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RESEARCH PAPERS

A study of tie confined fiber reinforced concrete under axial compression

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ABSTRACT

An experimental investigation on the behavior of tie confined fiber reinforced concrete (CFRC) has been carried out. The compressive strength of FRC depends on both the volume fraction (V_f) and aspect ratio (l / d) of the fibers, a common parameter, which combines these two effects, is used in the form of reinforcing index, RI (product of weight fraction and aspect ratio). The primary variables considered in this study are: (i) Five values of confinement index (C_i) (*Viz.*, 0.0, 0.08, 0.19, 0.3 and 0.56) and (ii) Five values of volume fraction of fibers, 0%, 0.3%, 0.6%, 0.9% and 1.2% (*Viz.*, reinforcing index values 0, 0.74, 1.47, 2.21 and 2.95 respectively). A total of 90 specimens of concrete prisms of size 150 mm X 150 mm X 300 mm were cast and tested under strain control rate of loading. The test results reveal that the ductile characteristics of FRC are further improved due to tie confinement and also improved the peak stress, strain at peak stress and ductility of concrete. The improvement is in proportion to the RI of the fiber for a given confinement index (C_i) of the lateral reinforcement.

1. INTRODUCTION

Seismic resistant design of structures demands high ductility. The ductility of concrete is being improved at present by confining it in steel binders, as ties in compression members and as stirrups in beams. It has been reported [4] that a concrete strain of 0.01 is sufficient to give full redistribution of moments, which results in the use of procedures of plastic analysis for analysis of concrete structures also. However, it can be seen that the higher the degree of indeterminacy of the structure the more will be the concrete strain at failure and consequently the rotation capacity required at the first plastic hinge which will form in the structure. The critical sections in statically indeterminate structures at which first hinge forms are incidentally also the sections having maximum shear force. The stirrup reinforcement, which is provided, has to take care of shear at that section and simultaneously provide confinement. It has been established by previous researchers [6] that only the stirrup reinforcement provided beyond what is required for resisting shear failure will only provide confinement. Hence with practical minimum spacing that can be provided at the critical sections there is limitation to the quantity of the confinement, which can be provided by the stirrups. Moreover, confinement of a column using a sophisticated arrangement of closely spaced stirrups not only interrupts the continuity and creates plane of weakness between the core and the concrete cover, but it also adds to the problem of steel congestion. Thus it may not be possible to sufficiently confine the structure by providing the laterals alone. Hence it would be useful if a supplementary or indirect confinement, in addition to the laterals, can be provided at the critical sections or a better alternative to confinement can be devised. Many investigations have revealed that the inclusion of steel fibers to concrete enhances several of its engineering properties such as tensile strength, ductility and fracture toughness. When short, randomly distributed fibers are added to concrete, the fibers improve the integrity of the material, in addition to the tensile properties of concrete. Recently a few investigations [1, 5] were made to confine the concrete using fibers in addition to laterals. The conclusions drawn were qualitative in nature but highlighted that fibers can give some confinement. Such type of concrete can be termed as CONFINED FIBER REINFORCED CONCRETE (CFRC). The present investigation is an attempt to investigate the stress-strain characteristics of CFRC and to develop prediction equation for the same.

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2. EXPERIMENTAL PROGRAMME

2.1 Scheme of experimental work

The experimental programme was designed to study the behaviour of confined steel fiber reinforced concrete under axial compression by testing prisms of size 150 mm × 150 mm × 300 mm. The variables in the study are reinforcing index (RI) of the steel fiber, which controls the behaviour of the FRC and confinement index (C_i) of lateral steel reinforcement. The reinforcing index is the product of weight fraction of steel and the aspect ratio of the fiber. The weight fraction (w_f) is the ratio of weight of steel fiber and weight of concrete.

The programme consisted in casting and testing 90 prisms, which were cast in two groups. The first group of prisms casted with M_{20} grade of concrete and the second one with M_{25} grade of concrete. Each group was cast in five batches. The prisms in each batch were divided into three sets. In each set three identical specimens were cast and tested and the average behaviour was taken to represent the behaviour for that set of three specimens. Hence in each batch the total number of prisms amounted to nine.

Out of three sets of group I category (A, B and C), in each batch, the first set consisted of two lateral ties (C_i = 0.0), second set consisted of five lateral ties (C_i = 0.30) and third set consisted of seven lateral ties (C_i = 0.56). And in the three sets of group II category (D, E and F), in each batch, the first set consisted of two lateral ties (C_i = 0.0), second set consisted of four lateral ties (C_i = 0.08) and third set consisted of seven lateral ties (C_i = 0.19).

Each group, out of five batches, the first batch with 0% (RI = 0.00) fiber, second batch with 0.30% (RI = 0.74) fiber, third batch with 0.60% (RI = 1.48) fiber, fourth batch with 0.90% (RI = 2.22) fiber and fifth batch with 1.20% (RI = 2.96) fiber were cast. Proper designation was given for each specimen. The details of prisms were given in Table 1.

Table 1 – Details of prisms

Sl. No.	Specimen designation	Longitudinal reinforcement		Lateral reinforcement				Steel fiber			
		Dia (mm)	f_y (MPa)	Dia (mm)	f_y (MPa)	Spacing (mm)	C_i	V_f %	RI	f'_c	ε_c x10^{-6}
1	A1	3.92	295	7.18	350	290	0.0	0	0	23.3	2100
2	B1	3.92	295	7.18	350	70	0.30	0	0	23.3	2100
3	C1	3.92	295	7.18	350	50	0.56	0	0	23.3	2100
4	A2	3.92	295	7.18	350	290	0.0	0.3	0.74	22.9	2050
5	B2	3.92	295	7.18	350	70	0.30	0.3	0.74	22.9	2050
6	C2	3.92	295	7.18	350	50	0.56	0.3	0.74	22.9	2050
7	A3	3.92	295	7.18	350	290	0.0	0.6	1.48	23.1	2075
8	B3	3.92	295	7.18	350	70	0.30	0.6	1.48	23.1	2075
9	C3	3.92	295	7.18	350	50	0.56	0.6	1.48	23.1	2075
10	A4	3.92	295	7.18	350	290	0.0	0.9	2.22	23.4	2125
11	B4	3.92	295	7.18	350	70	0.30	0.9	2.22	23.4	2125
12	C4	3.92	295	7.18	350	50	0.56	0.9	2.22	23.4	2125
13	A5	3.92	295	7.18	350	290	0.0	1.2	2.96	23.0	2000
14	B5	3.92	295	7.18	350	70	0.30	1.2	2.96	23.0	2000
15	C5	3.92	295	7.18	350	50	0.56	1.2	2.96	23.0	2000
16	D1	3.00	350	7.00	448	290	0.00	0.0	0.0	36.8	2020
17	E1	3.00	350	7.00	448	88	0.08	0.0	0.0	36.8	2020
18	F1	3.00	350	7.00	448	57	0.19	0.0	0.0	36.8	2020
19	D2	3.00	350	7.00	448	290	0.00	0.3	0.74	37.0	1980
20	E2	3.00	350	7.00	448	88	0.08	0.3	0.74	37.0	1980
21	F2	3.00	350	7.00	448	57	0.19	0.3	0.74	37.0	1980
22	D3	3.00	350	7.00	448	290	0.00	0.6	1.48	37.2	2010
23	E3	3.00	350	7.00	448	88	0.08	0.6	1.48	37.2	2010
24	F3	3.00	350	7.00	448	57	0.19	0.6	1.48	37.2	2010
25	D4	3.00	350	7.00	448	290	0.00	0.9	2.22	36.5	1970
26	E4	3.00	350	7.00	448	88	0.08	0.9	2.22	36.5	1970
27	F4	3.00	350	7.00	448	57	0.19	0.9	2.22	36.5	1970
28	D5	3.00	350	7.00	448	290	0.00	1.2	2.96	36.9	1990
29	E5	3.00	350	7.00	448	88	0.08	1.2	2.96	36.9	1990
30	F5	3.00	350	7.00	448	57	0.19	1.2	2.96	36.9	1990

2.2 Materials used

The 0.5 mm diameter steel fibers with aspect ratio of 75 were used in all the specimens. The 3 mm G.I wire used as longitudinal reinforcement in the prisms of all batches. The ties used were mild steel. The cement used was 43 grade conforming to IS: 8112-1981. Machine crushed hard granite chips passing through 12.5 mm IS sieve and retained on 4.75 mm IS sieve was used as coarse aggregate throughout the work. River sand procured locally and passing through 2.36 mm IS sieve was used. M_{20} and M_{25} Grade of concretes were used for group I and group II respectively throughout the work.

Fig. 1(a) – Reinforcement details in prisms.

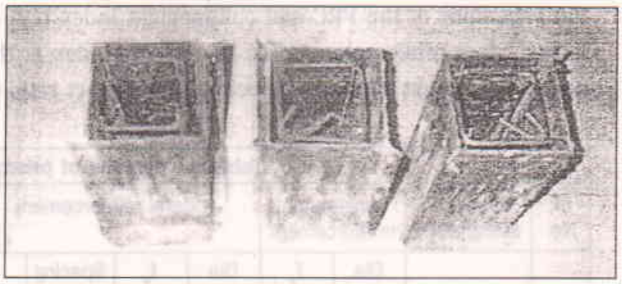

Fig. 1(b) – Moulds used for casting the prisms.

2.3 Preparation of specimen

The ties are tied to the four longitudinal bars at the required pitch in such a manner that the hooks were distributed evenly on all the four corners. Fig. 1(a) shows the reinforcement details of the specimen. Fig. 1(b) shows the moulds used for casting the prisms.

2.4 Casting of specimen

The prepared cage of reinforcement was kept in the moulds carefully. The prisms were cast in the vertical position. First mould was filled to about the half height and then a needle vibrator was used to compact the core concrete. The mould was filled in three layers in the same technique. The top face of the prism specimen was capped with a rich cement paste. The specimen was decoupled 24 hours after casting and cured for 28 days in the curing pond.

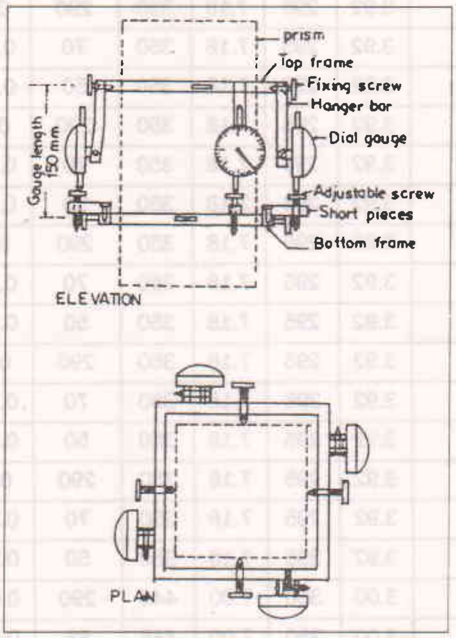

Fig. 2(a) – Details of compressometer.

2.5 Testing

The cured specimens were capped with plaster of Paris before testing to provide a smooth loading surface. A Tinius-Olsen testing machine of 1810 kN capacity was used for testing the prisms under axial compression. From the studies of previous investigators who worked on concrete confined with ties, it was observed that the cover concrete started spalling off at about 90 percent of the ultimate load. Along the concrete, the resistance strain gauges and demec points fixed to the concrete surface usually came off. Also the compressometer designed to measure the strains in standard concrete cylinders could not be fitted to the square prisms.

To overcome the above-mentioned difficulties, compressometer suitable for prisms, which were fabricated by the earlier investigators on confined concrete were adopted. Each compressometer consisted of two square frames, a top frame and a bottom frame made of 12 mm square mild steel bars. Each frame was attached to the concrete specimen by two diametrically opposite pairs of screws at four points. The two frames were attached to the specimen symmetrically at the required gauge length, *i.e.*, 150 mm apart. Two pairs of diametrically opposite dial gauges with a minimum count of 0.002 mm were attached to vertical hanger bars fixed to the top frame. The movable spindles of dial gauges rested on the plane circular heads of the adjustable screws, which were positioned in mild steel plates projecting horizontally from the bottom frame. The frames were attached to the specimen by means of screws, which would fit snugly to the concrete. Fig. 2(a) shows the details of the compressometer attached to the specimen. Fig. 2(b) shows the photograph of the same arrangement.

Fig. 2(b) – Compressometer attached to the prism.

The capped specimen with the compressometer attached was placed on the movable cross head of the testing machine and tested under strain rate control. The deformations were noted and strains were calculated. Fig. 3 shows the test arrangement. The test was continued until the load dropped to about 75 to 80 percent of the ultimate load in the post-ultimate region for both confined and unconfined concrete specimens.

3. INTERPRETATION AND DISCUSSION OF TEST RESULTS

3.1 Behaviour of test specimens under load

3.1.1 General

The load increased rapidly in the initial stages up to about 75% of peak load and there after increased at a slower rate until the peak load was reached. Tests were continued until the peak load dropped to about 0.75 to 0.80 times the peak load. Beyond the peak load, the strains increased at a rapid rate and were accompanied by a decrease in the load carrying capacity of the specimen.

3.1.2 Reinforced concrete (RC) specimens

In the case of RC specimens, vertical cracks appeared in the cover region at about half of the peak load. As the load increased, the number of cracks increased and the width of cracks widened. The spalling of concrete cover was noticed before the peak load (*i.e.*, at about 90% of peak load) was reached. But it was severe after passing the peak load.

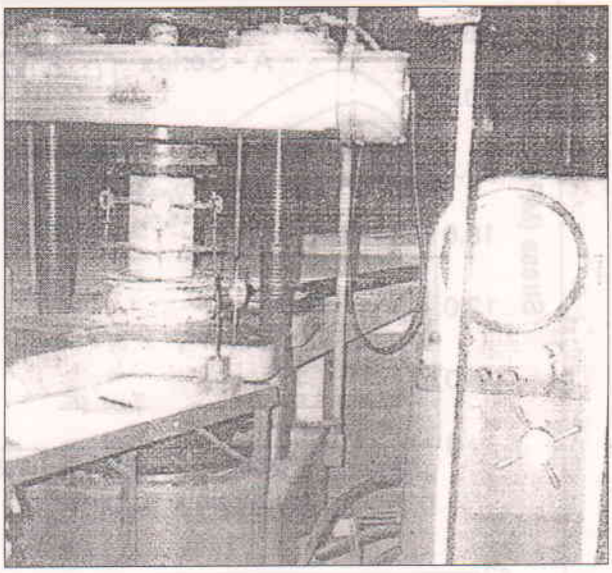

Fig. 3 – Testing of the prism on UTM.

3.1.3 CFRC specimens

In the case of CFRC specimens, fine vertical cracks appeared on the surface of the specimen at about 80% to 85% of the peak load. With increase of load, the number of cracks increased at a reduced rate compared to that of RC specimens. The behaviour of all the CFRC specimens up to 75% of the peak load of the confined RC specimens was about the same. Beyond the peak load, the fine vertical cracks were widened. The extent of the cracking and rate of decrease in load after the peak (in the descending portion of stress- strain curve) dependence up on the reinforcing index (RI) of the fiber, if the tie confinement indicated by confinement index (C_i) is the same. The higher the RI ,the lower is the rate of decrease in load and the extent of spalling. This may be due the improvement of the internal crack arresting mechanism, dimensional stability as well as integrity of the material caused by the presence of large volume fraction of the fiber present in the concrete. The maximum stress and strain at peak load and the strain at 85% of peak load in the descending portion of the stress-strain curve increased as a RI increased with the same C_i.

3.2 Experimental stress-strain curve

From the observed data, for a given specimen, the longitudinal deformations were calculated from the average readings of the four dial gauges of the compressometer. As there was no severe spalling in CFRC specimens until the load dropped by about 20 to 25% of ultimate load, the specimens were treated as dimensionally stable and hence the gross cross sectional area was used in cal-

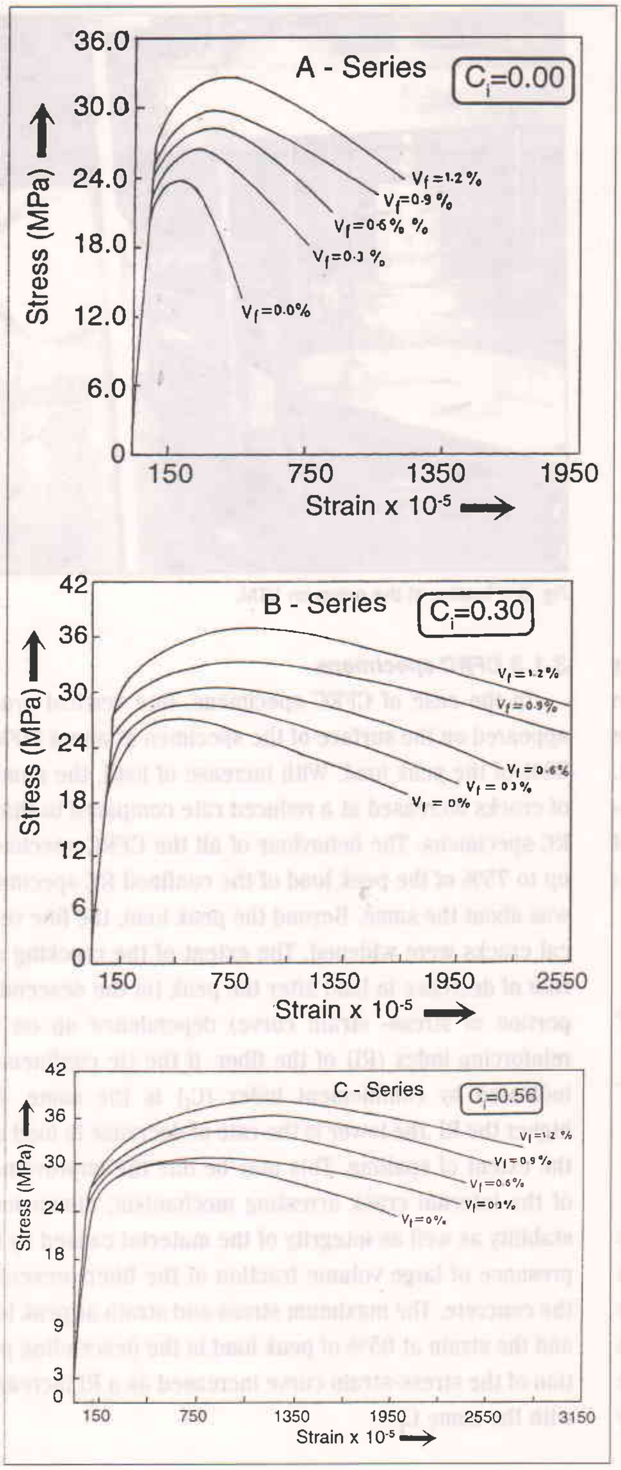

Fig. 4 – Stress-strain curves of 150 x 150 x 300 mm tie confined FRC prisms.

culating the stress values. Stress-strain curves were drawn for three companion specimens of a set with the same origin, and average curve was taken to represent the set. Such average curves for all the sets of one typical batch with a common origin are shown in Fig. 4.

3.3 Effect of reinforcing index (RI) on

3.3.1 Ultimate strength

The ultimate strength of concrete increased with an increase in reinforcing index (RI) for the same level of confinement. To quantify the effect of indirect confinement due to steel fiber on ultimate strength, the effect of tie confinement is separated using the following equation:

$$f_c A_g = K_1 f_c' A_g \quad (1)$$

= The load taken by the tie confined concrete

where, $K_1 = 1.0 + 0.55\, C_i$ (2)

$$C_i = (P_b - P_{bb}) \times (f_y/f_c')\sqrt{(b/s)} \quad (3)$$

Since the ultimate load carrying capacity (P) is experimentally determined (P - $f_y A_s$) gives the contribution to load carrying capacity due to both indirect confinement of the steel fiber and tie confinement. This value is non-dimensionalised by dividing with $f_c A_g$. This means that $K_2 = (P - f_y A_s)/f_c A_g$ gives the strength of concrete as the ratio of the strength of concrete confined by lateral ties only. A plot of K_2 versus (RI) is given in Fig. 5. An examination of plot shows that there is a linear relationship between K_2 and (RI). A straight line then fit between these two parameters. The linear equation thus obtained, with 95 percent confidence limits, is given below:

$$K_2 = (1.0228 \pm 0.0236) + (0.1024 \pm 0.0101)(RI) \quad (4)$$

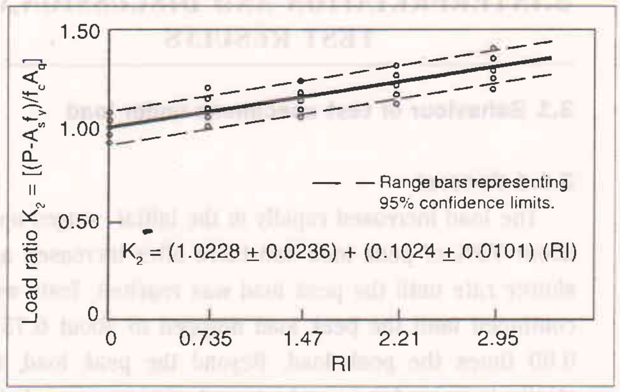

Fig. 5 – Load ratio (K_2) vs. reinforcing index (RI).

Hence, the final equation for the load carrying capacity (P) of concrete, indirectly confined with steel fiber in addition to lateral ties can be written as:

$$P = f_c'(1.0 + 0.55\, C_i)\,(1.0228 + 0.1024\,(RI))\, Ag + f_y A_s \quad (5)$$

3.3.2. Strain at peak strength

The ultimate strain increased with an increase in reinforcing index (RI). Fig. 6 shows the plot between the ratio

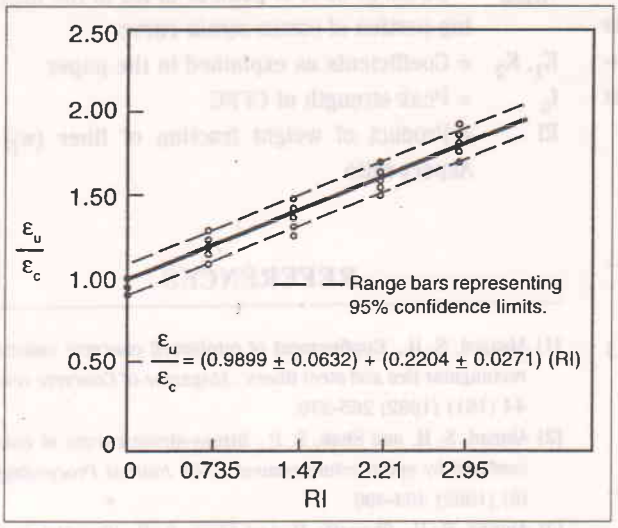

Fig. 6 – Ratio of strains at ultimate strength vs. reinforcing index (RI).

of observed strain (ε_u) at the peak strength of concrete indirectly confined by steel fiber, in addition to lateral ties, to the theoretical strain (ε_c) at peak strength of concrete confined by lateral ties only and the reinforcing index (RI). An examination of the plot clearly indicates that there is a linear relationship between RI and the ratio of strain at the peak strength of concrete indirectly confined by steel fiber in addition to lateral ties to the strain of concrete confined by lateral ties only.

A straight line fit between the reinforcing index (RI) and the ratio ($\varepsilon_u/\varepsilon_c$) resulted in the following equation:

$$(\varepsilon_u/\varepsilon_c) = (0.9899 \pm 0.0632) + (0.2204 \pm 0.0271)\ RI \quad (6)$$

The strain at peak strength of a CFRC section can be re written in the following form: (ε_u)

$$\varepsilon_u = \varepsilon_c (1.0 + 5.2\ C_i)\ (0.9899 + 0.2204\ (RI)) \quad (7)$$

3.3.3. Ductility of CFRC

The ductility of CFRC, as expressed by the strain at 85% of the peak strength in the descending portion of the stress-strain curve, is increased with the increase in the reinforcing index (RI). The observed strain at 85% of the peak strength) is expressed in terms of the theoretical strain (ε_u) given by Equation (7). A plot between the ratio ($\varepsilon_{0.85u}/\varepsilon_u$) and the reinforcing index (RI) is given in Fig. 7. The following relationship is obtained between the above ratio and the reinforcing index (RI):

$$(\varepsilon_{0.85u}/\varepsilon_u) = (1.8847 \pm 0.1223) + (0.121 \pm 0.0525)\ RI$$
$$\approx 1.8847 + 0.121\ RI \quad (8)$$

Further the area under stress strain curves, representing the toughness, increased with the increasing reinforcing index (RI) for a particular value of tie confinement

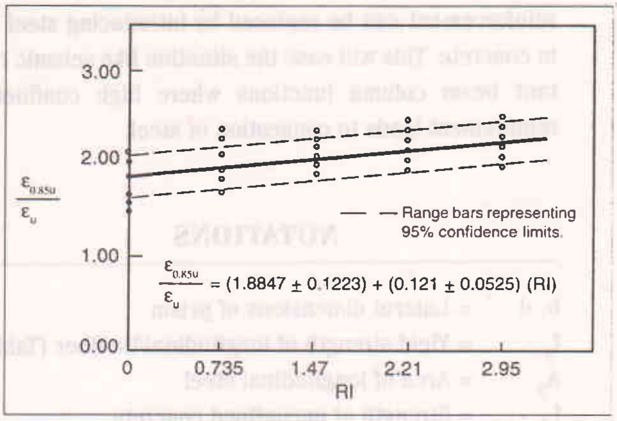

Fig. 7 – Ratio of strains at 0.85 of ultimate strength vs. reinforcing index (RI).

level (C_i). This gives an indication that for the same level of Energy absorption, the tie spacings may be increased with the inclusion of Fibers. This is advantageous in situations where congestion of reinforcement poses difficulty in casting.

4. CONCLUSIONS

The following conclusions can be drawn from the experimental investigation on CFRC:

1. The confinement of FRC improved the stress-strain behaviour under axial compression.

2. For the same level of tie confinement, the increase in volume fraction of fiber increased the ductility represented by the area under the stress-strain curve.

3. The tie confined FRC has improved the peak strength and strain at peak strength.

4. The improvement in strain is more pronounced compared to the improvement in the strength.

5. The improved peak strength with steel fiber varied linearly with reinforcing index (RI). The prediction equation for peak load carrying capacity of confined steel fiber reinforced concrete prism is:

$P = f_c^{'} (1.0 + 0.55\ C_i)\ (1.0228 + 0.1024\ (RI))\ Ag + f_y A_s$.

6. The improved strain at peak stress with indirect steel fiber confinement varied linearly with reinforcing index (RI) and can be expressed as:

$\varepsilon_u = \varepsilon_c^{'} (1.0 + 5.2\ C_i)\ (0.9899 + 0.2204\ (RI))$.

7. The ductility of CFRC, as expressed by the strain at 85% of the peak strength in the descending portion of stress-strain curve is increased with the increase in the reinforcing index (RI) and can be expressed as:

$\varepsilon_{0.85u} = \varepsilon_u\ (1.8847 + 0.121\ (RI))$.

8. For higher levels of confinement in reinforced concrete, there exist equivalent lower levels of confinement

in CFRC. Hence some amount of confining transverse reinforcement can be replaced by introducing steel fiber in concrete. This will ease the situation like seismic resistant beam column junctions where high confinement requirement leads to congestion of steel.

NOTATIONS

b, d = Lateral dimensions of prism

f_y = Yield strength of longitudinal/tie/fiber (Table 1)

A_s = Area of longitudinal steel

f'_c = Strength of unconfined concrete

ε_c = Strain at peak of unconfined concrete

f_c = $f'_c(1 + 0.55\ C_i)$ = Strength of tie confined concrete [7]

ε_c = $\varepsilon'_c(1 + 5.2\ C_i)$ = Strain at peak of tie confined concrete [7]

A_g = Gross cross sectional area

W_f = Weight fraction = ratio of weight of steel fiber and weight of concrete

C_i = Confinement index = $(P_b - P_{bb})\ (f_y/f'_c)\ (\sqrt{(b/s)})$

P_b = Ratio of volume of transverse steel to the volume of concrete

P_{bb} = Ratio of volume of transverse steel to the volume of concrete corresponding to a limiting pitch equal to 1.5b

f_v = Stress in the lateral ties

S = Spacing of ties

ε_u = Strain at peak of CFRC

$\varepsilon_{0.85u}$ = Strain at 85% of peak of CFRC in the descending portion of stress-strain curve

K_1, K_2 = Coefficients as explained in the paper

f_u = Peak strength of CFRC

RI = Product of weight fraction of fiber (w_f) and Aspect ratio

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