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Technical Note

BEHAVIOUR OF M_{60} CONCRETE USING FIBRE COCKTAIL IN EXTERIOR BEAM-COLUMN JOINT UNDER REVERSED CYCLIC LOADING

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ABSTRACT

The fracture of High Performance Concrete (HPC) occurs in a very explosive manner without previously exhibiting any cracking pattern as a warning. By using special fibre cocktails, (combinations of steel and polypropylene fibres) the explosive failure behaviour of High Performance Concrete (HPC) may be avoided. With the variation of cocktail composition different seismic performance of the material could be adjusted. An experimental programme has been carried out to compare the behaviour of high performance concrete and cocktail fibre reinforced high performance concrete beam column joint under reversed cyclic loading. HPC mix has been designed to obtain a concrete grade of M $_{60.}$ The mix was designed based on modified ACI 211 method suggested by Shetty [1]. Five numbers of exterior beam-column joints modeled to one fourth of a prototype of a building [2], designed according to Bureau of Indian Standards were cast and tested under reversed cyclic loading. The first specimen was made with high strength concrete and designed as per IS 456:2000 [3] and reinforced accordingly without considering the seismic requirement. The second specimen was made with high strength concrete,designed as per 1893 (Part I) 2002 [4] and reinforcements in the beam-column joint portion was detailed according to IS 13920-1993 [5], for seismic requirements. The remaining three specimens were similar to the first one but various combinations of cocktail fibre concrete in the joint region (constant % (1.5) of steel fibre and 0 to 0.4 % of polypropylene fibre) were used. The cocktail fibre combinations of 1.5% of steel fibre and 0.2% of polypropylene fibre have best performance considering the strength, energy dissipation capacity, and ductility factor. Results indicate that the addition of polypropylene fibre to the steel fibre is optimum for a percentage of 0.2, which have more energy absorbing capacity, less joint rotation, more shear strength, more curvature ductility factor and less reinforcement strain.

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Keywords: High strength concrete; hysteresis; cocktail; energy absorption; moment curvature and rotation

1. INTRODUCTION

Long-term performance of structures has become vital to the economics of all nations. Concrete has been the major instrument for providing stable and reliable infrastructure since the days of ancient civilization. So many research works were been carriedout in the 20th century. As a result, new materials and composites have been developed and improved cements evolved. Today concrete structures with a compressive strength exceeding 140 Mpa are being built world over. In research laboratories, concrete strengths of even as high as 800 Mpa are being produced. Damage in reinforced concrete structures are mainly attributed to shear force due to the inadequate detailing of reinforcement and the lack of transverse steel and confinement of concrete in the structural elements. High strength concrete is brittle in nature, demonstrating inadequate capacity to dissipate and absorb inelastic energy. The beam-column joints that are subjected to reverse cyclic loading require great care in detailing. The satisfactory performance of a beam-column joint, particularly under seismic loads, depends strongly on the lateral confinement of joint. The performance of beam-column joint by using steel fibre reinforced concrete under seismic conditions has been a research topic for many years [6,7 and 8]. The present study deals with the conventional reinforcement detailing in the beam-column joint and providing cocktail fibre [9,10 and 11] (which is the combinations of steel and polypropylene fibres) reinforced concrete. This cocktail fibre reinforced concrete in the joint region has more energy absorption capacity, less beam rotation, less reinforcement strain and less moment curvature.

Table 1: Details of the fibre

Sl. No	Properties	Steel fibre (corrugated)	Polypropylene fibre
1.	Length	30 mm	20 mm
2.	Diameter	0.50 mm	0.008 mm
3.	Aspect ratio	60	2500

2. EXPERIMENTAL PROGRAMME

HPC mix proportion for M_{60} grade concrete was obtained based on the guidelines given in modified ACI 211 method suggested by Shetty. Portion of the cement was replaced by micro fillers, such as silica fume and fly ash. In this study 10% replacement of cement by silica fume and 15% by fly ash were considered [12]. To increase the workability of concrete superplasticiser was added. The HPC mix proportion was M $_{60}$ grade concrete. The ratio of HPC mix was (1:0.19:0.07:1.42:2.07:0.35:0.021) (cement: silica fume: fly ash: sand: C.A: water: superplasticiser). The ratio of HPFRC mix was (1:0.19:0.07:1.31:1.92:0.35: 0.025+1.5%). Table 3 presents the characteristics of two different concrete mixes used to cast the test specimens. First mix was used for casting ordinary and seismic specimen and second mix by adding various proportion of polypropylene fibre was used for casting remaining three specimens. Table 2 shows the cube and cylinder compressive strength of different combinations of mixtures. The experimental programme included four specimens designed as per IS 456:2000 and one specimen designed as per IS 1893 (Part 1): 2002 and detailed as per IS 13920-1993. Figure 1 shows the beam-column joint with seismic detailing. Of the above four, three specimens were provided with cocktail fibres as presented in Table 2. Figure 2 shows the beam-column joint without seismic detailing and cocktail fibre reinforced concrete was used in the joint region. M_{60} concrete was used for casting the beam-column joints. Cocktail fibre combination (steel fibre and polypropylene fibre) consisting of 1.5% of steel fibre and 0 to 0.4% of polypropylene fibre with an increment of 0.2 % were used.

Table 2: Details of the test specimen and compressive strength

Sl. No	**Specimen Id**	**Detailing of lateral reinforcement**	**Percentage of fibre**		**Compressive strength N/mm^2**	
			Steel	**Polypropylene**	**Cube**	**Cylinder**
1	III O1	With out seismic detailing	-	-	76.5	61.2
2	III S1	With seismic detailing	-	-	76.5	61.2
3	III F11	With out seismic detailing	1.5	-	84.5	69.3
4	III F 21	With out seismic detailing	1.5	0.2	86.4	71.8
5	III F31	With out seismic detailing	1.5	0.4	82.6	66.9

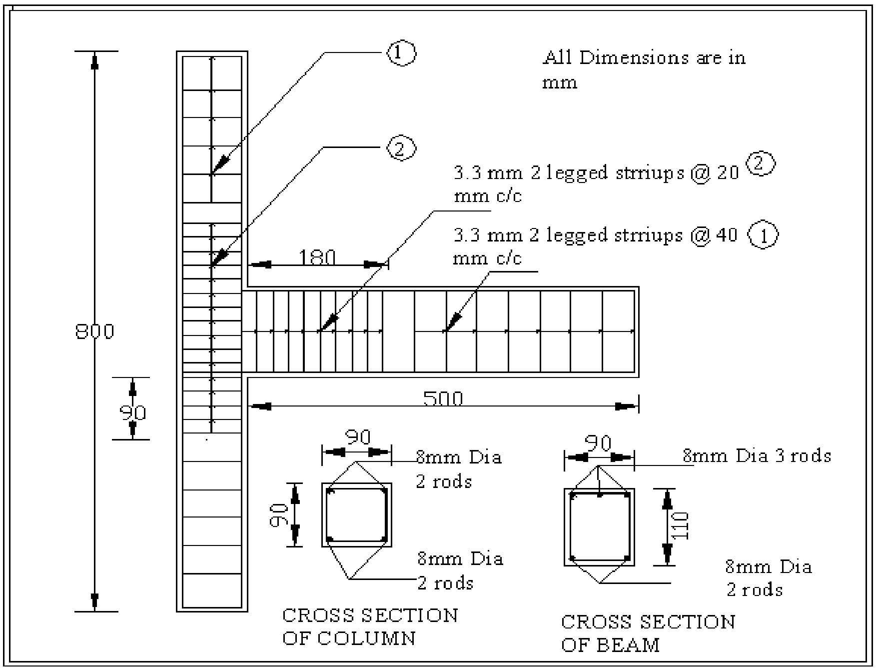

Figure 1. Seismic joint

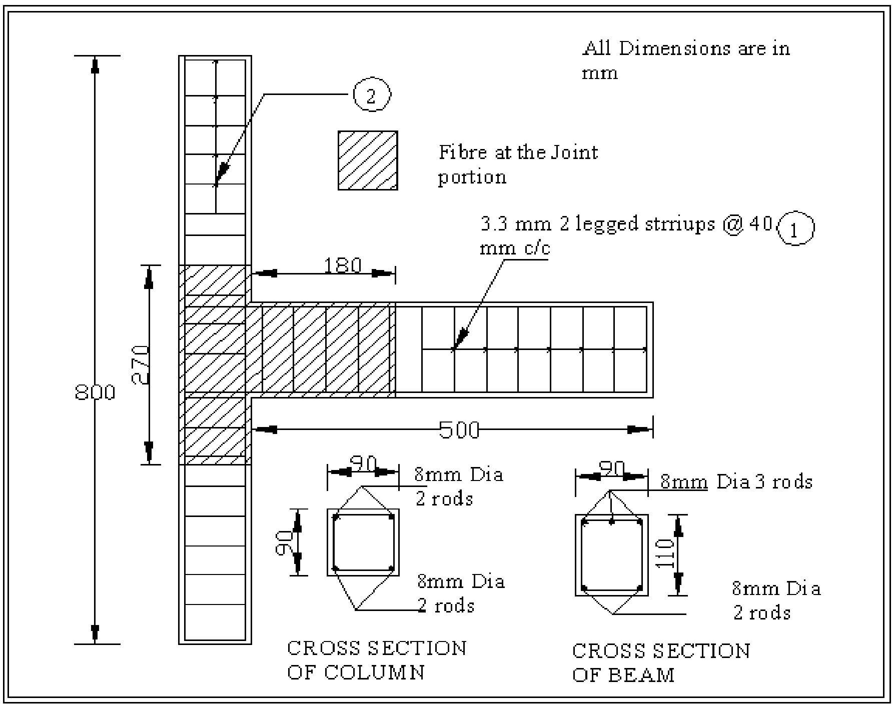

Figure 2. Fibre joint

Table 3: Characteristics of concrete mixtures (M_{60}) (1: 0.19: 0.07: 1.42: 2.07: 0.35: 0.021.)

Material	Unit	Plain concrete	Fibre concrete
Cement	kg/m^3	378	378
Fly Ash	kg/m^3	88	88
Silica fume	kg/m^3	36	36
Fine aggregarte (sand)	kg/m^3	656	608
Coarse aggregate (6mm to 10mm)	kg/m^3	962	891
Water binder ratio	kg/m^3	209	207
Super plasticiser	Lit/ m^3	10	11.75
Steel fibre (1.5 %)	kg/m^3	-	117.75
Polypropylene fibre (0, 0.2 and 0.4 %)	kg/m^3	-	(0,1.82 & 3.64)

2.1 Materials

Ordinary portland cement (OPC-53 Grade) was used in this study. Locally available river sand was used as fine aggregate. Ordinary portable water was used for casting as well as curing of the specimens. The macro corrugated steel fibres and polypropylene fibres (synthetic fibre) were used in this investigation. The physical properties of these fibres are presented in Table 1. Superplastisiser used in this study is Conplast Sp 430.

2.2 Test setup

The test setup is shown in Figure 3. The specimen was mounted such that the column is in vertical position and beam is in horizontal position. For strain controlled testing screw jack and hydraulic jack were used to apply displacement at the beam end [13 and 14]. The hydraulic jack was fixed at the strong floor and screw jack was fixed to the loading frame at the top. Reversed cyclic load was applied at 50 mm from the free end of the beam portion of the assemblage. The loading programme consisted of a simple history of reversed symmetric displacement of amplitudes 5mm, 10mm, 15mm, 30mm and 45mm. The test was displacement controlled and the specimen was subjected to an increasing reversed cyclic displacement up to failure. To record the load precisely, proving ring was used. The dial gauges were used to monitor the deflection. Each displacement cycle consisted of a cycle of upward and downward displacement of beam end position. Dial gauges were fixed at a distance of D and 2D from the column face on the beam to measure the beam deflections.

Strain gauges were fitted in the beam top and bottom reinforcement and column reinforcement to measure the reinforcement strain. This loading history permitted the evaluation of parameters such as load- displacement relation (hysteresis loop), strength, curvature ductility, moment curvature relation, beam and column rotation, shear and energy dissipation etc.

Figure 3. Test set-up

3. RESULTS AND DISCUSSION

3.1 Hysteresis loop

The force- displacement hysteresis loops for specimen III F21 is shown in Figure 4.

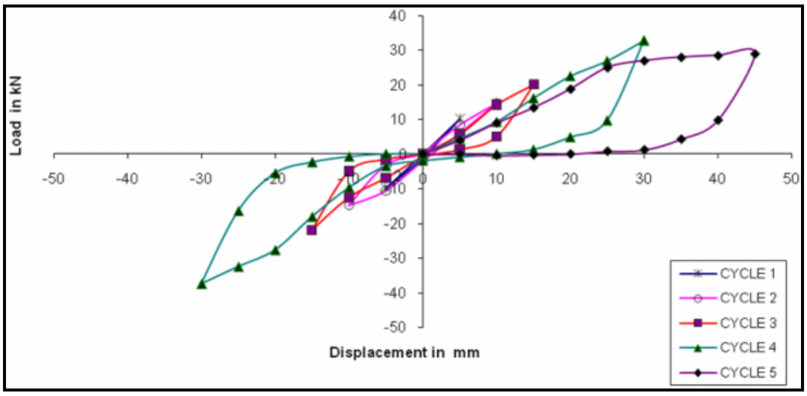

Figure 4. Histerisis loop for III F 21

3.2 Energy dissipation

The area enclosed by a hysteresis loop at given cycle represents the energy dissipated by the specimen during the cycle. The Figure 5 shows the energy dissipating capacity of all the specimens. From Table 4 it is observed that compared to ordinary specimen (III O1) the energy dissipation capacity is improved by 178.7% by the addition of 1.5% of steel fibre and 241% by adding cocktail fibre combination of 1.5% steel fibre and 0.2% of polypropylene fibre [15 and 16].

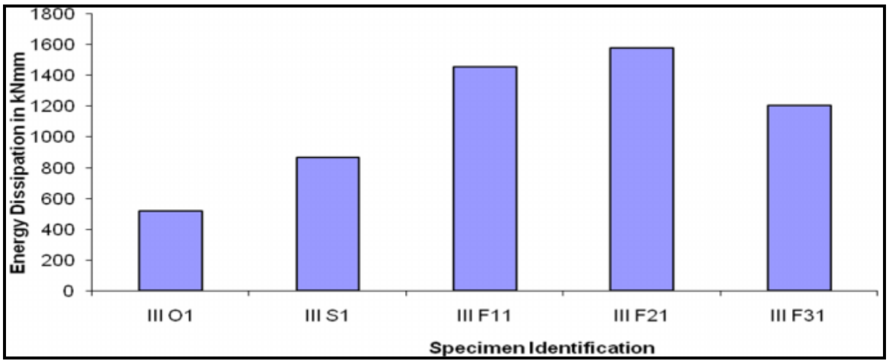

Figure 5. Energy dissipation capacity of all the specimens

Table 4: Ultimate load, maximum deflection at failure and energy dissipation capacity

Sl. No	**Specimen Id**	**Ultimate load (p_u) kN**		**Deflection (mm) (δ_u)**		**Energy dissipation capacity (e_{cu}) kNmm**
		Positive	**Negative**	**Positive**	**Negative**	
1	III O1	22	21.2	30	30	522
2	III S1	23.4	26	45	30	866
3	III F11	30.6	34.4	45	30	1455
4	III F 21	32.7	37.6	45	45	1781
5	III F31	28.4	30.4	45	30	1207

3.3 Joint shear stress

For the exterior beam-column joint the horizontal and vertical joint shear stresses [17] (τ_{jh}, τ_{jv}) can be given as

$$\tau_{jh} = \frac{H}{A^{h}core}\left[\frac{L_b}{d_b} - \frac{L_b + 0.5D_b}{L_c}\right] \tag{1}$$

and

$$\tau_{jv} = \frac{H}{A^{v}\text{core}}\left[1 - \left(\frac{L_b + 0.5D_c}{L_c}\right)\left(\frac{L_c - D_b}{d_c}\right)\right] \quad (2)$$

Where L_b and L_c are the length of beam and column respectively; D_b and D_c are the total depth of beam and column respectively; d_b and d_c are the effective depth of beam and column respectively; A^{h}core and A^{v}core are the horizontal and vertical cross sectional areas of the joint core resisting the horizontal and vertical joint shear forces, respectively.

Table 5 shows the horizontal and vertical shear stresses developed at the joint due to the ultimate load and the limiting shear stress specified in the joint as per IS codes. The shear stresses induced in the joint (nominal shear capacity) which is calculated at the ultimate load are significantly higher than the limiting values specified in the Joint ACI-ASCE document [11]. ACI suggests the nominal joint shear stress of $1.7\sqrt{f_c'}$ if confined on four faces, $1.25\sqrt{f_c'}$ if confined on three faces and $1.0\sqrt{f_c'}$ for other cases [17].There is no limiting shear stress values in the code for fibre concrete. From Table 4 it is observed that the specimen III F21 has the max vertical and horizontal shear stresses and when we used fibre concrete in the joint region the shear resisting capacity is more than the conventional and seismic joint (joint with lateral reinforcement). From this table and Figure 6 it is observed that the specimen III F21 has the maximum ultimate load, vertical and horizontal shear stress.

Table 5: Comparison of observed ultimate shear capacity with code prescribed limiting values

Sl. No	**Specimen Id**	f_c' **N/mm²**	**Ultimate load kN**	**Horizontal shear stress** τ_{jh}, **in** 10^{-3} **N/mm²**	**Vertical shear stress** τ_{jv} **in** 10^{-3} **N/mm²**	**Limiting shear stress as per ACI=** $1.0\sqrt{f_c'}$ 10^{-3} **N/mm²**
1	III O1	61.2	22	13.2	11.6996	7.82304
2	III S1	61.2	26	15.6	13.8268	7.82304
3	III F11	69.3	34.4	20.64	18.2939	8.32466
4	III F 21	71.8	37.6	22.56	19.9957	8.47349
5	III F31	66.9	30.4	18.24	16.1667	8.17924

3.4 Moment curvature behaviour

By using the test result the moment-curvature relationship for all the specimens were calculated. The strains measured at 15mm below the extreme compression fibre and 15mm above the extreme tension fibre have been used to calculate the curvature, ϕ of the beam for every loading stage using the relation [18]

$$\Phi = \frac{e_t + e_b}{(d - dc)} \tag{3}$$

Where,

e_t = strain in the top reinforcement

e_b = strain in the bottom reinforcement

d = effective depth

dc = compressive reinforcement cover

The values of moment M were calculated using the experimental values of load and lever arm. Table 6 shows the moment, curvature ductility at peak load and yield load. These values of M and Φ were used to obtain moment-curvature plots for the joint.

Table 6: Moment and Curvature Ductility Factor

Sl.No	Specimen Id	Φ_u (Curvature at peak load) $X10^{-2}\ \frac{1}{m}$	Φ_y (Curvature at yield load) $X10^{-2}\ \frac{1}{m}$	Curvature ductility factor	Moment at peak load kNmm
1	III O1	4.0927	3.0772	1.33	9900
2	III S1	5.4159	3.0772	1.76	11700
3	III F11	9.4373	3.0443	3.1	15480
4	III F 21	10.828	3.0416	3.56	16920
5	III F31	11.472	3.0510	3.76	13680

3.5 Curvature ductility factor

The capacity of the member to deform beyond its initial yield deformations with minimum loss of strength and stiffness depends upon the ductility factor which is defined as the ratio of the ultimate deformation to its yield deformation at first yield. Ductility may be defined easily in the case of elastoplastic behaviour. Ductility factors in beam-column joint have been defined in terms curvature at critical section as

$$\text{Curvature ductility factor} = \frac{\Phi_u}{\Phi_y} \tag{4}$$

Where,

Φ_u = curvature at peak load

$$\Phi_y = \text{curvature at yield} = \frac{f_y}{E_s\left(d-x\right)} \tag{5}$$

Where,

f_y = the yield strength of reinforcement

E_s =Modulus of elasticity of steel

d = the effective depth

x = depth of neutral axis

The curvature at peak load and curvature ductility factor thus calculated for all specimens are given in Table 6. From the table it may be noted that the cocktail fibre reinforced specimens have better values than the other specimens.

4. CONCLUSIONS

1. The fibres are effective in resisting deformation at all stages of loading from first crack to failure.
2. Comparing High strength concrete joint, in the fibre reinforced concrete joint, the first crack load and ultimate load were found to be increased.
3. However from the experimental results it is known that the ductile behaviour , ultimate strength, joint shear stress, curvature ductility factor and energy dissipation capacity are also increased by adding polypropylene fibre in addition to the steel fibre with the cocktail combination of 1.5 % steel fibre and 0.2% polypropylyne fibre.

Hence the cocktail combination of 1.5% of steel fibre and 0.2% of polypropylene fibre is highly recommended in beam column joint subjected to reverse cyclic loading for High Strength Concrete (M_{60}).

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