International Journal of Science and Research (IJSR)
ISSN: 2319-7064
ResearchGate Impact Factor (2018): 0.28 | SJIF (2018): 7.426

Predicting Flexural Behavior of Fiber Reinforced High Performance Concrete Beams

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Abstract: ***High-performance concrete (HPC) is concrete that has been designed to be more durable and stronger than conventional concrete. HPC mixtures are composed of essentially the same materials as conventional concrete mixtures, but the proportions are designed, or engineered, to provide the strength and durability needed for the structural and environmental requirements of the structures. Fibre Reinforced High Performance Concrete (FRHPC) is developing quickly to a modern structural material with a high potential. The behavior of tensile strain-hardening of FRHPC beams is presented. Though there are many publications proposing stress block models for FRHPC beams, a universally accepted stress block model is yet to be developed. In most design standards, the conventional rectangular stress block developed for Normal Strength Concrete is still being used for design of FRHPC beams. As per IS: 456-2000 for concrete of grades higher than M55, design parameters given in the code of practice may not be applicable and values may be obtained from specialized literatures and experimental results. Whilst there are many publications proposing stress block models for HPC beams, stress block model for FRHPC is yet to be developed. In most design standards, the conventional rectangular stress block developed for Normal Strength Concrete is still being used for design of FRHPC beams. For the preparation of acceptable design recommendations for FRHPC a number of principles should be respected. The code should be consistent with existing deign recommendations for structural concrete. The aim of this study was to study the flexural behavior of FRHPC beams. In this paper, model proposed in European design standard EC: 02-2004 and current Indian Standard IS: 456-2000 have been analysed to compare the experimental and theoretical moment capacities considering actual F_{ck} and F_y values obtained from tests done.Twelve numbers Reinforced Concrete Beams of size 200 x 200 x 2400 mm using concrete mix with three different w/c ratios (0.45, 0.35 and 0.25) were cast for flexural strength test. For flexural strength assessment, the beam was placed in simply supported condition over two fixed steel pedestals to get a clear span of 2000 mm (Figure-1). Keeping in view the specimen size to be tested and failure load, the loading was decided to be applied at the rate of 0.2 mm/minute in displacement control. Two mechanical dial gauges were placed at L/3 distance from both the supports for measurement of deflection. Apart from this one linear variable differential transformer (LVDT) was placed in centre of beam to measure mid-point deflection.***

Keywords: Fibre Reinforced High Performance Concrete, FRHPC, Applications, research, recommendations

1. Introduction

The strength and durability of the concrete used in reinforced concrete structures have been increasingly related to technological developments. The availability and advancement of material technology and the acceptance has led to the production of high performance concrete. High performance concrete offers superior engineering properties i.e. compressive strength, tensile strength, durability, modulus of elasticity and overall better performance when compared to the conventional concrete. Due to its enhanced strength and improved structural properties, high performance concrete has been increasingly used for present day constructions. FRHPC - utilizes randomly oriented discrete steel fibre reinforcement in the mixture and offers a practical way of obtaining these properties for most applications. Research on the behavior of FRHPC beams has been carried out in the past and is still continuing, to understand the behavior of FRHPC beams in flexure. Whilst there are many publications proposing stress block models for FRHPC beams, a universally accepted stress block model is yet to be developed. In most design standards, the conventional rectangular stress block developed for Normal Strength Concrete is still being used for design of FRHPCbeams. Rectangular stress block is generally used to calculate the ultimate moment capacity of reinforced concrete beams. The stress-strain curves for FRHPC are more linear than parabolic and hence it was reasonable to infer that the rectangular stress block parameters could be different.

The rectangular stress block model was first introduced by Hognestad et al (1955) from experimental work involving normal strength concrete. Ashour [16] has shown that the flexural rigidity increases as concrete compressive strength increases. From the experimental study by Oztekin et al [17], it was observed that the rectangular stress block parameters used in ordinary concrete members cannot be used safely for fiber reinforce high performance concrete members. Attard and Stewart (1998) examined the applicability of ACI 318-95 rectangular stress block parameters to FRHPC. They have shown that for a ductile singly-reinforced rectangular section, the ultimate moment capacity is relatively insensitive to the stress block model. An experimental study on the evolution of depth of neutral axis at failure with the ductility at bending on FRHPC beams was carried out by Bernardo & Lopes (2004). It was found that the theoretical formulations based on the use of the rectangular block diagram for the concrete to compute the depth of neutral axis at failure gave substantially smaller values as compared to the experimental values. As such, it was concluded that the rectangular stress block diagram proposed by ACI 318-1989 was not adequate for FRHPC beams.

2. Concrete Ingredients

2.1 Materials

Crushed aggregate with a maximum nominal size of 20 mm was used as coarse aggregate and natural riverbed sand confirming to Zone II as per IS: 383 was used as fine aggregate. Their physical properties are given in Table 1.Thepetrographic studies conducted on coarse aggregate indicated that the aggregate sample is medium grained with a crystalline texture and partially weathered sample of granite. The major mineral constituents were quartz, biotite, plagioclase-feldspar and orthoclase-feldspar. Accessory minerals are calcite, muscovite, tourmaline and iron oxide. The petrographic studies of fine aggregate indicated that the minerals present in order of abundance are quartz, orthoclase-feldspar, hornblende, biotite, muscovite, microcline-feldspar, garnet, plagioclase-feldspar, tourmaline, calcite and iron oxide. For both the coarse aggregate and fine aggregate sample the strained quartz percentage and their Undulatory Extinction Angle (UEA) are within permissible limits. Feldspar grains are partially fractured and shattered. The quality of both coarse and fine aggregate is fair. The silt content in fine aggregate as per wet sieving method is 0.65 percent.

Table 1: Properties of Aggregates

Property		Granite		Fine Aggregate
		20 mm	10 mm	
Specific gravity		2.80	2.80	2.61
Water absorption (%)		0.3	0.3	0.78
Sieve Analysis Cumulative Percentage Passing (%)	20mm	98	100	100
	10 mm	1	68	100
	4.75 mm	0	2	95
	2.36 mm	0	0	87
	1.18 mm	0	0	68
	600 µ	0	0	38
	300 µ	0	0	10
	150 µ	0	0	2
	Pan	0	0	0
Abrasion Value		18	-	-
Crushing Value		18	-	-
Impact Value		12	-	-
Flakiness %		28	-	-
Elongation %		24	-	-

One brand of Ordinary Portland cement (OPC 53 Grade) with fly ash and silica fume are used in this study. The chemical and physical compositions of cement OPC 53 Grade, Properties of flyash and silica fume are given in Table 2. Polycarboxylic group based superplasticizer for w/c ratio 0.20, 0.25, 0.30 and 0.35 and Naphthalene based for w/c ratio 0.45 complying with requirements of Indian Standard: 9103 is used throughout the investigation. Water complying with requirements of IS: 456-2000 for construction purpose was used.The 3 days, 07 days and 28 days compressive strength of cement OPC 53 Grade were 36.00N/mm2, 45.50N/mm2 and 57.50N/mm2 respectively. The 28 days compressive strength of controlled sample and sample cast with flyash was 38.53 N/mm2 and 31.64 N/mm2 respectively, when testing was done in accordance with IS: 1727. The 07 days compressive strength of controlled sample and sample cast with silica fume was 12.76N/mm2 and 14.46N/mm2 respectively, when testing was done in accordance with IS: 1727.

Table 2: Physical, Chemical and Strength Characteristics of Cement

Characteristics		OPC -53 Grade	Silica Fume	Fly Ash
Physical Tests:				
Fineness (m^2/kg)		320.00	22000	403
Soundness Autoclave (%)		00.05	-	-
Soundness Le Chatelier (mm)		1.00	-	-
Setting Time Initial (min.)		170.00	-	-
Setting Time Initial (max.)		220.00	-	-
Specific gravity		3.16	2.24	2.2
Chemical Tests:				
Loss of Ignition (LOI) (%)		1.50	1.16	-
Silica (SiO_2) (%)		20.38	95.02	-
Iron Oxide (Fe_2O_3) (%)		3.96	0.80	-
Aluminium Oxide (Al_2O_3)		4.95	-	-
Calcium Oxide (CaO) (%)		60.73	-	-
Magnesium Oxide (MgO) (%)		4.78	-	-
Sulphate (SO_3) (%)		2.07	-	-
Alkalies (%)	Na_2O	0.57	-	
	K_2O	0.59	-	
Chloride (Cl) (%)		0.04	-	-
IR (%)		1.20	-	-
Moisture (%)		-	0.43	-

2.2 Mix design details

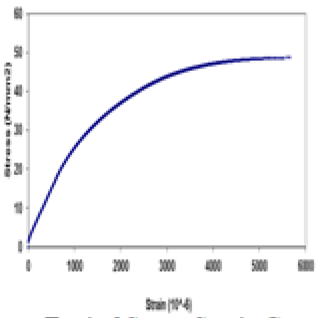
In this study, the five different mixes ranging from w/c ratio 0.45 to 0.30 using granite aggregate were studied for determining short term mechanical properties of FRHPC. For each type of aggregate, three separate batches were prepared. The slump of the fresh concrete was kept in the range of 75-100 mm. A pre-study was carried out to determine the optimum superplasticizer dosage for achieving the desired workability based on the slump cone test as per Indian Standard. The mix design details of specimens are given in Table 3. Adjustment was made in mixing water as a correction for aggregate water absorption. For conducting studies, the concrete mixes were prepared in pan type concrete mixer. Before use, the moulds were properly painted with mineral oil, casting was done in three different layers and each layer was compacted on vibration table to minimize air bubbles and voids. After 24 hours, the specimens were demoulded from their respective moulds. The laboratory conditions of temperature and relative humidity were monitored during the different agesat27±2°C and relative humidity 65% or more.The specimens were taken out from the tank and allowed for surface drying and then tested in saturated surface dried condition.

Table 3: Concrete Mix Design Details for study done

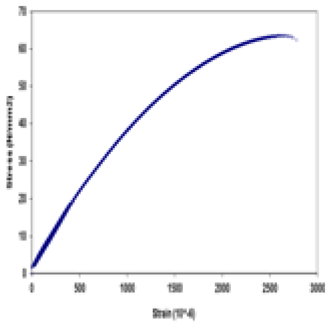
w/c	Total Cementitious Content [Cement C + Flyash (FA) + Silica Fume (SF)] (Kg/m^3)	Water Content (Kg/m^3)	Admixture % by weight of Cement	Steel Fibre (Crimped Shape) Dia - 0.45 mm Length – 36 m) (Kg/m^3)	Fine Aggregate as % of Total Aggregate by weight	28-Days strength of concrete (N/mm^2)
0.45	**362** (290+72+0)	170	1.00	18.10	35	45.72
0.35	**417** (334+83+0)	150	0.45	20.85	39	68.57
0.25	**525** (400+75+50)	140	0.70	26.25	39	88.60

Stress Strain study on FRHPC and Normal Strength Concrete

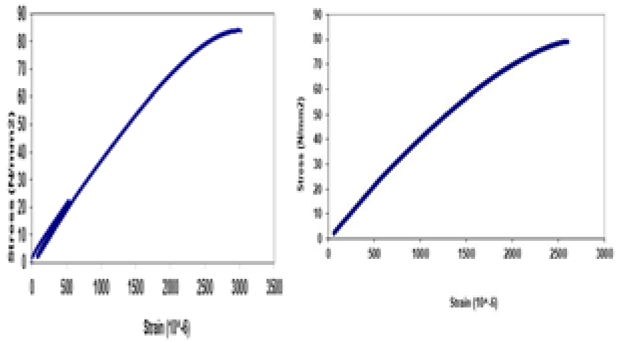
The concrete specimens were tested in a closed-loop servo hydraulic compression testing machine (Make-Controls) of 3000 KN capacity. Two extensometers at the middle half of the height were used to get strain and two strains were averaged. To obtain a full stress-strain curve, a slow rate of loading in the range of 1300 to 1500 N/sec was adopted for a whole compression test. Total 24 specimens were tested to get full stress-strain curves. Due to their brittle characteristics, limited specimens showed the complete ascending and descending branches of stress-strain curve. In general, the normal strength concrete gradually fails after reaching its peak load, but the FRHPC suddenly explodes at peak load. Typical splitting rupture failure was noticed. The typical stress strain curve for M40, M55 and M70 grade concrete is shown below.



Typical Stress Strain Curve for M40 grade



Typical Stress Strain Curve for M55 grade



Typical Stress Strain Curve for M75 grade

A typical relationship between stress and strain for normal strength concrete indicates that after an initial linear portion lasting up to about 30 to 40 percent of the ultimate load, the curve becomes non-linear, with large strains being registered for small increments of stress. The non-linearity is primarily a function of the coalescence of micro-cracks at the paste-aggregate interface. The ultimate stress is reached when a large crack network is formed within the concrete, consisting of the coalesced micro-cracks and the cracks in the cement paste matrix. From the stress strain curves shown above it can be inferred that the stress-strain curves for FRHPC are more linear than parabolic and hence it is reasonable to infer that the rectangular stress block parameters could be different. Past studies have also indicated that the typical stress-strain curve for FRHPC is more linear than parabolic and the ultimate strain is lower for FRHPC. The linear part of the ascending branch stretches to nearly more than 90% of the peak stress of FRHPC whereas lower strength concrete shows negligible linear part. The stress decays very fast in FRHPC after peak stress has been reached.

The stress-strain curve as per both IS: 456-2000 and EC: 2 are given below. The Stress-Strain curve given in IS: 456-2000 is for ordinary and standard concrete up to M55 grade and fixed value of strain at peak stress and ultimate strain is given for design purposes. Likewise, in IS: 456-2000, Eurocode-2 also gives fixed value of strain at peak stress and ultimate strain up to M50 grade concrete for design purposes Whereas, in EC: 2 the Stress-Strain curve is giving different strain values at peak stress above M50 grade. It also gives different ultimate strain for different grade above M50.In IS: 456-2000 stress strain curve for concrete the maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending. This strain limit will not hold good in case of FRHPC above M50. The experimental value also shows that ultimate strain in concrete decreases with increase in strength of concrete. The experimental values for ultimate strain also showed same trend that is shown by Euro Code ultimate strain empirical equation for concrete grades above M50. Therefore, stress-strain curve for FRHPC needs to be revised in IS: 456-2000 for Design of FRHPC. The stress-strain curve of Euro Code EC-02-2004 gives more realistic value of ultimate strain of concrete above M55 to M100.

3. Experimental Study on Reinforced Cement Concrete (RCC) Beams in Flexure

Research on the behavior of FRHPC beams with concrete strength higher than 50MPa has been carried out in the past and is still continuing, to understand the behavior of FRHPC beams in flexure. Whilst there are many publications proposing stress block models for FRHPC beams, a universally accepted stress block model is yet to be developed. In most design standards, the conventional rectangular stress block developed for Normal Strength Concrete (NSC) is still being used for design of FRHPC beams. As established by the past studies that the rectangular stress block theory is applicable only for under-reinforced sections, only those beams that were under-reinforced were considered for the purpose of analysis. The experimental data obtained from testing of beams in flexure have been considered with a view to compare the ultimate strength of beams in bending to the capacity predicted by Euro code EC: 02-2004. For a comparison to be made between the actual moment capacities and theoretical moment capacities, the theoretical moment capacities are based on the same parameters as the actual beams tested. Ten numbers Reinforced Concrete Beams of size 200 x 200 x 2400 mm using concrete mix with three different w/c ratios (0.45, 0.35 and 0.25) were cast for flexural strength test. The design details of beams are given in Table-4.

Table 4: Design Details of Beams

Sl. No.	Concrete Grade	B (mm)	D (mm)	d (mm)	A_{st} (mm2)	Bar Details (HYSD)	pt (%)
1	M45	200	200	165	339	3 Nos. 12 mm	1.03
2	M55	200	200	165	470	2 Nos. 10 mm & 1 Nos. 20 mm	1.09
3	M65	200	200	165	540	2 Nos. 12 mm & 1 Nos. 20 mm	1.64

4	M70	200	200	165	829	2 Nos. 20 mm & 1 Nos. 16 mm	2.13
5	M80	200	200	165	741	2 Nos. 20 mm & 1 Nos. 12 mm	2.24

The concrete mix used in RCC beams were as per mix design details given in Table 4. For flexural strength assessment, Flexural Testing Machine of 500 KN capacity having displacement rate control facility was used. The beam was placed in simply supported condition over two fixed steel pedestals to get a clear span of 2000 mm. The loading setup was made for four points bending by placing a distributor beam over two roller supports at one-third span distance from supports. Keeping in view the specimen size to be tested and failure load, the loading wasdecided to be applied at the rate of 0.2 mm/minute in displacement control (Figure-1).

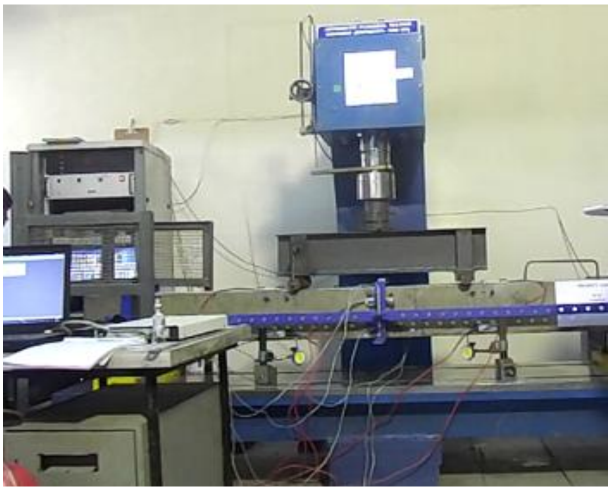
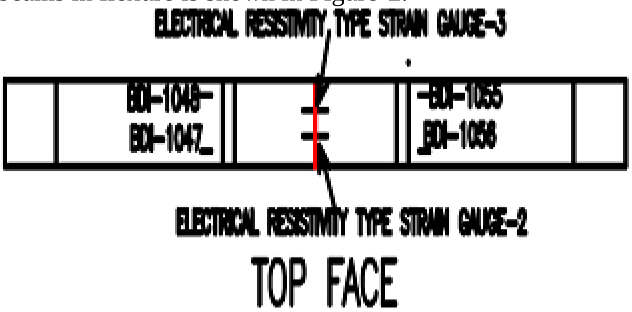


Figure-1: Test set up for Flexural Strength of Reinforced Concrete Beams

The three additional set of concrete cubes were cast and tested at same day on which the testing was performed. The compressive strength obtained from the testing of these cube samples were used for checking the predicted moment as per design codes.

For testing of beams, total eight strain gauges per beam of electrical resistivity type and eight BDI (Bridge Diagnostic Inc.) strain gauges per beam were used. Two mechanical dial gauges were placed at L/3 distance from both the supports for measurement of deflection. Apart from this one LVDT was placed in centre of beam to measure mid-point deflection. The instrumentation scheme for the testing of beams in flexure is shown in Figure-2.



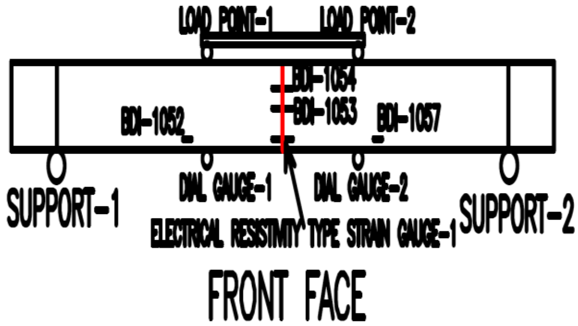


Figure 2: Instrumentation scheme for the testing of RCC beams in flexure

The failure load and comparison of experimental moment to moment predicted as per design procedures of Euro Code, existing IS Code and Singaporean Code is given in Table-5. The use of formulae for calculating the moment capacity of reinforced concrete beams by engineers makes it obvious that the theoretical moment capacity should be less than the actual moment capacity.

As per EC-2 (Euro code):

The limiting value of the ratio of the neutral axis depth at the ultimate limit state to the effective depth, (x/d), is expressed as a function of the ratio of the redistributed moment to the moment before redistribution, δ as follows:

$$\left(\frac{x}{d}\right)_{\lim} = \frac{\delta - k_1}{k_2} \; for \; f_{ck} \le 50MPa \quad \text{(EC2 5.5(4))}$$

$$\left(\frac{x}{d}\right)_{\lim} = \frac{\delta - k_3}{k_4} \; for \; f_{ck} > 50MPa \quad \text{(EC2 5.5(4))}$$

For reinforcement with $f_{ck} \le 500$ MPa, the following values are used:

$k_1 = 0.44$ (EC 5.5(4))

$k_2 = 1.25(0.6 + 0.0014/\varepsilon_{cu2})$ (EC 5.5(4))

$k_3 = 0.54$ (EC 5.5(4))

δ is assumed to be 1

Clause 6.5.1.3 EC: 02-2004 Minimum and Maximum Reinforcement

The minimum flexural tension reinforcement required in a beam section is given by the maximum of the following two limits:

$$A_{s,\min} = 0.26\frac{f_{ctm}}{f_{yk}}bd \quad \text{(EC2 9.2.1(1))}$$

$$A_{s,\min} = 0.0013bd \quad \text{(EC2 9.2.1(1))}$$

where f_{ctm} is the mean value of axial tensile strength of the concrete and is computed as:

$f_{ctm} = 0.30 f_{ck}^{(2/3)}$ for $f_{ck} \le 50MPa$ (EC2 3.12, Table 3.1)

$f_{ctm} = 2.121n(1 + f_{cm}/10)$ for $f_{ck} > 50MPa$ (EC2 3.12, Table 3.1)

$f_{ctm} = f_{ck} + 8MPa$ (EC2 3.12, Table 3.1)

The minimum flexural tension reinforcement required for control of cracking should be investigated independently by the user. As upper limit on the tension reinforcement and compression reinforcement has been imposed to be 0.04 times the gross cross-sectional area (EC 9.2.1.1(3))

As per IS 456

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam

d = effective depth, and

f_y = characteristic strength of reinforcement in N/mm^2

Maximum reinforcement – The maximum area of tension reinforcement shall not exceed 0.4 bD.

Clause 26.5.1.2 of IS:456-2000 Compression Reinforcement

The maximum area of compression reinforcement shall not exceed 0.04 bD. Compression reinforcement in beams shall be enclosed by stirrups for effective lateral restraint. The arrangement of stirrups shall be as specified in 26.5.3.2

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This paper presents the detailed illustration on (a) IS Approach (extension to higher grades), (b) Euro Code approach and (c) mixed approach (using IS Code Equations along with incorporation of strain values from Euro Code). The detailed calculation of theoretical and experimental moments using IS Code current formulae, EC-02-2004 code and mixed approach (using IS Code Equations along with incorporation of strain values from Euro Code) is given below. In the mixed approach, equation of IS code was used considering limiting strain values of Euro Code which was verified experimentally.

The design methodology based on the idealized short-term (uniaxial) stress-strain diagram, accepted in the Codes for normal strength concrete, can generally be safely applied to higher strength concrete. However, the stress-strain diagrams, may need minor modifications to allow for the lower ultimate strain of FRHPC. With a reduction in ultimate strain comes a reduction in ductility. One way to enhance ductile behavior is through the provision of longitudinal and transverse reinforcement. Even if only nominal amount is needed, some should be provided in the compression zone. In beams, the requirements for minimum shear reinforcement should in most cases result in providing an adequate amount of confinement in the compression zone. For slabs, the amount of reinforcement in the compression zone should not be less than the minimum amount of reinforcement required in the tension zone. High strength precast floor units, such as hollow-core units, should not usually be used without a structural concrete topping having the required minimum amount of reinforcement, unless sufficient top steel is provided in the unit itself.

From the design analysis, it is seen that the calculated moments are slightly lesser in-case of IS code when compared to Euro Code. From the test conducted, it is seen that the same stress-strain curve as provided in IS: 456-2000 can be adopted using permissible strain values given in EC: 02-2004. The approach works out to be conservative, convenient and easy to understand. The results obtained theoretically for the calculation of ultimate strength must be conservative. The design rules should provide similar level of conservativeness for normal and FRHPC.

Typical Failure Mode of RCC Beams

The beams failed due to widening and extending of flexural cracks into compression zone and crushing of concrete in the compression zone, between the loading points (Figure-3 & 4). No shear cracks in the shear zone and also no damage at the anchorage zone of beam is observed.

Figure 3: Concrete crushing at top at ultimate load

Figure 4: Close-up view of concrete crushing at top of the beam at ultimate load

Table 5: Comparison of Experimental Moment to Moment predicted by Euro Code

S. No	Concrete Grade (fck) (MPa)	Code	Design Load	Max strain		Neutral Axis		Area of Steel		Balanced Moment (KN-m)		Me (Moment as per Experimental Results)	Me/Mp
				Steel	Concrete	Balanced	Actual	Balanced	Actual	Balanced	Actual		
1	Class 45-55	IS: 456:2000	63.70	0.004175	0.0035	0.46d	0.219d	712 (2.09)	339	42.47	22.76	35.70	1.56
	Class 45-55	Euro code EC:2:2004	76.89	0.0045	0.0035	0.448d	0.179d	844 (2.48)	339	51.26	23.26		1.53
2	Class 50-60	IS 456:2000	71.80	0.004175	0.0035	0.46d	0.258d	802 (2.36)	470	47.87	30.69	46.77	1.52
	Class 50-60	Euro code EC:2-2004	89.94	0.0045	0.0035	0.448d	0.224d	938 (2.76)	470	59.96	31.64		1.47
3	Class 55-67	IS 456:2000	77.61	0.004175	0.0035	0.46d	0.286d	867 (2.55)	540	51.74	35.13	45.56	1.29
	Class 55-67	Euro code EC:2-2004	74.02	0.0045	0.0031	0.35d	0.245d	774 (2.27)	540	49.35	36.08		1.26
	Class 55-67	Mixed Approach	73.14	0.004175	0.0031	0.426d	0.286	803 (2.36)	540	48.76	35.13		1.29
4	Class 60-75	IS 456:2000	86.80	0.004175	0.0035	0.46d	0.393d	970 (2.85)	829	57.87	51.19	64.68	1.26
	Class 60-75	Euro code EC:2-2004	84.18	0.0045	0.0029	0.41d	0.358d	902 (2.65)	829	56.12	52.79		1.22
	Class 60-75	Mixed Approach	79.29	0.004175	0.0029	0.409d	0.393d	863 (2.54)	829	52.86	51.19		1.26
5	Class 70-85	IS 456:2000	98.44	0.004175	0.0035	0.46d	0.309d	1100 (3.23)	741	65.63	47.66	70.00	1.46
	Class 70-85	Euro code EC:2-2004	95.56	0.0045	0.0027	0.412d	0.299d	1019 (2.99)	741	63.71	48.64		1.43
	Class 70-85	Mixed Approach	87.06	0.004175	0.0027	0.393d	0.309d	940 (2.76)	741	58.04	47.66		1.46

4. Conclusion

The following conclusions are drawn based on the findings for the materials used and tests performed in this study:

1) In IS: 456-2000 stress strain curve for concrete the maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending. This strain limit will not hold good in case of FRHPC above M50. Based on trend shown by Experimental Results the ultimate strain value given in Euro Code seems to be more realistic.
2) From the stress-strain study on FRHPC, it is seen that the typical stress-strain curve for FRHPC is more linear than parabolic and the ultimate strain is lower for FRHPC. The linear part of the ascending branch stretches to nearly more than 90% of the peak stress of FRHPC whereas lower strength concrete shows negligible linear part. The stress decays very fast in FRHPC after peak stress has been reached
3) From the design analysis, it is seen that the calculated moments are slightly lesser in-case of IS code when compared to Euro Code. From the test conducted, it is seen that the same stress-strain curve as provided in IS: 456-2000 can be adopted using permissible strain values given in EC: 02-2004. The approach works out to be conservative, convenient and easy to understand. The results obtained theoretically for the calculation of ultimate strength must be conservative. The design rules should provide similar level of conservativeness for normal and FRHPC.

References

[1] Collins, M.P., "Reinforced Concrete in Combined Shear and Flexure", Proceedings of the Mark W. Huggins Symposium, University of Toronto, Sept. 1978, pp. 154-171.
[2] Carrasquillo, R.L., Nilson, A.H., Slate, F.O., "Properties of FRHPC Subject to Short- Term Loads", AC1 Structural Journal, V.78, No. 3, May-June 1981, pp. 171-178.
[3] Alendar, M.G., 2001. An Experimental critique of the BS 8110 Method of estimating concrete elastic modulus, Magazine of Concrete Research, 43: 291-303
[4] Ozturan, T., and Cecen, C., (1997), Effect of Coarse Aggregate Type on Mechanical Properties of Concretes with Different Strength, Cement and Concrete Research, Vol.27, Issue 2, pp 165-170.
[5] Beeby, A.W. and R.S. Narayanan, 1995. Designers Handbook to Euro-code 2 Part1.1: Design of concrete Structures, Thomas Telford Services Ltd, London
[6] Cetin, A. and Carrasquillo, R. L. 1998. High-Performance Concrete: Influence of Coarse Aggregates on Mechanical Properties, Journal of ACI Materials, 95: 252-261.
[7] Ashour, S. A., "Effect of compressive strength and tensile reinforcement ratio on flexural behavior of high-strength concrete beams," Engineering Structures, Vol.22, 2000, pp. 413-423.
[8] Oztekin, E., Pul, S., and Husem, M., "Determination of rectangular stress block parameters for high performance concrete," Engineering Structures, Vol. 25, 2003, pp. 371-376

[9] HELLAND, S.; HOFF, A.; AND EINSTABLAND, T., 1983, "HoyfastBetong," ("High-Strength Concrete,") presented at NorskBetongdag, Trondheim, Norway