0.00798	3.84	0.011	5.29
CB5H	500	1214	0.97	0.01030	4.95	0.0161	7.74
CB6H	600	952	0.76	0.01402	6.74	0.0218	10.48
CB8H	800	590	0.45	0.0200	9.62	0.0310	14.90

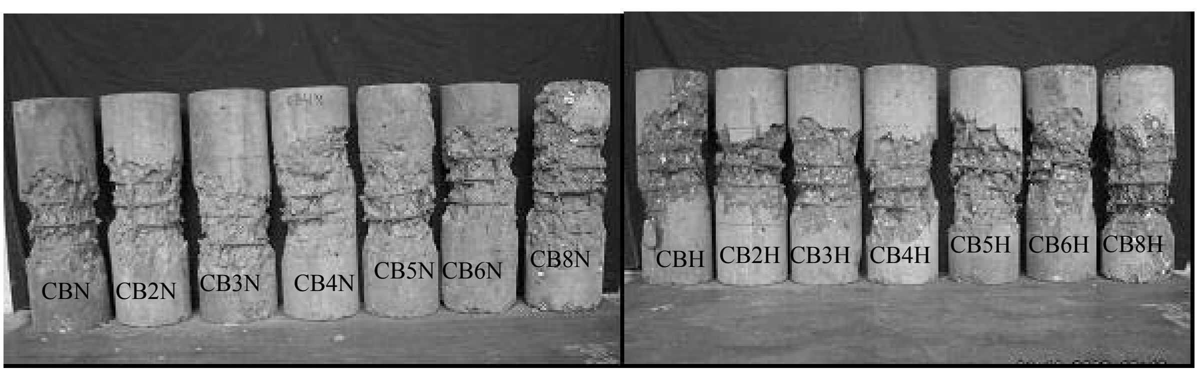

Figure 1 Appearance of some of the specimens after testing

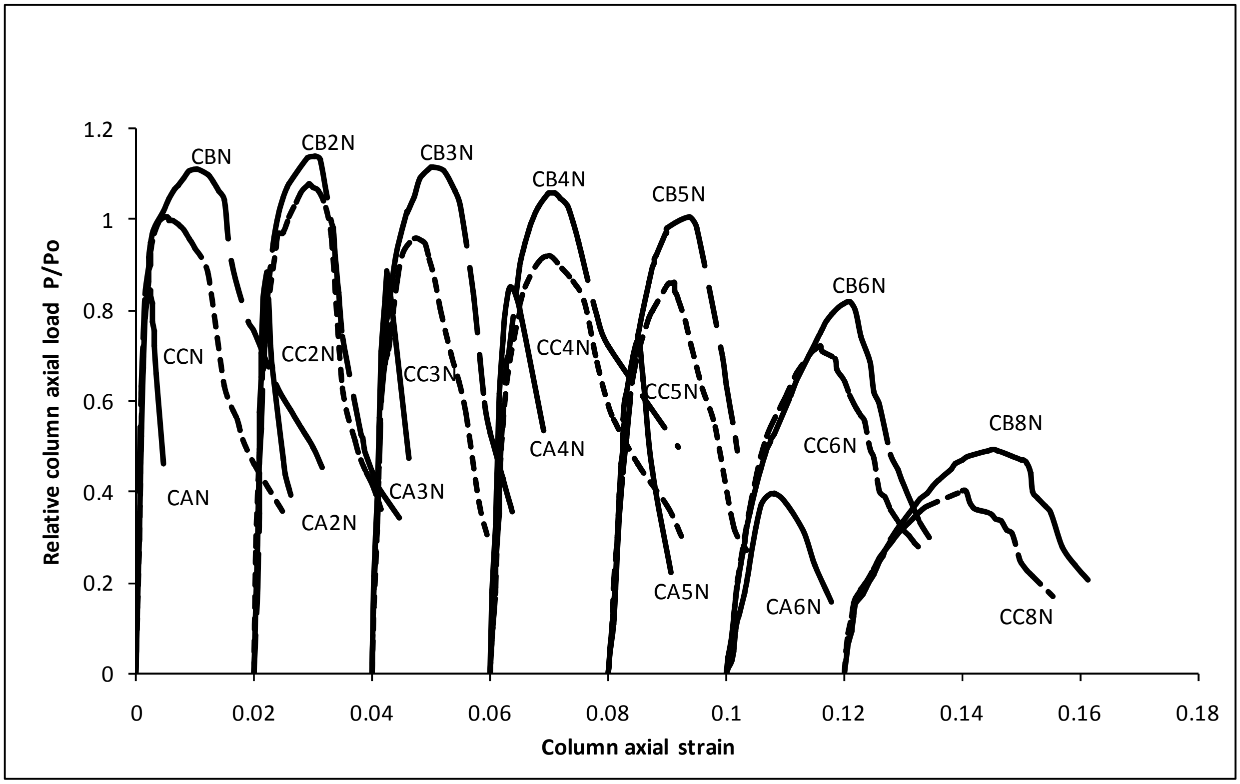

Figure 2 Effect of spacing (volumetric ratio) of lateral confining hoop reinforcement

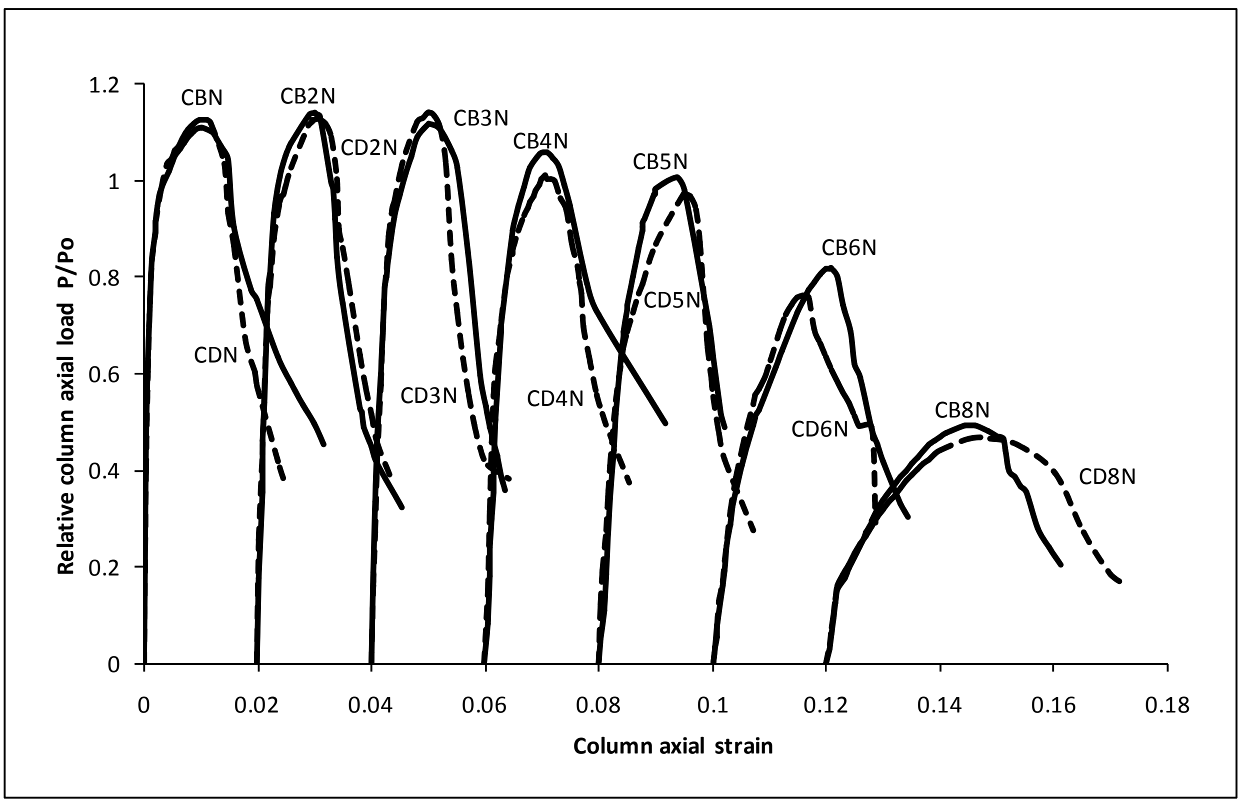

Figure 3 Effect of yield strength of lateral confining hoop reinforcement

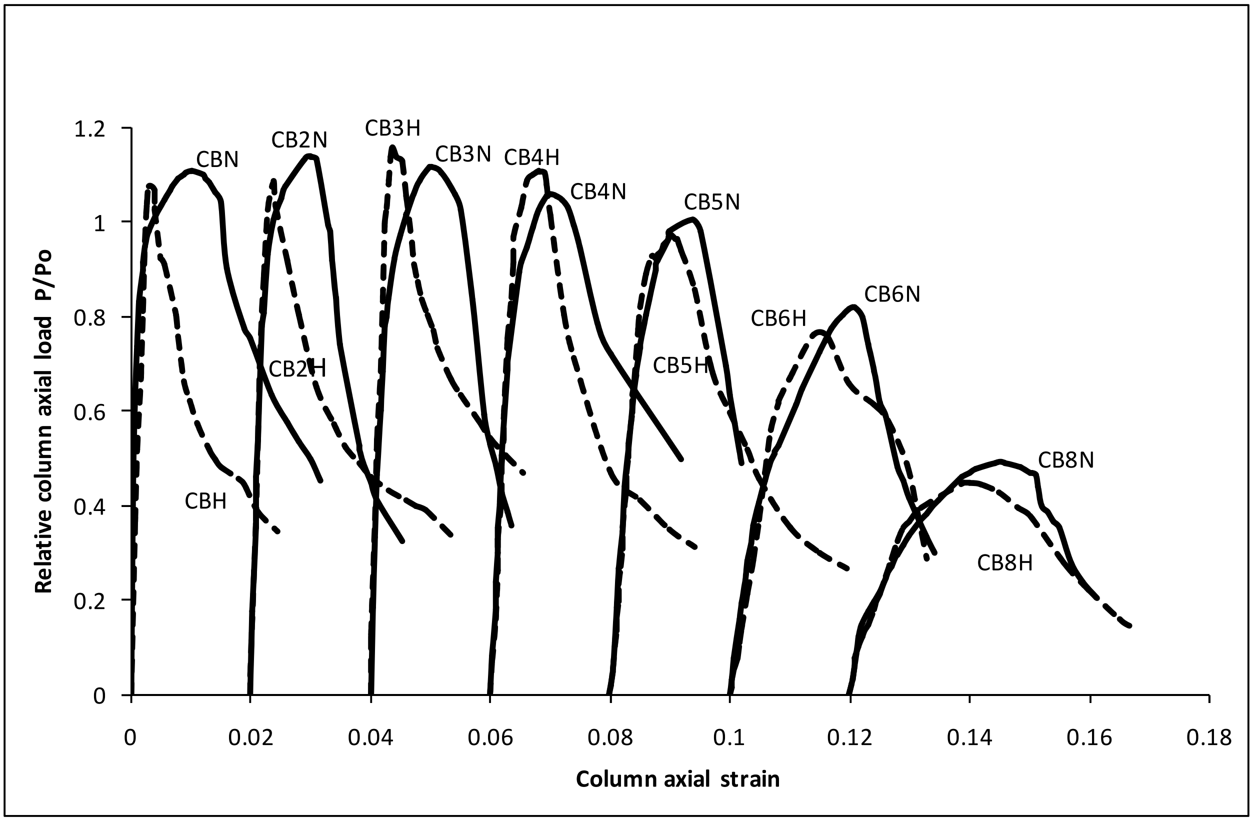

Figure 4 Effect of concrete strength

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