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Fibre reinforcing concrete columns

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Abstract

This paper explores the effects of adding steel fibres to high-strength reinforced concrete columns and in particular only to the cover of the columns. An experimental program was conducted where seven circular reinforced concrete columns were tested with varying fibre content – one contained no fibres, three contained fibres throughout the cross-section and three contained fibres only in the outer concrete. The other column properties were kept the same for all the seven columns. All seven columns were tested by the application of a concentric, axial compression force. It was found that although only minor improvements were noticeable for a fibre content of 1%, the addition of 1.5% and 2% steel fibres increased the load at which cover spalling took place. It was also found that the columns containing both FHSC (fibrous high strength concrete) in the outer concrete and HSC in the core exhibited higher levels of ductility than the columns containing FHSC throughout the entire cross-section.

Keywords: Fibre, RC Columns, Ductility

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1.0 Introduction

Over the past twenty years, research of and improvements to concrete mix design have resulted in an increase of three to four-fold in available concrete strengths. With respect to columns, a larger compressive strength means a smaller cross-sectional area required, resulting in better utilisation of available space and materials. These higher-strength columns, however, have been shown to contain weaknesses. As the compressive strength increases so too does the brittleness and while increasing the amount of lateral reinforcement reduces this brittleness, it also increases the column's susceptibility to early cover spalling. In order for high-strength concrete (HSC) columns to be effective and superior to normal strength concrete columns, these weaknesses need to be overcome.

Early spalling of the cover concrete arises due to the confinement effect provided by the reinforcement. When a column is axially compressed, material is pushed outwards resulting in increased cross-section dimensions. The core concrete is confined by reinforcement, but the unconfined cover concrete outside the reinforcement continues to be pushed outwards, placing it in tension, and forcing it to separate from the core concrete. Once this cover has spalled the column has less cross-sectional area and hence has a reduced load-carrying capacity. Research has shown that the addition of fibres to the concrete helps to arrest the onset of early cover spalling [1,2,3]. These studies have also shown that ductility of the HSC columns can be improved through the addition of fibres.

This study of fibre reinforced concrete columns investigates a new method of column construction which results in fibrous high strength concrete (FHSC) being located only in the cover concrete while plain HSC is located in the remaining core concrete. It is proposed that this new type of column will perform in a superior manner to columns which contain FHSC throughout the entire cross-section as problems such as the movement of fibres towards the centre of the column, away from the cover, during vibration will be overcome. This new type of column also provides a more efficient use of materials as fibres are located in the cover to prevent early cover spalling, while the remainder of the column contains only plain HSC so the integrity and density of the core remain unaffected.

2.0 Fibres Within Concrete Columns

Adepegba & Regan (1981) [4] carried out tests on steel fibre reinforced concrete columns. The columns were made up of concrete strengths in the range 32 to 44 MPa, tested in axial compression and 2.4 m in height. The central 0.8 m had a square cross section of 200 mm whilst outside this region the section was enlarged gradually and reached a cross-section size of 200 mm x 500 mm at both ends. The main variable in the experiment was the fibre content of each column which ranged from 0% to 2%. It was found that the addition of steel fibres at any of the tested fibre contents did not increase the ultimate load of the column. It was noted, however, that the experiments did not investigate the post-failure behaviour improvements gained by the addition of steel fibres.

Ganesan & Murthy (1990) [5] studied the effect of varying the amount of lateral reinforcement on steel fibre-reinforced and non-fibre-reinforced concrete columns. The columns were 200 mm square and 1000 mm high, consisted of concrete in the range 20 MPa to 30 MPa, and were subject to a concentric axial load. The fibrous columns contained 1.5% (by volume) steel fibres. It was found that as the amount of lateral reinforcement increased, larger strength increases were obtained from the fibrous columns compared with the non-fibrous columns and the addition of steel fibres resulted in better strength and ductility.

Campione (2002) [6] presented a mathematical model which determined the stress-strain relationship for fibre reinforced concrete columns. The analytical expressions allowed the determination of the maximum strength and strain capacity of circular or square, high-strength or normal-strength, fibrous or non-fibrous concrete columns. Explanations were also given regarding the region of the column cross section which could be considered effectively confined by the reinforcement. It was also noted that the model was verified through experimental testing.

Sarker (2001) [3] studied the effect of adding synthetic fibres to high strength concrete columns. The fibres used were 3M Polyolefin (25/38) fibres which were added to 175 mm square columns with a concrete strength of 62 MPa. The columns were subjected to single and double curvature bending and it was found that the inclusion of fibres increased the ductility of the columns and arrested the early spalling of the cover. It was also proposed that longer fibres at a higher percentage content would produce better column performance.

3.0 Experimental Programme

From a variety of steel fibres present on the market, FIBRESTEEL® 184EE supplied by Bosfa was chosen for its anchorage and availability in small quantities and short lengths of 18 mm. The short length of steel fibres were needed to ensure that they could orient in a random pattern in the 22.5 mm space (cover) between the helix and the external formwork. The fibre was an 'Enlarged end fibre' made from cold rolled low carbon high strength sheet with well defined enlarged 'dumbbell' ends. The fibres used in this experiment had dimensions 18 x 0.6 x 0.3 mm, an aspect ratio of 38 and contained approximately 38400 fibres per kilogram.

Seven columns (925 height and 205 mm diameter) were cast and tested. Three percentages of fibres were used, 1%, 1.5% and 2% by volume. One column denoted R was the reference column where no fibres were used. Columns 1F, 1.5F and 2F had fibres added to the entire cross section with the percentages %1, 1.5% and 2% by volume, respectively. Similarly Columns 1C, 1.5C and 2C had fibres in their covers only with the percentages by volume 1%, 1.5% and 2%, respectively.

The fibre content and location for each column are shown in Table 1 and details and diagrams of the columns are presented in the following sections. The reinforcement of all columns consisted of 6N12 bars (12 mm deformed bars with 500 MPa nominal tensile strength) and R10 (10 mm plain bars with 250 MPa tensile strength) at 50 mm helices. Three N12 samples were tested in tension and revealed an average tensile strength of 557.4 MPa. Similarly three samples of the R10 bars were tested in order to determine their tensile strength. These tests showed that the average tensile strength of the tested R10 bars was 424.2 MPa.

3.1 Casting Procedure

The high strength concrete (HSC) was placed into two wheelbarrows so casting of Column R could begin. In order to create the fibrous high strength concrete (FHSC) the HSC was placed directly from the concrete chute into the concrete mixer. The fibres were added evenly to the concrete by hand as it descended the chute. The mixing drum was filled to the correct predetermined height so that a total volume of $0.125m^3$ was present. The mixer was switched on for 1.5 to 2 minutes until the fibres were dispersed evenly throughout the mix. The quantities of fibres added to the concrete were 7.46 kg for the 1% mix, 11.18 kg for the 1.5% mix and 14.92 kg for the 2% mix.

Table 1: Column Number by Fibre Content and Location.

Column	Fibre content (%)	Fibre location
R	0	-
1C	1	Cover only
1F	1	Entire Cross section
1.5C	1.5	Cover only
1.5F	1.5	Entire Cross section
2C	2	Cover only
2F	2	Entire Cross section

In order to determine the properties of both the HSC and FHSC, sample cylinders and beams were cast. For each of the four batches (0%, 1%, 1.5% and 2%) six small cylinders were made for compressive tests (three for 7-day strength and three for 28-day strength), three large cylinders were made for indirect tensile tests at 28 days, and three beams were made for flexural tests at 28 days. Each of the samples was cleaned and lubricated prior to casting.

For the columns which had uniform cross-sections, pouring and vibrating were conducted in three stages. The concrete was scooped into the columns for 1/3 of the height before being vibrated with an electric vibrator. The middle and top 1/3 were poured in the same manner and finally the surface was finished with a wet trowel.

The columns which were to have fibres only located in the cover (Columns 1C, 1.5C and 2C) were constructed with a Perspex sheet fabricated into a tube with 163 mm diameter located outside, but in contact with, the steel reinforcement, see Figure 1. The HSC was placed in the centre of the Perspex tube whilst the FHSC was placed around the outside of the tube in the gap between the Perspex and the formwork. Although the gap in which to place the FHSC was smaller than the gap for the PVC pipe, the concrete dropped to the bottom of the column with ease as there were no obstructions and pushing with 6 mm bars was not required. The column was filled in this way until the HSC inside the Perspex tube and the FHSC outside the Perspex tube were equal and at roughly 1/3 the column height (see Figure 2a). The Perspex tube was then lifted until it was roughly 50 mm inside the concrete. The vibrator was then placed down the centre of the Perspex tube to vibrate the core whilst the cover was rodded with 6 mm bars and tapped with a mallet. This process was then repeated for the middle and upper 1/3 of the column (see Figures 2c and 2d), although the upper 1/3 was not vibrated immediately. The final 40 mm was not placed straight away (see Figure 2e) in order to allow for the strain gauge wires to be threaded through the hole in the side of the formwork. Once this was carried out, the Perspex tube was replaced in the column, the final 40 mm was cast and vibration of the top 1/3 took place. Finally, the top surface was finished with a wet trowel. At the conclusion of casting, all specimens were covered with wet Hessian and plastic sheets to prevent moisture loss. The cylinder and beam samples were stripped and placed in a curing tank whilst the columns were stripped after 7 days and placed under wet Hessian and covered with plastic sheets.

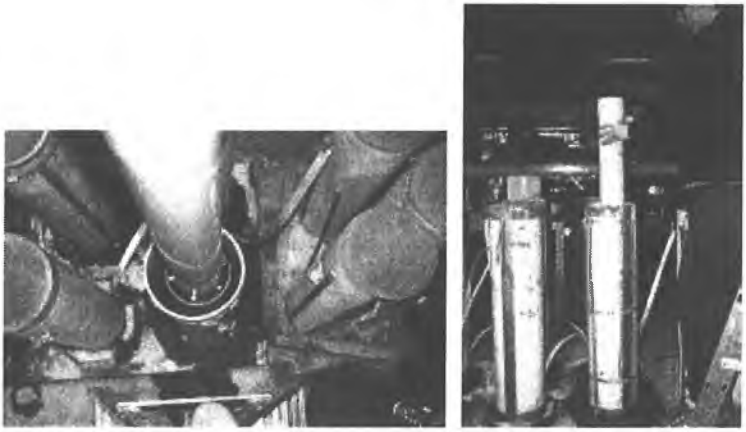


Figure 1. Columns 1C, 1.5C and 2C.

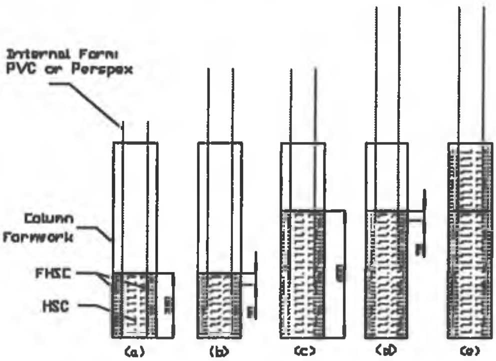


Figure 2. Column casting procedure.

4.0 Column Testing

In order to test the columns the following procedure was employed. The columns were removed from the curing environment and each was weighed and measured (the recorded values were an average of two height measurements and an average of two diameter measurements at mid height). Each column was capped at the base with high-strength plaster and once the plaster had set, the column was inverted, lifted into the testing machine and capped at the opposite end, under a pre-load of around 12 kN provided by the testing machine. While the high-strength plaster was allowed to set a galvanised safety cage was placed around the column to minimise any potential damage caused by flying debris. Once sufficient time was allowed for the plaster to set, testing of the column took place. Loading of the column was controlled by displacement, rather than by load, so that post-peak performance could be monitored. The rate of loading was varied depending on the response of the column but a value of 0.3 mm/min was found to be ideal for speed and accuracy. During testing, deflection readings were taken every 20 kN so that load-deflection curves could be created.

5.0 Results

Testing the concrete for its compressive strength revealed that it had a value of 75.5 MPa. This value increased to 86 MPa, 85 MPa and 90 MPa for the fibre mixes of 1, 1.5 and 2, respectively. Table 2 shows yield load, corresponding displacement, ultimate load and the maximum displacement of the seven tested columns.

Table 2: Results of testing the columns

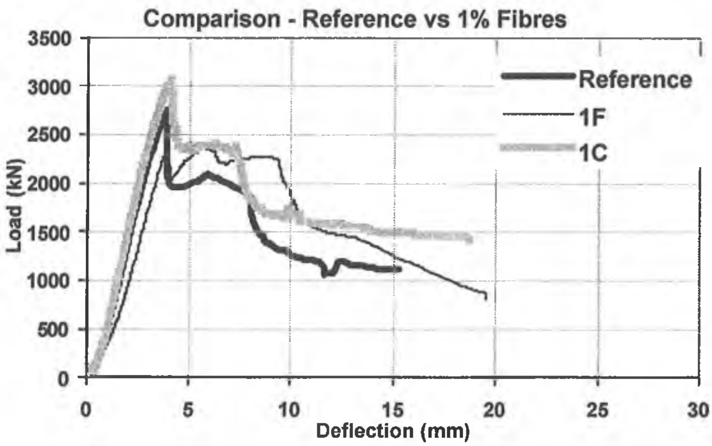
Col	Yield Load (kN)	Displ. at Yield Load (mm)	Ult. Load (kN)	Displ. at Ult. Load (mm)	Max Displ. (mm)
R	2740	3.91	2080	5.95	15.26
1C	2380	3.96	2361	5.75	19.53
1F	3055	4.12	2384	6.42	18.71
1.5C	2904	3.54	2469	6.32	16.25
1.5F	2800	4.05	1623	8.37	19.53
2C	3195	4.65	2551	7.06	24.08
2F	2959	4.88	3025	5.70	16.66

Table 3: Calculated ductility of the tested columns

Col	Area Under the Load Deflection Curve (kNm)	Relative to Column R
R	22820	1
1C	30896	1.37
1F	32510	1.44
1.5C	25332	1.12
1.5F	29160	1.29
2C	35463	1.57
2F	28664	1.27

Ductility of each of the tested columns was calculated and are shown in Table 3. These ductilities were calculated as the area under the load deflection curve of each of the columns. The ductility of Column R was used as a reference value for the remaining columns.

The effect of adding steel fibres into different locations of the column can be observed by comparing the two fibre configurations at 1% by volume to the reference column as seen in Figure 3. It can be seen in all three columns in Figure 3 that the ultimate load occurs at the yield section of the load – deflection curve where the spalling of the cover has a detrimental effect on its load carrying capacity. Column 1F exhibited a much lower ultimate failure load due to the helix breaking prematurely at the weld. However, when comparing the columns at 6mm displacement and onwards, both columns consisting FRHSC held a much higher load than the reference column. It is also noted that column 1F held a substantial axial load up until 9.4 mm before its load capacity declined, Column 1C only held the load to 7.8mm before the load began to drop, hinting that Column 1F possessed more ductility. The deflection at which the cover spalling took place in all columns was approximately 4



mm.

Figure 3. Reference versus 1% FHSC Configurations

Comparing the different fibre configurations for the 1.5% FRHSC, it can be seen in Figure 4, that both locations of fibres give a slightly higher compressive strength in comparison to the Reference column. Column 1.5C displays a lag in the response to the deflection as it is loaded from 0 mm deflection to 1.5 mm deflection as shown in Figure 4. This lag is possibly due to the settlement of the high strength plaster capping and can be seen in all testing of the columns. It was noted that Columns 1.5C and 1.5F did not experience rapid loss of cover where the cover very slowly cracked. It was not until deflections higher than 8 mm that the cover detached from the core. It can be noted from Figure 4 that column 1.5F held the axial load up until 7.0 mm before its load carrying capacity deteriorated, however, the load carrying capacity of Column 1.5C continually decline in a controlled manner after the yield load. The helix in Column 1.5F failed at a deflection of approximately 16.1 mm.

Again by comparing the different fibre configurations, it can be seen in Figure 5, that both locations of 2% FRHSC give a much higher compressive strength in comparison to the plain HSC Reference column. The strength increase in Column 2F is approximately 17% higher than Column R. Column 2C has a higher compressive strength than Column R in the order of 8% and experienced a second maximum higher than the one present at the yield load. Column 2C also displays a lag in the response to the deflection as it is loaded. It was noted that Columns 2C and 2F did not experience rapid loss of cover where the cover very slowly cracked away only in the centre of the column. It was not until deflections higher than 7.5 mm and 10 mm for columns 2C and 2F, respectively, that the cover in the centre region of the column detached from the core. Figure 5 demonstrates that Column 2C continually deteriorated after the yield load in a controlled manner. Column 2F incrementally deteriorated post yield load and held the axial load of approximately 2500 kN up until 9.5 mm deflection before its load carrying capacity steadily decreased.

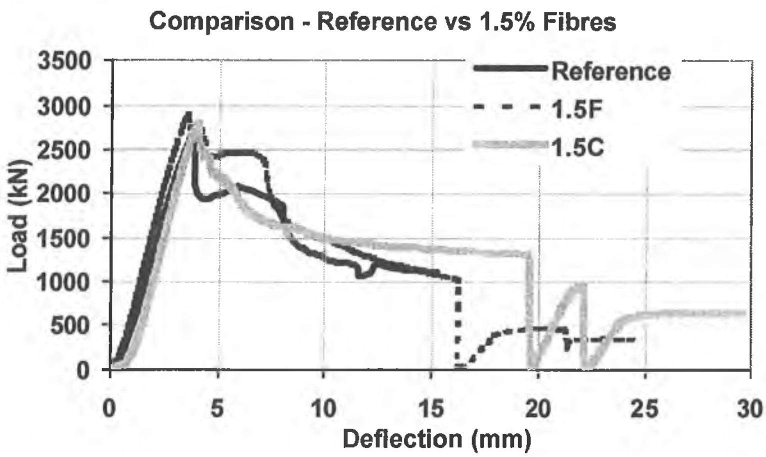


Figure 4 Reference versus 1.5% FRHSC Configurations

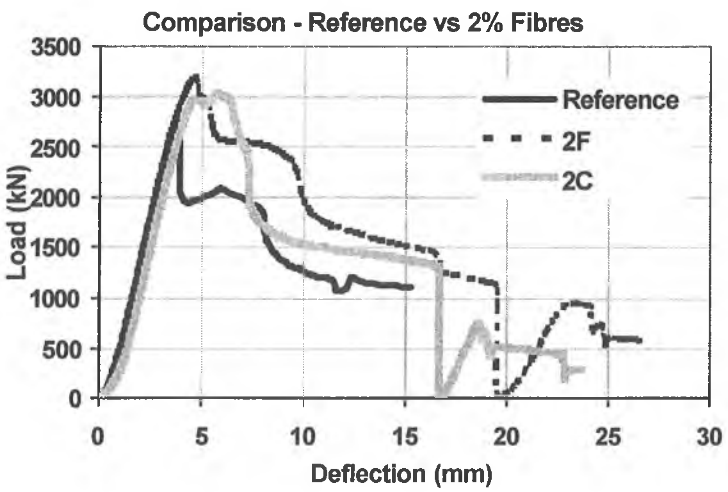


Figure 5 Reference versus. 2% FRHSC Configurations

A comparison of the columns with FHSC in the cover only to the columns with FHSC in the full cross section can be seen in Figures 6 and 7. The columns with FHSC in the cover only and plain HSC in the core experience the first yielding failure around the same region of approximately 2900 kN. The load – deflection curves are unpredictable in that the failure pattern is somewhat random between 4 mm and 8 mm deflection where no similarities can be seen. After 8 mm deflection where the covers of all three columns have begun to fail, they follow a very similar load - deflection line decreasing from 1600 kN to 1400 kN at similar displacements. After 8 mm deflection and the covers have completely failed, the only structural elements left are the plain confined HSC core, this being the reason that all three columns follow the same load deflection curve.

The columns with a full cross section of FHSC experience a higher first yielding failure and ultimate load proportional to the content of steel fibre, the greater the amount of steel fibre, the higher the load at failure. It can also be seen that the columns with a full cross section of FHSC possess a unique failure pattern where the load incrementally drops down in steps. The failure pattern of Column 1F is slightly different due to the helix failing prematurely, resulting in a strange pattern.

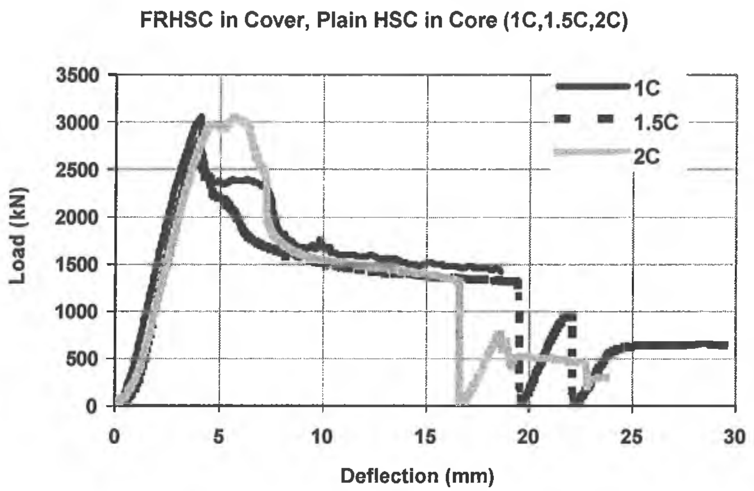


Figure 6 Fibres in Cover Only

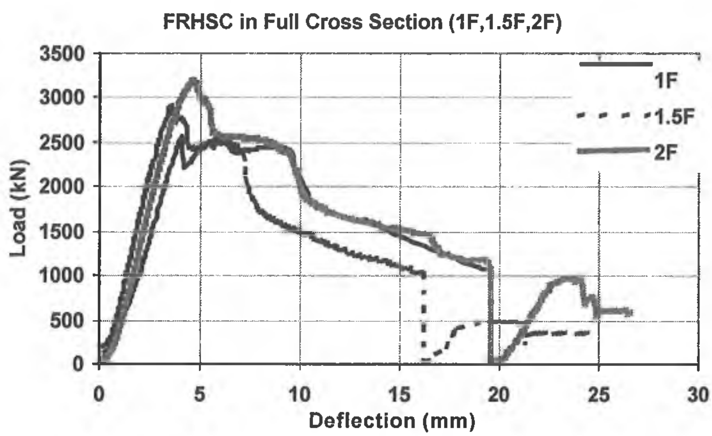


Figure 7 Fibres in Full Cross Section

6.0 Conclusions

This study of fibre reinforced concrete columns involved the testing of seven high strength concrete columns with varying fibre contents and fibre locations. The main aim of this study was to determine whether placing fibrous concrete only in the outer cover, would be sufficient to maintain, or enhance, the desirable properties of high strength concrete columns. The following is a list of the conclusions drawn from this study.

- The addition of enlarged end steel fibres into the matrix of the high strength concrete significantly enhances the compressive strength, flexural strength and tensile strength of the concrete.
- The addition of 1.5% to 2% steel fibres by volume into the matrix of the high strength concrete produced the optimal performance in terms of compressive, flexural and tensile strength of the concrete.
- The higher the amounts of steel fibres present in the concrete, the lower the workability and the higher chance of variations in concrete strength and cavities.
- The addition of steel fibres into the concrete of the column increases the load and strain at which cover spalling takes place regardless of the content and location.
- The addition of enlarged end steel fibres into the cover of the column considerably increases its ductility.
- The columns consisting of 2% FHSC exhibited minimal cover spalling and a much higher ultimate strength. Also, columns consisting 2% FHSC in the cover only are able to resist higher amounts of lateral expansion due to axial loading.

7.0 References

1.Foster, S.J. 2001. "On Behaviour of High Strength Concrete Columns: Cover Spalling, Steel Fibers, and Ductility". *ACI Structural J.* 98(4): 583-589.
2.Foster, S.J. & Attard, M.M. 2001. "Strength and ductility of fiber-reinforced high-strength concrete columns". *J. of Structural Eng.* 127(1): 28-34.
3.Sarker, P.K. (2001). "Fibre Reinforced High Strength Concrete Columns, in conference proceedings: Adding Value Through Innovation". *Concrete Inst of Australia, Perth, September*, pp. 37-42.
4.Adepegba, D. & Regan, P. 1981. "Performance of steel fibre reinforced concrete in axially loaded short columns", *Int J of Cement Composites and Lightweight Concrete* (Harlow). 3(4): 255-259.
5.Ganesan, N. & Murthy, J. 1990. "Strength and behavior of confined steel fiber reinforced concrete columns". *ACI Materials J.* 87(2): 221-227.
6.Campione, G. 2002. "The effects of fibers on the confinement models for concrete columns. *Canadian J. of Civil Eng.* 29(5): 742-750.

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