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# An experimental investigation on flexural behaviour of fibre-reinforced pond ash-modified concrete

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**KEYWORDS**

Steel fibre;
Pond ash;
SFRC beams;
Ductility;
Crack width

**Abstract** The flexural behaviour of plain and fibre-reinforced pond ash concrete (FRC) beams under monotonic loading condition was analysed. Sixteen beams reinforced with top and bottom longitudinal deformed steel bars and transverse steel stirrups were tested. The beams were cast using three different percentages of pond ash, namely, 10%, 20% and 30% by weight of cement. Grooved type steel fibres were incorporated at different percentages of 0.5%, 1% and 2% by volume of concrete. Beams of cross section 150 mm × 150 mm and length 700 mm were tested in flexure under three-point bending system (one loading point plus two simple supports). Addition of fibres increased the failure load of the beams and ensured ductile behaviour. Ductility index and flexural rigidity of the beams were also studied. The predicted crack width (ACI 224 R-01) was compared with the measured crack width, and a good correlation was obtained.


## 1. Introduction

Pond ash is a combination of unused fly ash and bottom ash from thermal power plants, mixed in slurry form and deposited in ponds [1]. Research showed that fibres bridge cracks in a structural member such as a beam, thus increasing the ductility of the member [2,3]. The bridging action of the fibres is related to pull-out resistance, resulting in diffused cracks in the members. The addition of fibres increases the bearing capacity of the beams and ensures more ductile behaviour. Moreover, the presence of fibres reduces the cover spalling process [4]. An experimental study was conducted on carbon fibre-reinforced polymer concrete specimens [5], and based on the rigid joint model, it was concluded that the crack tip deformation played an important role in accurately characterising the mixed mode fracture toughness of interfaces bonded by hybrid material.

It has been reported in [6] that by using glass fibre-reinforced polymer rods as the main reinforcement for concrete beams, reasonable flexural strength can be achieved. The strengthening of reinforced concrete structures with externally bonded fibre-reinforced polymer laminates has shown excellent performance [7].

An equation for predicting the shear strength of steel fibre-reinforced concrete (SFRC) beams has been developed based on the existing experimental results. A large database containing 222 shear strength tests of SFRC beams without stirrups

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was divided into six different groups based on their span-depth ratio, concrete compressive strength and steel fibre shapes (hooked, crimped and plain). The proposed equations were obtained by performing both linear and non-linear regression analyses on each database [8]. The strengthening effects of hybrid FRPs on ductility and stiffness of RC beams depend on the placement of FRP layers [9].

From the experimental results and from the results of non-linear analysis on glass fibre-reinforced polymer concrete beams, a distinct unloading process was found as a consequence of sudden change of flexural stiffness [10]. The effect of strength of concrete and reinforcement on toughness of beams was studied. It was observed that the most influential parameter in the energy absorption capacity of the beams was the amount of steel reinforcement [11].

The shear capacity of concrete beams reinforced with fibre-reinforced polymers was investigated. Steel stirrups were used as shear reinforcement in all beams. Based on this investigation, a simplified expression for the shear capacity of FRP reinforced concrete members was introduced [12]. The flexural fatigue of self-compacting fibre-reinforced concrete was studied. The two-million cycle fatigue strength of SCFRC has been found to be higher than that of normally vibrated fibre-reinforced concrete [13]. It was found that the first-crack strength and the whole post-cracking behaviour were mainly influenced by the amount of fibres [14].

Addition of fibres delayed the initiation of flexural cracks and decreased the crack width [15]. The compressive strength of concrete increased proportionately with the increase in volume ratios of propylene fibres [16]. Steel fibres improve the concrete quality and the post-crack performance and reduce the brittle behaviour of normal concrete and high strength concrete [17].

Addition of short fibres increases the tensile strength of the matrix approximately by 40% when water binder ratio (w/b) of 0.3 was used in concrete mix [18]. At a high fibre dosage, segregation problems arise in the fresh mixture and its detrimental effects on workability are very limited [19]. The effect of palm oil fuel ash in concrete beams has been studied and is reported that for all mixtures, there is an increase in water-binder ratio which reduces the first cracking strength and flexural strength [20].

This paper presents an experimental investigation on flexural behaviour of fibre-reinforced pond ash-modified concrete. Concrete beams of pond ash-modified concrete provided with nominal tensile reinforcement and fibre reinforcement were tested. Flexural behaviour and crack development were studied. Though many researchers studied the flexural behaviour of ash-modified beams and steel fibre beams, so far no study has been made to highlight the flexural behaviour of beams of pond ash-modified concrete reinforced by steel fibre. Thus, this paper focuses on a hitherto unexplored research area.

### *1.1. Objective of the study*

The objective of the paper was to study flexural behaviour of fibre-reinforced pond ash-modified concrete. Laboratory scale beams were cast and tested under three-point bending system (one loading point plus two simple supports) to study the yield load, ultimate load, first cracking load, failure modes, crack patterns, ductility index and flexural rigidity.

## 2. Experimental programme

### *2.1. Test materials*

#### *2.1.1. Cement*

Ordinary Portland 53 grade cement was used for making the concrete specimens. The specific gravity of the cement was found to be 3.15. The initial setting time and the final setting time of the cement were found to be 140 min and 245 min respectively [21]. The chemical compositions of the cement and the pond ash are shown in Table 1.

#### *2.1.2. Pond ash*

The pond ash used in the test programme was obtained from Mettur-Thermal power station, TN, India. The specific gravity of the pond ash was found to be 2.04. The SEM images and the EDAX images of the ash are shown in Figs. 1 and 2 respectively. Pond ash content was varied as 0%, 10%, 20% and 30% by the weight of cement.

#### *2.1.3. Aggregate*

Aggregates of size ranging from 20 mm and 12 mm were used in this work. The specific gravity of the coarse aggregates was found to be 2.78, and the water absorption of the coarse aggregates was 0.5%. The specific gravity of the fine aggregates was 2.60 and its water absorption was 1.02% [22–24].

#### *2.1.4. Fibre reinforcement*

Discrete steel fibres conforming to ASTM A 820/A 820M-04 were used [25]. They were Type 1 cold-drawn, wire-grooved shown in Fig. 3. The steel fibres had a length ($l_f$) of 50 mm and a diameter ($d_f$) of 1 mm. Hence, their aspect ratio was 50. The tensile strength of the fibre was found to be 1098 MPa using tensometer. Fibre content was varied as 0%, 0.5%, 1% and 2% by volume of concrete.

#### *2.1.5. Composition and preparation of mixtures*

Workability measurements based on the slump value were carried out on fresh fibre-reinforced pond ash concrete. The mix designation and the results of the slump values are presented in Table 2. Fresh concretes containing 10%, 20% and 30% pond ash as cement replacement in mass basis were prepared by modifying the reference Portland cement concrete. Similarly, fresh fibre reinforced concretes containing 0.5%, 1.0% and 2.0% steel fibre in volume basis were also prepared. Inclusion of the steel fibres reduced workability. Increase in the steel fibre content caused additional reduction in workability. However, increase in the pond ash content increased workability.

For each cubic metre of concrete, w/c + pond ash ratio was determined considering the pond ash content, and the mix was prepared accordingly. Table 2 shows the w/c + pond ash ratios for different ash contents. The specimens were named as A1, A2, etc. to indicate different pond ash contents and fibre contents (Table 2). Mixture design was made in accordance with the Indian Standard Code 10262-2009 [26]. Table 3 shows the mix proportions of all the specimens. The procedures for mixing the fibre-reinforced concrete involved the following steps:

(i) Gravel and sand were placed in a concrete mixer and dry mixed for 1 min; (ii) then, cement and fibre were spread and

**Table 1** Chemical composition of cement and pond ash (%).

| Parameter | $SiO_2$ | $Al_2O_3$ | $Fe_2O_3$ | CaO | MgO | $SO_3$ | $K_2O$ | $Na_2O$ | Loss of ignition |
|---|---|---|---|---|---|---|---|---|---|
| Cement | 19.94 | 5.15 | 3.38 | 63.37 | 1.58 | 1.93 | 0.90 | 0.24 | 0.98 |
| Pond ash | 54.46 | 33.11 | 1.76 | 7.97 | 0.35 | 1.20 | 0.00 | 0.00 | 2.16 |

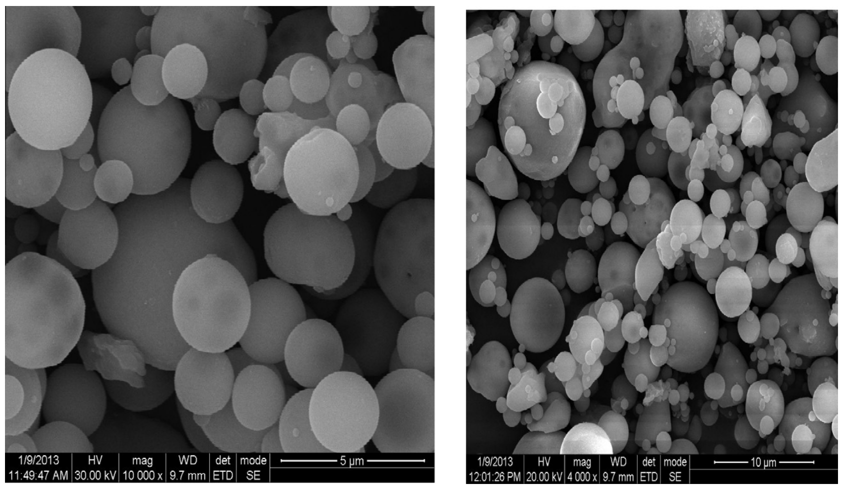
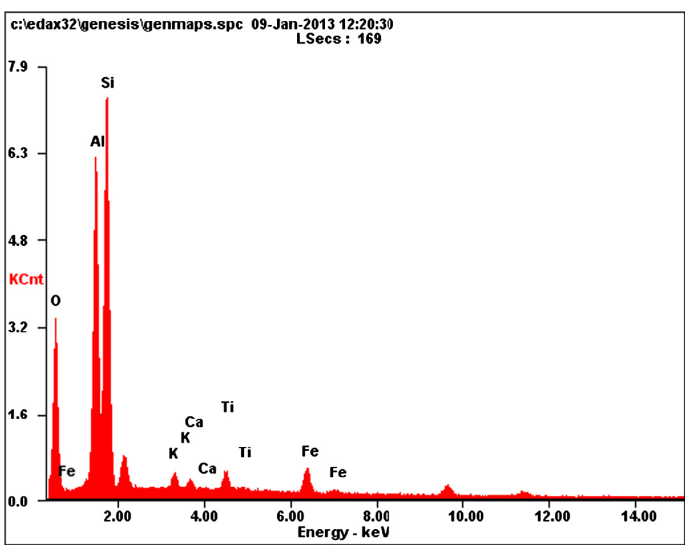


**Figure 1** SEM images of pond ash.




| Element | Wt (%) | At (%) |
|---|---|---|
| OK | 43.71 | 58.37 |
| AlK | 19.73 | 15.63 |
| SiK | 30.91 | 23.52 |
| KK | 01.01 | 00.55 |
| CaK | 00.51 | 00.27 |
| TiK | 01.34 | 00.60 |
| FeK | 02.79 | 01.07 |
| Matrix | Correction | ZAF |

**Figure 2** EDAX image of pond ash.

dry mixed for 1 min; (iii) after that, mixing water was added and mixed for approximately 2 min; (iv) and finally, the freshly mixed fibre-reinforced concrete was cast into specimen moulds and vibrated to remove entrapped air. After casting, each of the specimens was allowed to stand for 24 h before demoulding. Demoulded specimens were stored in water at 23 ± 2 °C until testing days.

### 2.1.6. Tests conducted

Fundamental properties of concrete were determined from compressive strength test, split tensile test and elastic modulus tests. Cubes of size 100 mm × 100 mm × 100 mm were cast for all series to perform compressive strength. The compressive strength of the specimens was determined according to the IS: 516-1959 [27]. The cube specimens were placed under compression testing machine to find compressive strength of the specimen. Cylinders having diameter of 100 mm and height 200 mm were also cast for each series to perform cylindrical splitting tests. The split tensile strength of cylindrical specimens was determined as per ASTM C496 [28]. Cylinders were placed under compression testing machine and the load at which the specimen fails was noted to calculate the splitting tensile strength. Cylinders having diameter of 100 mm and

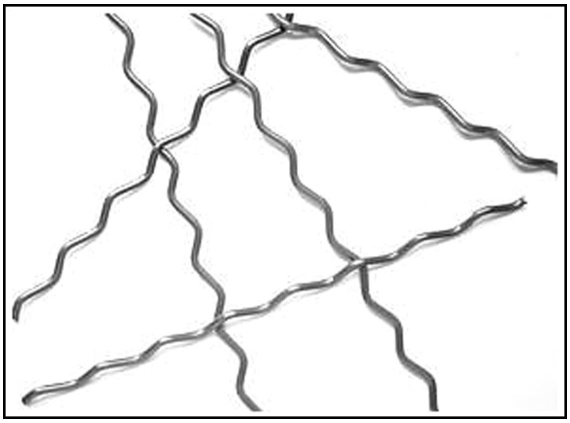


**Figure 3** Grooved steel fibres.

height 300 mm were also cast for each series to elastic modulus of concrete. The test was performed as per ASTM C469-02 [29]. Extensometers were fixed to the specimens to calculate the strains. Table 4 shows the results of the above mentioned tests.

A total of 16 small beams of cross section 150 mm × 150 mm and length 750 mm were cast and tested in flexure under a universal testing machine of capacity 100 tons. The beams were reinforced with two numbers of 16 mm ∅ bar in the top and bottom portion. Stirrups of 6 mm ∅ bars were provided at a spacing of 50 mm. Fig. 4 is a photograph of steel reinforcement. The cover of beam was maintained 20 mm for all the specimens. The beams were cast using cement replaced by pond ash and addition of steel fibres of varying its per cent. Deflectometers were used to measure the deflections of the beam, one placed on the mid-span of the beam and other two in the support on the upper portion of a beam at a distance of 25 mm from the end of the beam. Deflections were measured at an interval of every 10 kN and it was continued until the failure of the beam occurs. First cracking load, yield load, ultimate load and their corresponding deflections were recorded (Table 5).

**Table 2** Mix variables.

| Mixture no. | Pond ash content (%) | Steel fibre content (%) | Slump value (mm) | w/(c + pond ash) ratio |
|---|---|---|---|---|
| A1 | 0 | 0 | 28 | 0.45 |
| A2 | 0 | 0.5 | 27 | |
| A3 | 0 | 1 | 26 | |
| A4 | 0 | 2 | 25 | |
| B1 | 10 | 0 | 29 | 0.46 |
| B2 | 10 | 0.5 | 28 | |
| B3 | 10 | 1 | 27 | |
| B4 | 10 | 2 | 27.5 | |
| C1 | 20 | 0 | 30 | 0.47 |
| C2 | 20 | 0.5 | 29 | |
| C3 | 20 | 1 | 28.5 | |
| C4 | 20 | 2 | 28 | |
| D1 | 30 | 0 | 32 | 0.50 |
| D2 | 30 | 0.5 | 31 | |
| D3 | 30 | 1 | 29.5 | |
| D4 | 30 | 2 | 29 | |

**Table 3** Mix proportions.

| Ingredients | Cement (kg/m$^3$) | Pond ash (kg/m$^3$) | Fine aggregate (kg/m$^3$) | Coarse aggregate | |
|---|---|---|---|---|---|
| | | | | 12 mm (kg/m$^3$) | 20 mm (kg/m$^3$) |
| 0% pond ash | 413.33 | – | 671.84 | 410 | 762 |
| 10% pond ash | 372.00 | 41.33 | 667.88 | 407 | 758 |
| 20% pond ash | 330.67 | 82.66 | 652.08 | 397 | 740 |
| 30% pond ash | 289.34 | 124.00 | 653.06 | 398 | 738 |

### 2.1.7. Theoretical ultimate load

Theoretical ultimate load for reinforced beams was calculated as per the Indian Standard Code 456: 2000 [30]. The parallel ACI code is ACI Committee 544-4R-88:1999 [31]. The ultimate strength of steel fibre reinforced concrete beams was calculated as per the procedure suggested by Henager and Doherty (1976) with modifications. From the stress block shown in Fig. 5, the total compressive force in concrete ($C_{uc}$) and the total compressive force in steel ($C_{us}$) are written as,

$$C_{uc} = 0.36 f_{ck} b X_u \tag{1}$$

and

$$C_{us} = (f_{sc} - 0.416 f_{ck}) A_{sc} \tag{2}$$

where $f_{ck}$ is the mean cube compressive strength of fibrous concrete, $b$ is the width of the beam, $X_u$ is the depth of neutral axis, $f_{sc}$ is the stress in compression steel and $A_{sc}$ is the area of compressive reinforcement.

Referring to Fig. 5, the total tensile force ($T$) is given by $0.446\ f_{ck}$. $T_{fc}$ is the tensile force of fibrous concrete and $T_s$ is the tensile force of bar reinforcement. The depths of rectangular portion and parabolic portion of stress block are equal to $(3X_u/7)$ and $4X_u/7)$.

Hence,

$$T = 0.87 f_y A_{st} + \sigma_t b(D - X_u) \tag{3}$$

where $f_y$ is the yield strength of reinforcing bar, $A_{st}$ is the area of tensile reinforcement, $\sigma_t$ is the tensile stress in fibrous

**Table 4** Test results.

| Mixture no. | Compressive strength (N/mm$^2$) | Elastic modulus (GPa) | Split tensile strength (N/mm$^2$) |
|---|---|---|---|
| A1 | 38.10 | 18.47 | 2.50 |
| A2 | 35.52 | 14.51 | 2.60 |
| A3 | 39.70 | 14.51 | 2.72 |
| A4 | 39.92 | 16.95 | 2.79 |
| B1 | 37.50 | 17.68 | 2.48 |
| B2 | 36.40 | 15.36 | 2.51 |
| B3 | 38.20 | 19.10 | 2.61 |
| B4 | 38.80 | 14.22 | 2.63 |
| C1 | 36.44 | 18.35 | 2.31 |
| C2 | 37.60 | 20.70 | 2.51 |
| C3 | 37.86 | 19.37 | 2.61 |
| C4 | 38.90 | 15.64 | 2.71 |
| D1 | 35.40 | 12.54 | 2.18 |
| D2 | 35.60 | 12.10 | 2.43 |
| D3 | 36.50 | 13.80 | 2.55 |
| D4 | 37.34 | 14.12 | 2.65 |

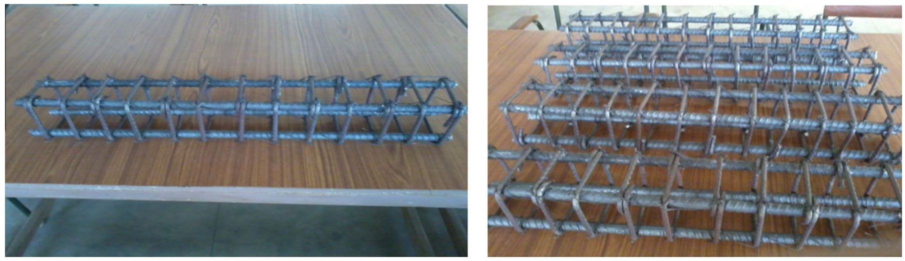


**Figure 4** Steel reinforcement.

**Table 5** Test results in flexure and ductility index.

| S.no | Beam ID | First cracking load ($P_{cr}$, kN) | Yield load ($P_y$, kN) | Deflection on ultimate load ($\delta_y$) | Ultimate load ($P_u$, kN) | Deflection on ultimate load ($\delta_u$) | Ductility index ($\mu$) |
|---|---|---|---|---|---|---|---|
| 1 | A1 | 50.00 | 81.20 | 0.90 | 88.20 | 1.05 | 1.16 |
| 2 | A2 | 62.20 | 82.40 | 2.33 | 90.40 | 2.76 | 1.18 |
| 3 | A3 | 78.40 | 86.60 | 2.35 | 92.60 | 2.89 | 1.22 |
| 4 | A4 | 80.40 | 120.80 | 2.34 | 131.80 | 2.92 | 1.24 |
| 5 | B1 | 53.00 | 95.30 | 1.43 | 103.00 | 1.57 | 1.09 |
| 6 | B2 | 63.40 | 102.20 | 1.98 | 110.20 | 2.20 | 1.11 |
| 7 | B3 | 72.40 | 123.40 | 1.78 | 130.40 | 2.41 | 1.35 |
| 8 | B4 | 77.80 | 128.60 | 1.82 | 133.60 | 2.65 | 1.45 |
| 9 | C1 | 56.60 | 117.80 | 1.26 | 122.80 | 1.55 | 1.23 |
| 10 | C2 | 61.60 | 121.00 | 1.19 | 129.00 | 1.60 | 1.34 |
| 11 | C3 | 74.40 | 127.40 | 1.09 | 132.40 | 1.61 | 1.47 |
| 12 | C4 | 77.60 | 130.60 | 1.27 | 135.60 | 1.95 | 1.53 |
| 13 | D1 | 54.20 | 118.20 | 2.32 | 124.20 | 2.4 | 1.03 |
| 14 | D2 | 62.40 | 124.80 | 2.32 | 131.80 | 2.65 | 1.14 |
| 15 | D3 | 73.00 | 127.40 | 1.74 | 132.40 | 2.20 | 1.26 |
| 16 | D4 | 76.20 | 132.20 | 1.40 | 137.20 | 1.90 | 1.35 |

concrete and $D$ is the total depth of the beam. $\sigma_t$ is calculated as per ACI 544-4R [31] as

$$\sigma_t = 0.00772(l/d_f)\rho_f F_{be} \tag{4}$$

where $(l/d_f)$ is the aspect ratio of steel fibres, $\rho_f$ is the per cent by volume of steel fibres and $F_{be}$ is the bond efficiency of the fibre, which varies from 1.0 to 1.2. In this work, $F_{be}$ value of 1.2 was taken for analysis of beams.

Equating the total compressive force and total tensile force, the depth of neutral axis can be obtained. The moment of resistance ($M_R$) of beam is computed from the following equation:

$$M_R = 0.87 f_y A_{st}(d - 0.416X_u) + \sigma_t b(D - X_u)[(D + X_u)/2) - 0.416\, X_u] \tag{5}$$

Theoretical ultimate loads for SFRC beams were calculated using the above equation. First cracking load, yield load, ultimate load and the corresponding deflections are given in Table 5. Calculated ultimate loads were then compared with the experimental values given in Table 6. The ratios between theoretical ultimate loads and experimental loads were calculated and are presented in Table 6.

### 2.1.8. Flexure tests

The tests were performed using the Universal Testing Machine (UTM). Fig. 6 shows the experimental set-up. Figs. 7 and 8 show the formation of cracks and failure of beams respectively.

## 3. Results and discussion

### 3.1. Compressive strength

For a pond ash content of 20%, compressive strength increased from 36.44 N/mm$^2$ to 38.90 N/mm$^2$ when the fibre content increased from 0% to 2% for a curing period of 28 days. Since the pozzolanic reaction proceeds slowly, the initial strength of ash-modified concrete would be lower than that of plain concrete. Due to continued pozzolanic action, the strength of ash-modified concrete increases with curing period and may even exceed the strength of plain concrete. There may be some variation observed in compressive strength of fibre-reinforced concrete, which was mainly due to a non-homogeneous distribution of the steel fibres within the concrete.

### 3.2. Split tensile strength

The increase in split tensile strength was about 12% when fibre content increased from 0% to 2% for plain concrete for a curing period of 28 days. When the fibre content increased from 0% to 2% for a pond ash content of 20%, the average split tensile strength increased by about 17%. Further, for a pond ash content of 30%, the average split tensile strength increased to 22% when the fibre content increased from 0% to 2%. Addition of steel fibres to concrete contributes to increase in tensile strength.

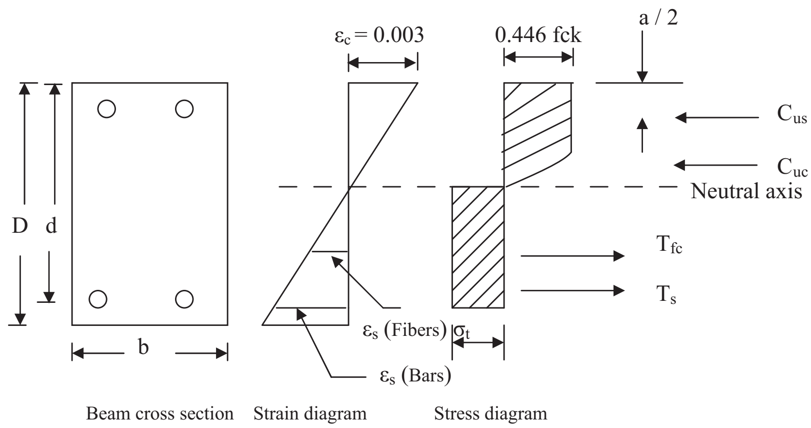



**Figure 5** Stress strain diagram of beam section.

**Table 6** Ratio of test results in flexure.

| S.no | Beam ID | Load (theoretical) $(P_u)_{\mathrm{Th}}$ (kN) | Ultimate load (experimental) $(P_u)_{\mathrm{Exp}}$ (kN) | $\frac{(P_u)_{\mathrm{Th}}}{(P_u)_{\mathrm{Exp}}}$ |
|---|---|---|---|---|
| 1 | A1 | 59.88 | 88.20 | 1.47 |
| 2 | B1 | | 103.00 | 1.72 |
| 3 | C1 | | 122.80 | 2.05 |
| 4 | D1 | | 124.20 | 2.07 |
| 5 | A2 | 68.03 | 90.40 | 1.32 |
| 6 | B2 | | 110.20 | 1.61 |
| 7 | C2 | | 129.00 | 1.89 |
| 8 | D2 | | 131.20 | 1.92 |
| 9 | A3 | 88.07 | 92.60 | 1.05 |
| 10 | B3 | | 130.40 | 1.48 |
| 11 | C3 | | 132.40 | 1.50 |
| 12 | D3 | | 132.40 | 1.50 |
| 13 | A4 | 122.00 | 131.80 | 1.08 |
| 14 | B4 | | 133.60 | 1.09 |
| 15 | C4 | | 135.60 | 1.11 |
| 16 | D4 | | 137.20 | 1.12 |

**Figure 6** Testing of beam under universal testing machine.


**Figure 7** Formation of cracks for A4 beam specimen.

**Figure 8** Failure pattern of beam specimen.

### 3.3. Modulus of elasticity

The modulus of elasticity decreased by 6%, when fibre content increased from 0% to 2% for plain concrete. Further, for a pond ash content of 10% and 20%, the modulus of elasticity reduced by 24% and 17% respectively when fibre content increased from 0% to 2%. There was a marginal increase of about 12% for a pond ash content of 30%, when the fibre content increased from 0% to 30%. But it cannot be concluded that modulus of elasticity increased when pond ash content increased, since the elastic modulus decreased by 5%, 0.7% and 47% for pond ash contents of 10%, 20% and 30% when compared with 0% fibre content concrete specimens. From the data on elastic modulus, it can be concluded that 20% pond ash can be effectively used as a replacement of cement in concrete.

### 3.4. Flexural behaviour of beam specimens

Fig. 9 shows the load–deflection curves of reinforced concrete beams without pond ash. It shows that, due to the presence of steel fibres, flexural rupture characterised by the yielding of longitudinal steel occurs. When 2% steel fibres were added, the ultimate load increased when compared to the case of 0% steel fibres. The ultimate load of beams with 0% pond ash + 2% steel fibre increased by 49% when compared with that of beams with 0% pond ash + 0% steel fibre. Initially flexural cracks were observed when the load in flexure increased; further, the propagation of diagonal cracks and shear failure were also observed.

When a pond ash content of 10% was added to 2% steel fibre beams, the ultimate load increased by 30% when compared with that of 10% pond ash beams of 0% steel fibre. When 10% pond ash was added as a replacement of cement, the interfacial bond increased between the particles. The ultimate load of 10% pond ash reinforced beam with 0% steel fibre increased when compared with that of conventional beam (Fig. 10). When 2% steel fibres were added in the 10% pond ash concrete beam, the ultimate load of the beam increased when compared to 0% steel fibres.

At 20% pond ash, the ultimate load of the 0% steel fibre beam increased in comparison with 0% pond ash and 10% pond ash beams with 0% steel fibre. In beams with 20% pond ash + 1% steel fibre, the increment was about 43% when compared with that of beams with 0% pond ash + 1% steel fibre (Fig. 11).

When the pond ash content increased to 30% in 0% steel fibre beam, the ultimate load increased by 41% when compared with 0% pond ash and 0% steel fibre beam. When pond ash content increased from 20% to 30% in 0.5% steel fibre beam, the ultimate load increased by 2%. For 30% pond ash content, when the fibre content increased to 1% and 2%, the increase in the ultimate load was about 43% and 4% respectively when compared with 0% pond ash and 0% steel fibre beam specimens (Fig. 12).

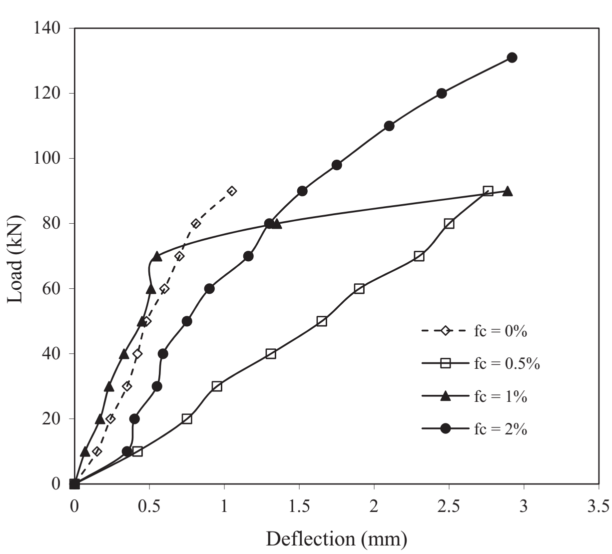



**Figure 9** Load vs mid-span deflection of fibre reinforced concrete beam (0% pond ash).

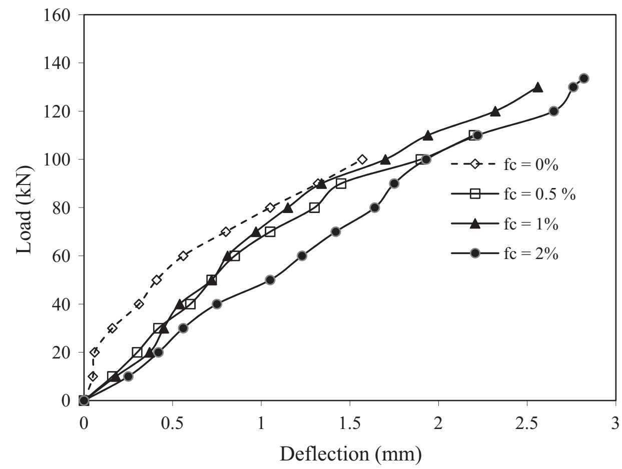



**Figure 10** Load vs mid-span deflection of fibre reinforced concrete beam (10% pond ash).

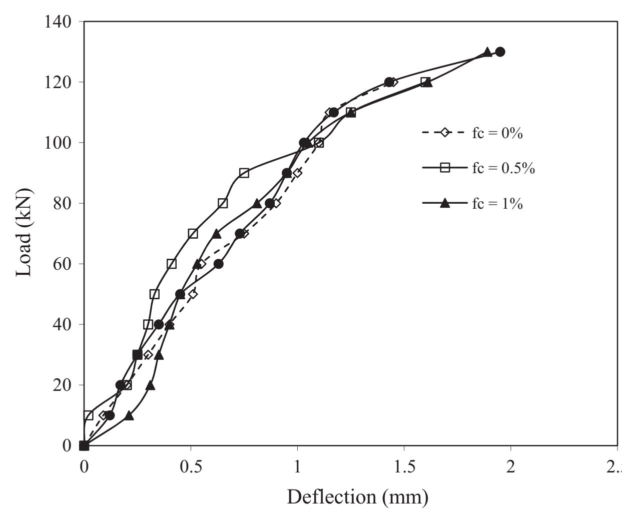



**Figure 11** Load vs mid-span deflection of fibre reinforced concrete beam (20% pond ash).

Due to the bridging capacity of fibres across the cracks, the shear strength of the beams increases and failure is ductile and in flexure. The failure in the tension of stirrups is also delayed due to the presence of steel fibres. A ductile material is one that can undergo large strains while resisting loads. When applied to RC members, the term ductility implies the ability to sustain significant inelastic deformation prior to collapse.

It was observed that flexural failure occurred for 0% pond ash beam with 0% fibre content. Further, a few wide shear cracks were found in the case of beam without fibre. After the peak load in flexure, the propagation of diagonal cracks and shear failure were observed at the end of the test.

With fibre reinforcement, due to its bridging capacity across the cracks, the shear strength of the beam increased and the failure was ductile. When the fibre content (%) increased, a similar failure pattern was observed in the beams. Due to the presence of fibres, stirrups are less stressed and the flexural

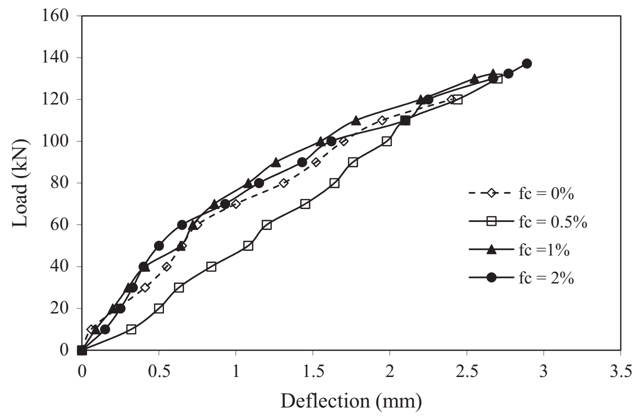



**Figure 12** Load vs mid-span deflection of fibre reinforced concrete beam (30% pond ash).

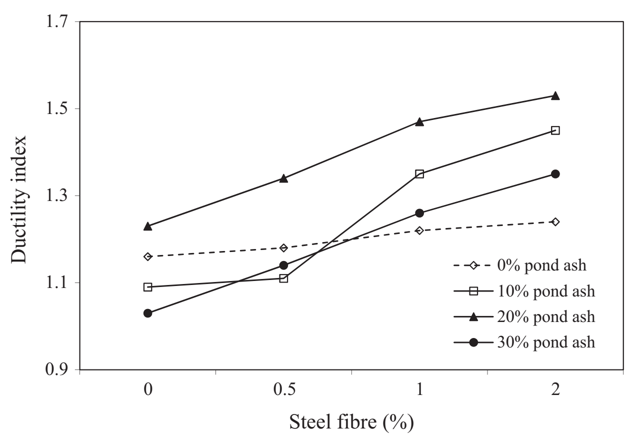



**Figure 13** Effect of steel fibre on ductility index.

failure of stirrups is delayed. In the tensile zone of the beams, flexural failure was observed. Fine flexural cracks were observed in fibre concrete beams, thus indicating a higher ductility.

### 3.5. Ductility index

Ductility of a structure can be defined as its ability to absorb energy without critical failure. Ductility generally refers to the amount of inelastic deformation which a material or structure experiences before complete failure. This deformation can be measured in terms of displacement, strain or curvature. Ductile behaviour allows a structure to undergo large plastic deformations with little decrease in strength and hence prevents brittle failure. Conventional steel reinforced beams have a distinct elastic and inelastic phase of deformation before and after yielding of steel. Hence for these structures, ductility can be defined quantitatively as the ratio of the total deformation at failure divided by the deformation at the elastic limit.

Ductility index ($\mu$) is conventionally defined as the ratio between $\delta_u$ and $\delta_y$ where $\delta_u$ is the mid-span deflection at the beam ultimate load and $\delta_y$ is the mid-span deflection at the yielding load of the tensile steel reinforcement at the central support. Table 5 shows the ductility index ($\mu$) values. Ductility index increased from 7% to 33% when steel fibre

**Table 7** Flexural rigidity.

| S.no | Beam ID | Ultimate load (kN) | Service load (kN) | Flexural rigidity (MN-m$^2$) |
|---|---|---|---|---|
| 1 | A1 | 88.20 | 30.87 | 0.503 |
| 2 | A2 | 90.40 | 31.64 | 0.234 |
| 3 | A3 | 92.60 | 32.41 | 0.228 |
| 4 | A4 | 131.80 | 46.13 | 0.322 |
| 5 | B1 | 103.00 | 36.05 | 0.468 |
| 6 | B2 | 110.20 | 38.57 | 0.393 |
| 7 | B3 | 130.40 | 45.64 | 0.386 |
| 8 | B4 | 133.60 | 46.76 | 0.360 |
| 9 | C1 | 122.80 | 42.98 | 0.566 |
| 10 | C2 | 129.00 | 45.15 | 0.576 |
| 11 | C3 | 132.40 | 46.34 | 0.587 |
| 12 | C4 | 135.60 | 47.46 | 0.496 |
| 13 | D1 | 124.20 | 43.47 | 0.369 |
| 14 | D2 | 131.80 | 46.13 | 0.355 |
| 15 | D3 | 132.40 | 46.34 | 0.430 |
| 16 | D4 | 137.20 | 48.02 | 0.516 |

increased from 0% to 2%. Fig. 13 shows that ductility index value increases when steel fibre per cent increases.

### 3.6. Flexural rigidity

Based on elastic deformation theory, the experimental flexural rigidity of a simply supported beam can be obtained by using Eq. (6). In this study, the flexural rigidity of the beams was determined after cracking. Table 7 shows the flexural rigidity values.

$$(E_cI) = PL^3/48\delta_{exp} \tag{6}$$

where $L$ is the span length of the beam, $E_c$ is the elastic modulus of concrete.

When 2% steel fibres were used, flexural rigidity increased when compared with 0% and 0.5% SFRC beams.

### 3.7. Measurement of crack width

Formation of flexural cracks in beams is detrimental to structural safety as it can affect the serviceability of the structure. Excessive crack width can impair corrosion resistance. Therefore, control of cracking helps improve serviceability. The width of the crack was measured using a crack width microscope.

The following factors need to be considered for investigating flexural crack width:

1. Stress or strain in steel is the most important parameter affecting crack width.
2. The thickness of concrete cover and the cross-sectional area of concrete surrounding each bar are also important geometric variables.
3. The crack width on the tension face is affected by the strain gradient from the level of the steel to the tension face.

ACI 224 R-01 [32] recommends the following equation for predicting crack width:

$$w = 2.2\beta\varepsilon_s^3\sqrt{d_cA} \tag{7}$$

**Table 8** Crack width.

| S.no | Beam ID | First cracking load ($P_{cr}$) (kN) | Yield load ($P_y$) (kN) | Average crack width ($w_{Exp}$) (mm) | Predicted crack width ($w$) (mm) |
|---|---|---|---|---|---|
| 1 | A1 | 50.00 | 81.20 | 0.34 | 0.28 |
| 2 | A2 | 62.20 | 82.40 | 0.28 | 0.22 |
| 3 | A3 | 78.40 | 86.60 | 0.21 | 0.19 |
| 4 | A4 | 80.40 | 120.80 | 0.18 | 0.14 |
| 5 | B1 | 53.00 | 95.30 | 0.36 | 0.28 |
| 6 | B2 | 63.40 | 102.20 | 0.26 | 0.18 |
| 7 | B3 | 72.40 | 123.40 | 0.22 | 0.14 |
| 8 | B4 | 77.80 | 128.60 | 0.16 | 0.11 |
| 9 | C1 | 56.60 | 117.80 | 0.29 | 0.28 |
| 10 | C2 | 61.60 | 121.00 | 0.22 | 0.16 |
| 11 | C3 | 74.40 | 127.40 | 0.19 | 0.12 |
| 12 | C4 | 77.60 | 130.60 | 0.14 | 0.10 |
| 13 | D1 | 54.20 | 118.20 | 0.27 | 0.28 |
| 14 | D2 | 62.40 | 124.80 | 0.21 | 0.21 |
| 15 | D3 | 73.00 | 127.40 | 0.17 | 0.19 |
| 16 | D4 | 76.20 | 132.20 | 0.12 | 0.13 |

where $w$ = most probable crack width, ß = ratio of distance between neutral axis and tension face to distance between, neutral axis and centroid of reinforcing steel, $\varepsilon_s$ = strain in reinforcement due to applied load, $d_c$ = thickness of cover from tension face to centre of closest bar, and $A$ = area of concrete symmetric with reinforcing steel divided by the number of bars.

Using the above equation, crack width was calculated (Table 8). The predicted values of crack width were compared with those measured experimentally. Experimental results showed that, when fibre content increased, the crack width decreased due to the bridging action of fibres. It is found that there is a good correlation between the measured values and calculated values.

## 4. Conclusions

The following conclusions can be drawn from the experimental study:

1. When pond ash content increased for a given steel fibre content in the beams, ultimate load increased. For example, in beams with 10% pond ash + 2% steel fibre, the increment was about 30% when compared with that of 10% pond ash beams of 0% steel fibre. When pond ash content of 20% was added to 1% steel fibre, the increment was about 43% when compared with that of beams with 0% pond ash + 1% steel fibre. When the pond ash content increased to 30%, for 1% fibre content, the increment was 43% when compared to 0% pond ash + 1% steel fibre.
2. Similarly, for a given pond ash content in beams, when steel fibre increased, ultimate load increased. The ultimate load of beams with 0% pond ash + 2% steel fibre increased by 49% when compared with that of beams with 0% pond ash + 0% steel fibre.
3. Ductility index increased from 7% to 33% when steel fibre increased from 0% to 2%. A good agreement between the predicted crack width and the measured crack width was obtained.
4. The use of pond ash and steel fibres in judicious quantities in combination with traditional steel reinforcement augments the performance of beams in comparison with the conventionally reinforced beams.

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