BEHAVIOUR OF HIGH PERFORMANCE FIBER REINFORCED CONCRETE BEAM JOINTS UNDER SEISMIC LOADING

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Abstract*:* The strength and ductility of structures primarily depend on proper detailing of reinforcement in beam-column joints. Under seismic excitations, beam-column joint region is subjected to high horizontal and vertical forces whose magnitude is much higher than those within the adjacent beams and columns. Beam-column joints have been recognized as critical element in seismic design of reinforced concrete (RC) frames. Conventional concrete loses its strength after formation of multiple cracks. Fiber reinforced high performance concrete (FRHPC) can be utilized to sustain for cyclic loading. Present study is aimed at investigating structural behaviour of beam-column joints using normal strength concrete (NSC) and FRHPC based beam-column joints utilizing steel fibers in varied aspect ratios, types and fiber contents. Beam-column joint of a multi-storeyed building has been modelled and scale down model experimented considering the scaling effect. Fifteen specimens of beam-column exterior joint were cast and tested using FRHPC in different fiber contents to study load-deformation behaviour, failure pattern, stiffness degradation and ductility associated parameters. The typical results illustrate significant increase in compressive, tensile and flexural strength values in HPFRC based control specimens. Beam-column joints corresponding to different grades of HPFRC with varied fiber content and aspect ratio & shape has been observed to give maximum load carrying capacity, energy absorption capacity and resilience. An optimum fiber contents corresponding to this value may therefore be utilized to provide significant dimensional stability, integrity, strength and ductility to beam- column joints subjected to cycle loading and can be substituted for conventional transverse reinforcement thereby allowing for relaxation in ties and stirrups in beam column joints.

Keywords: *Beam-column joint, High performance concrete, Cyclic loading, Stiffness degradation.*

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1. Introduction

The main reasons in a joint shear failure, are in-adequate transverse reinforcement in the beam-column joint region and strong beam / weak column design. At the same time, detailing of reinforcement in beam-column joints affects the strength and ductility of structures. It has been identified that insufficient seismic performance in the reinforced concrete (RC) structures built with low strength concrete or insufficient reinforcement and improper reinforcement detailing, resulting in non-ductile performance of moment resisting frames, lead to brittle failure of the members in a devastating manner. These types of local failures can cause global failure of mechanism required in seismic upgrading of deficient structures.

It has been observed that conventional concrete loses its tensile resistance when multiple cracks develop in a structure. Transverse reinforcement in the form of closely spaced hoops was recommended in the ACI-ASCE Committee 352 report [1]. However, casting of beam-column joints become difficult due to congestion of reinforcement which may lead to honeycombing in concrete [2].

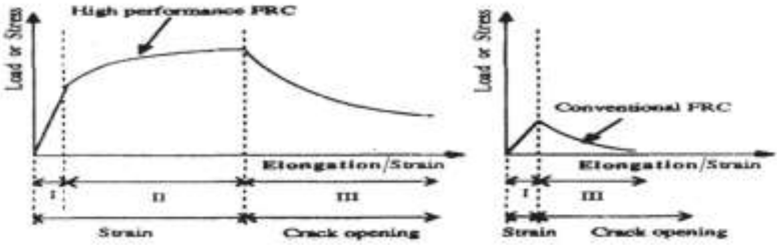


Figure 1: Stress-strain behaviour of cementitious matrices [3]

High Performance implies an optimized combination of structural properties such as strength, toughness, energy absorption, stiffness, durability, multiple cracking and corrosion resistance considering the final cost of the material and above all, the produce manufactured. Fiber reinforced high performance concrete (FRHPC) is defined by an ultimate strength higher than their first cracking strength and the formation of multiple cracking during inelastic deformation process. High performance is meant to distinguish structural material from the conventional one, as well as to optimize a combination of properties in terms of final application in real civil engineering structures, Figure 1.

The objectives of present study are to develop FRHPC of higher strength and performance, and to utilize such FRHPC in beam-column joint region, to replace transverse reinforcement at an optimum fiber content, type and aspect ratio.

The critical review of existing literature reveals that various studies were conducted to investigate behaviour of beam-column joints with normal strength concrete. [4] studied effect of fibers on the beam-column joints and developed equations for predicting shear strength of joints for normal strength concrete. [5] investigated behaviour of fiber reinforced concrete corners under opening bending moments. These investigations indicated that because of low fiber volume percentage, there is only a noticeable gain in efficiency with increase in fiber volume fraction up to a certain limit beyond which there is a drop-in mix workability and joint efficiency. [6] investigated experimentally effect of using SIFCON in the hinging zones of multi-storeyed frames subjected to cyclic loading. It was concluded that the use of SIFCON in the hinging zones increases first crack load and ductility by 40 &100 percent, respectively. The energy absorption capacity was also increased by 50 percent by adopting SIFCON in the selected fuse locations of R C structures. [7] also experimentally proved the confinement effects of fibers in the joint region, and a reduction in the lateral reinforcement by using fiber concrete. [8] conducted extensive research on parameters that influence behaviour of cyclically loaded joints and derived equations for calculating shear strength of the joints. [9] described the experimental results of ten steel fiber reinforced high performance concrete (SFRHPC) exterior beam-column joints under cyclic loading. Test results indicated that the provision of steel fiber reinforced high performance concrete (SFRHPC) in beam column joints enhances strength, ductility and stiffness, and is one of the possible alternative solutions for reducing the congestion of transverse reinforcement in beam-column joints.

Several researchers [10-13] also studied beam-column connections subjected to opening bending moments. It was found that in all the RC specimens, the joints failed before reaching the capacity of the connecting members. There was significant difference in different joint's efficiency due to variety of reinforcement details. Based on the comparison of observed responses, it was found that the addition of 1.5 percent steel fibers were effective in reducing amount of steel bars in the beam–column joints of railway bridges. Further, it is observed that earlier researchers used SFRC in the beam-column joints and fiber volume content (Vf) was redistricted to 2 percent by volume. Tests of three fiber reinforced cantilever walls with no transverse reinforcement provided for shear or confinement showed that high volume fractions of steel fibers (between 3.5% and 6%) could potentially replace the reinforcement required to resist shear forces [16]. However, the specimens were subjected to relatively low nominal shear stresses, which are not representative of the levels of base shear stress during strong ground motions.

2 Experimental Investigation

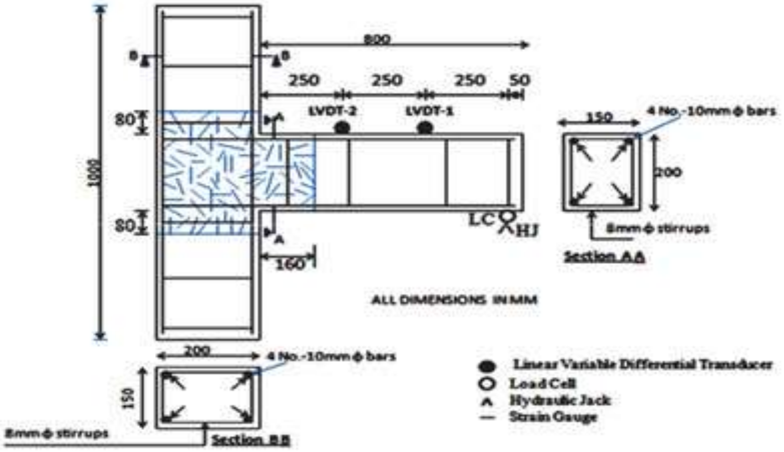


Figure 2: Experimental setup and position of steel fiber in a typical beam-column specimen

To investigate structural performance of beam column joints using FRHPC, an exterior beam-column joint region of a multi-storeyed building was analysed. The dimensional analysis was carried out to one third scale to simulate model study of the prototype structure, Figure 2. High Performance Concrete HPC) mix was designed as per guide lines provided [1] to achieve compressive strength of the order of M 50 grade, Table 1.

Table 1: Composition of HPC Mix

Cement (kg/m3)	Silica Fume (kg/m3)	Fine Aggregate (kg/m3)	Coarse Aggregate (kg/m3)	Super-Plasticizer (kg/m3)	Steel Fiber (%)	Water (Kg/m3)	Water Cement Ratio
492	123	615	984	4.90	0	154	0.25

The steel fibers corresponding to 6, 8, 9 and 10 percent were added to HPC to achieve FRHPC, keeping all other constituents same as those for HPC. The proportioning of various constituents of HPC was based on ACI method, conforming to Table 2.

Table 2: Typical Range of HPC Mix Composition [14]

Constituent	Typical Range (kg/m3)
Powder	400-750
Water	150-210
Coarse Aggregate	750-1000
Fine Aggregate	360-550

The fine aggregate, locally available natural river sand having fineness modulus of 2.57 and crushed stone aggregate and crushed foundry slag in ratio of 1.5: 0.5 with combined fineness modulus 6.52, as coarse aggregate conforming to IS: 383 were used. Straight and crimped cylindrical steel fibers of 40 and 50 mm length, 0.50mm diameter with aspect ratio (k) of 80 and 100 respectively were added in different proportions of 0, 6, 8, 9 and 10 percent by volume of concrete mass, to achieve FRHPC.

Figure 3 illustrates typical view of tested specimens of crimped and straight steel fibers based control FRHPC cubes and cylinders of size 150 mm, 300 x 150 mm respectively. These controlled specimens were tested to obtain compressive, split tensile and flexural strengths, at ages of 28 days respectively.

2.1 Exterior Beam-Column Joint Specimens

For casting exterior beam column joint specimens, water proof shuttering grade ply-wood moulds were used. Reinforcement cages were fabricated with nominal reinforcement using 10 mm dia Fe 500 grade bars as main reinforcement in beam and column whereas 8 mm dia bars were used for shear stirrups. Required quantity of cement, silica fume, fine and coarse aggregate were mixed thoroughly in a drum type concrete mixture in which 50% of water was added to the dry mix. The remaining 50% water was mixed with super-plasticizer and these constituents are mixed till a uniform mixture is obtained. After 24 hours these specimens were demoulded and cured for 28 days. The specimens could become dry after 28 days for some time and painted before testing, Figure 4. A special steel loading frame of 600 kN load capacity was used to test these specimens. A constant cyclic load was applied through hydraulic jack of 100 kN capacity with 5 kN incremental loading.

Figure 3: Tested specimens

Figure 4: Beam-column joint specimen during testing

3. Analysis of Results and Discussion

Seventeen nos. beam-column joint specimens cast as per the procedure discussed earlier were tested. These specimens were designated as per nomenclature mentioned in Table 3 for different type of fibers (straight and crimped), aspect ratio and fiber contents (0, 6, 8, 9 and 10 percent) by volume respectively. These specimens were tested to investigate the compressive, tensile and flexural strength of HPC & FRHPC at 28 days. Table 3 illustrates compressive, tensile and flexural strength of FRHPC control specimens. The compressive strength increases significantly by increasing the fiber content from 6 to 9 percent. The compressive strength increased with increasing steel fiber volume due to the transverse confinement effect of the steel fibers, which restrained the lateral expansion of SFRC specimens. In most cases there is decrease in strength from 9 to 10 percent fiber volume contents. In comparison to Non-Fibrous Concrete (NFC), it is increasing for SF 40/6, SF 40/8, SF 40/9, ZZ 50/6, ZZ 50/8 & ZZ50/9 and further decreasing for SF 40/10, SF 50/10, ZZ 40/10 & ZZ 50/10. The compressive, split tensile and flexural strengths for the specimen containing crimped fibers volume of 9 percent with k= 80 was found to be maximum values 72.0, 20.96 and 16.85 N/mm2 respectively. Therefore, fiber content corresponding to 9 percent by volume of crimped fiber and k= 80 is considered as optimum for practical consideration. It has also been noted that the value of σc / σt decreases from 8.36 to 3.44, σc / σf decreases from 9.04 to 4.27 and σt / σf decreases from 1.31 to 0.86. It is seen that FRHPC composites differ from conventional HPC in the sense that matrix consists of very fine particles from behavioural view point. FRHPC tensile strain is much higher than that of the conventional HPC. This in turn, improves cracking behaviour, ductility and energy absorption capacity of the composites.

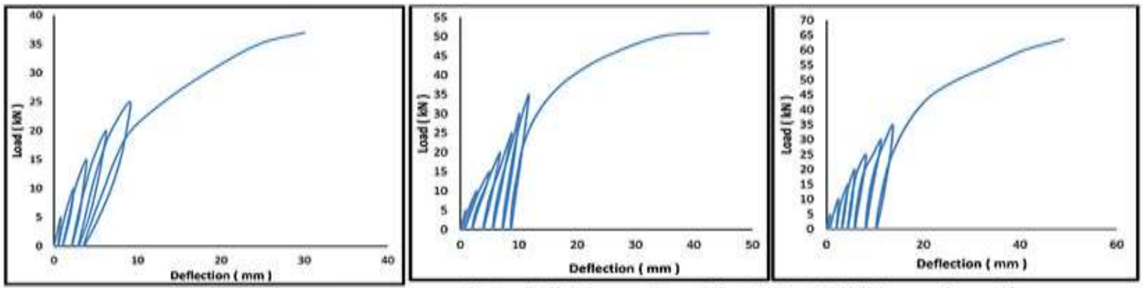
Table 3: Strength Characteristics of FRHPC Control Specimens

Specimen Designation	Compressive Strength, σc (N/mm2)	Split Tensile Strength, σt (N/mm2)	Flexural Strength, σf (N/mm2)	(σc / σt)	(σc / σf)	(σt/σf)
NFC	53.0	6.34	5.86	8.36	9.04	1.08
SF 40 / 6	54.5	7.25	6.38	7.52	8.54	1.14
SF 40 / 8	56.2	7.78	7.20	7.22	7.81	1.08
SF 40 / 9	59.8	12.46	10.65	4.80	5.62	1.17
SF 40 / 10	58.3	9.48	9.91	6.15	5.88	0.96
SF 50 / 6	56.2	8.14	8.25	6.90	6.81	0.99

SF 50 / 8	58.8	9.28	10.76	6.34	5.46	0.86
SF 50 / 9	71.6	15.46	12.23	4.63	5.85	1.26
SF 50 / 10	59.2	9.81	10.39	5.99	5.66	0.94
ZZ 40 / 6	68.4	12.67	10.26	5.40	6.67	1.23
ZZ 40 / 8	70.2	15.49	11.85	4.53	5.92	1.31
ZZ 40 / 9	72.0	20.96	16.85	3.44	4.27	1.24
ZZ 40 / 10	61.5	13.06	10.98	4.71	5.60	1.19
ZZ 50 / 6	62.8	11.29	9.56	5.56	6.57	1.18
ZZ 50 / 8	65.6	14.25	11.34	4.60	5.78	1.26
ZZ 50 / 9	69.5	16.20	14.64	4.29	4.75	1.11
ZZ 50 / 10	60.7	9.78	10.22	6.21	5.94	0.96

Load Deflection Behaviour

Table 4 illustrates parametric study of load deflection behaviour of beam column joints. It has been observed that the maximum value of first crack and ultimate loads were found to be 23.5 and 63.6 kN in specimen ZZ 40/9. The first crack deflection (Δcr) and ultimate deflection(Δu) were also found to be maximum in the same specimen. This illustrate that the specimen ZZ 40/9 provides maximum strength and ductility when compared with other specimens. Figure 5 shows the load deflection curves of beam-column joints for various fiber types, fiber volume and aspect ratios at critical location of LVDT-1.



(a) For NFC beam-columns Joint **(b) For SF40 /9 beam-column Joint** **(c) For ZZ40 /9 beam-column Joint**

Figure 5: Load deflection curve for beam-column joints at LVDT-1

Table 4: Parametric Study of HPC & FRHPC Beam-Column Joints

Designation	First Crack Load, Pcr (kN)	Ultimate Load, Pu (kN)	First Crack Deflection (mm)	Ultimate Deflection (mm)
			Δ cr	Δu
NFC	13.8	37.0	7.356	30.050
SF 40 / 6	14.2	40.5	6.026	40.020
SF 40 / 8	15.0	43.2	6.705	41.920
SF 40 / 9	18.6	51.0	7.366	42.500
SF 40 / 10	15.8	48.5	6.225	39.660
SF 50 / 6	14.5	46.5	5.788	39.315
SF 50 / 8	16.7	48.2	6.738	40.915

SF 50 / 9	19.6	55.0	7.241	41.920
SF 50 /10	17.6	51.5	5.682	38.565
ZZ 40 / 6	19.8	48.8	7.478	45.650
ZZ 40 / 8	20.5	52.5	8.047	46.980
ZZ 40 / 9	23.5	63.6	9.192	48.935
ZZ 40 / 10	22.4	58.2	8.434	44.310
ZZ 50 / 6	17.2	45.2	6.415	44.740
ZZ 50 / 8	19.5	49.3	7.715	45.695
ZZ 50 / 9	21.8	59.4	9.092	48.605
ZZ 50 / 10	20.3	55.7	8.515	43.560

It has been observed that the first crack load (Pcr) and the ultimate load (Pu) of the FRHPC beam-column joint specimens have been significantly improved in comparison to NFC. It can therefore be inferred that the FRHPC utilizing the fiber content up to 9 percent increases load carrying capacity of the joint significantly. The enhancement, however, is not significant when fiber content is increased to 10%. The Vf = 9 is therefore considered as an optimum value.

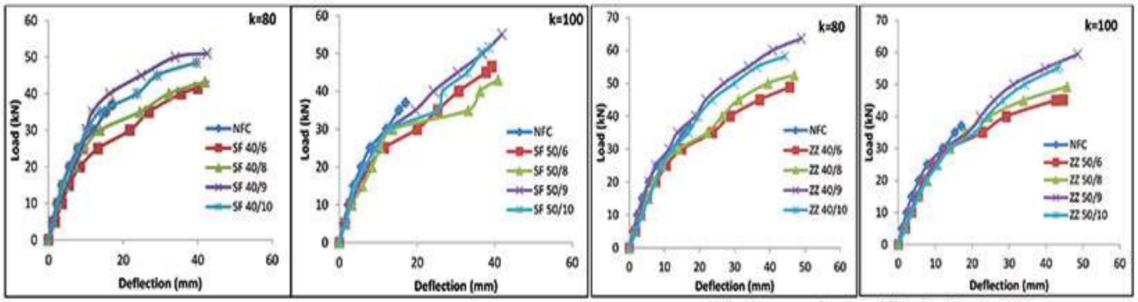
The area under the curve up to the first crack load represents the resilience of the specimens. The result shows that the percentage increase in resilience was 488, 399, 361 and 357 for specimens ZZ 40/9, ZZ 50/9, SF 50/9 and ZZ 40/10 respectively. The area under the load deflection curve up to the failure of the specimens represents the energy absorption capacity, i.e. Toughness. The results illustrate that percentage increase in toughness was 302,262,226 and 211 for the specimens ZZ 40/9, ZZ 50/9, ZZ 40/10 and ZZ 40/8 respectively, as compared to NFC. The ductility is defined as the ability to sustain inelastic deformation by the structural member without significant loss in resistance and without substantial loss of strength. The percentage increase in ductility was 87, 82, 76, and 68 for specimens ZZ 50/6, SF 50/6, SF 40/6 and SF 40/10 respectively. The result values of resilience, toughness and ductility are shown in Table 5.

Table 5: Ductility Associated Parameters for FRHPC Beam-Column Joint Specimens

Specimen Designation	Resilience	Increase in Resilience (%)	Toughness (kN.mm)	Increase in Toughness (%)	Toughness Index		Ductility	Ductility Index	Increase in Ductility(%)
					I3	I5			
NFC	22	-	516	-	9.27	14.98	3.20	4.20	-
SF 40 / 6	52	139	1071	107	6.01	12.47	5.64	6.64	76
SF 40 / 8	71	230	1247	141	5.79	12.61	5.25	6.25	64
SF 40 / 9	91	322	1488	188	5.92	13,15	4.77	5.77	49
SF 40 / 10	62	190	1284	149	6.72	13.23	5.37	6.37	68
SF 50 / 6	57	164	1106	114	5.93	12.54	5.79	6.79	82
SF 50 / 8	81	276	1290	150	5.53	12.66	5.07	6.07	59
SF 50 / 9	99	361	1548	200	5.95	13.37	4.79	5.79	50
SF 50 / 10	69	224	1349	161	6.24	13.48	5.78	6.79	81
ZZ 40 / 6	75	249	1430	177	5.90	13.52	5.10	6.10	60
ZZ 40 / 8	89	316	1608	211	6.04	14.15	4.84	5.83	51

ZZ 40 / 9	126	488	2076	302	5.94	14.99	4.32	5.32	35
ZZ 40 / 10	98	357	1685	226	6.88	15.22	4.25	5.25	33
ZZ 50 / 6	62	190	1382	168	5.93	13.94	5.97	6.97	87
ZZ 50 / 8	83	288	1564	203	6.91	14.68	4.92	5.92	54
ZZ 50 / 9	107	399	1869	262	6.78	15.65	4.35	5.35	36
ZZ 50 / 10	92	329	1502	191	5.88	13.12	4.12	5.12	29

Ductility associated parameters like ductility index and toughness index increases significantly in FRHPC based beam column joints. FRHPC joints undergo small displacement without developing wider cracks when compared to NFC joint. The failure is characterized by multiple closely spaced finer cracks. This indicates that FRHPC based joints impart very high ductility which is one of the essential property for beam column joint. FRHPC based beam column joints would impart dimensional stability and integrity of the joints. The failure was found to be accompanied by dissipation of large amount of dissipation energy as the area under the curve corresponding to ZZ 40/9 percent fiber content was maximum illustrating high ductility.



(a) Envelope curves for straight fibers With k = 80 and k=100

(b) Envelope curves for crimped Fibers k = 80 and k=100

Figure 6: Comparison of the envelope curves

The envelope curves are also obtained by joining the peak points of each cycle. A comparison of envelope curves for different volume fraction of fibers is shown in Figure 6. In cyclic loading, moreover, when unloading takes place, tip of the crack becomes blunt and during the subsequent cyclic loading, more energy is required to propagate the crack or to change the direction of crack propagation from the blunt crack tip. This in turn increases ultimate load capacity of the joint. It has been observed that toughness is maximum corresponding to specimen ZZ 40/9. It may therefore be inferred that energy absorption capacity of beam-column joint increases up to Vf = 9 percent, beyond which widening of multiple micro cracks started and additional load applied is dissipated in widening of these cracks.

Stiffness Degradation

During the testing of specimen under cyclic loading, the materials (i.e. concrete and steel) are subjected to loading, unloading and reloading operations, which starts the development of micro-cracks inside the joint, leading to failure of joint at ultimate load. To obtain the degradation of stiffness, the following method has been adopted.

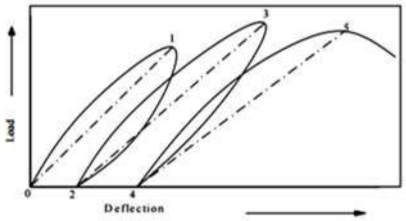
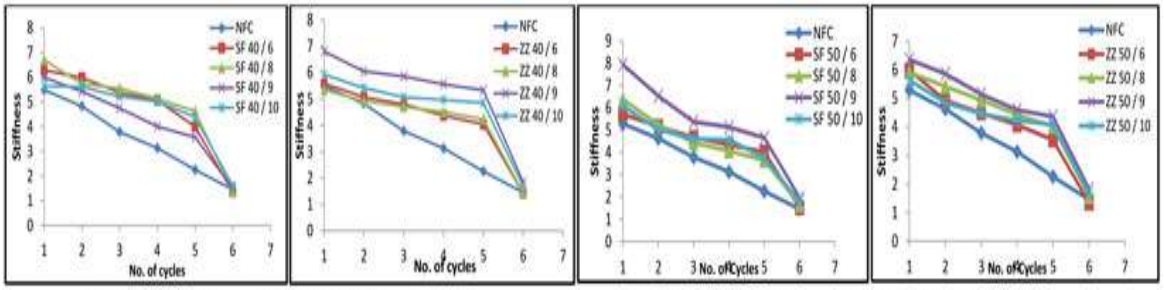


Figure 7: The method adopted for determination of secant stiffness

The slope of the line joining 0 - 1 for the first cycle, 2 - 3 for second cycle and similar procedure was adopted for all other cycles. The first five cycles of loading are 0-5, 0-10, 0-15, 0-20 and 0-25 kN and the 6th cycle of loading shown in stiffness degradation curves is the last loading cycle of each specimens. The values of secant stiffness obtained for each cycle is obtained and plotted for all the specimens as shown in Figure 8. The above behaviour may be concluded to the fact that at the first cycle, micro-cracks would not have initiated and hence the fibers were not effective in the absence of crack formation. During this process, stiffness of the FRHPC joints will not undergo much reduction when compared to NFC.



Straight Fiber k=80 Crimped Fiber k=80 Straight Fiber k=100 Crimped Fiber k=100

Figure 8: Relationship between stiffness and number of cycles

Failure Mechanism

The failure pattern of beam column junctions corresponding to various fiber contents is shown in Figure 9. It has been observed that multiple fine cracks were developed at the interface of FRHPC based beam-column joints specimens and propagated away from the beam-column joints. It has further been observed that in case of straight fiber based specimens, failure occurred in fiber core portion of the beam, as illustrated in Figure 9 (b). However, in crimped fiber based specimens, multiple hair line cracks were developed in beam portion first which propagated towards the column core on further increasing the load, as illustrated in Figure 9(c). Multiple fine cracks were found to develop in almost all specimens indicating that inclusion of fibers impart significant ductility in beam-column joint region which is considered as one of the essential property for structures constructed in earthquake prone regions. The dimensional stability and integrity of the joint was also found to be improved. It may therefore be inferred that the spacing of lateral ties in columns and stirrups in beams, in the beam-column joints region may be increased due to inclusion of fibers to avoid congestion of conventional reinforcement.

(a) Failure pattern of NFC beam-column joint (b) SF40/9 HPFRC beam-column joint (c) ZZ 40/9 HPFRC beam-column joint

Figure 9: Failure pattern in beam-column joints

Ductility associated parameters were also calculated, Table 5. FRHPC based beam-column joints were found to exhibit increase in ductility by about 87 and 82 percent in FRHPC beam-column joint corresponding to ZZ 50/6 and SF 50/6 respectively when compared with their companion specimen NFC. Likewise, toughness (or energy absorption capacity) has been found to increase by 302 and 262 percent corresponding to ZZ 40/9 and ZZ 50/9 designated specimens whereas resilience has been found to increase by 488 and 399 percent in specimen designated as ZZ 40/9 and ZZ 50/9 respectively, when compared with their companion control specimen NFC.

4. Conclusion

In summary, it may therefore be inferred that crimped fibers provided in optimum amount of 9 percent by volume with aspect ratio 80 can substitute for conventional transverse reinforcement thereby allowing for relaxation in stirrups congestion which is many times experienced in seismic detailing of beam-column joints. A simplified design equation to determine fiber contents needed to replace stirrups whilst retaining same level of strength and ductility may be developed.

1. The compressive, split tensile and flexural strength values have been increased by 36, 230 and 188 percent respectively in FRHPC based controlled specimens corresponding to ZZ 40/9 when compared with their companion specimen of NFC.
2. The first crack load and the ultimate load have been found to increase by 70 and 72 percent respectively in FRHPC based beam-column joints corresponding to ZZ 40/9 when compared with their companion specimen NFC.
3. The ductility associated parameters like toughness and resilience have been found to increase tremendously by 302 and 488 percent in FRHPC based beam column junctions corresponding to ZZ 40/9.
4. An optimum fiber content has been found to be 9 percent with aspect ratio 80 in case of crimped fibers.
5. Addition of steel fibers significantly improves dimensional stability, strength consistency and integrity of the beam-column joints.
6. The addition of fibers in optimum amount led to significant enhancement in ductility which is particularly significant in earthquake resistant structures.
7. Steel fibers in optimum amount can substitute for conventional transverse reinforcement and thus allow relaxation in stirrups congestion experienced in seismic detailing.

8. The addition of steel fibers to beam-column joint decreases rate of degradation of stiffness as compared to NFC based specimens. Hence, addition of steel fibers in beam-column joints can be considered as an useful solution in case of joints subjected to cyclic loading.

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