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# Fracture and impact properties of short discrete jute fibre-reinforced cementitious composites

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This paper conducted research on fracture and impact properties of short discrete jute fibre reinforced cementitious composites (JFRCC) with various matrix for developing low-cost natural fibre reinforced concretes and mortars for construction. Fracture properties of JFRCC were tested on notched concrete beams at 7, 14 and 28 days and the results were interpreted by the two-parameter fracture model (TPFM). Impact resistance of JFRCC were examined on mortar panels with the dimensions of $200 \times 200 \times 20\ mm^3$ at 7, 14 and 28 days through repeated dropping weight test. Qualitative and quantitative analyses were conducted for crack pattern, impact resistance and energy absorbed by JFRCC mortar panels based on eye observations and measurement from an oscilloscope. In addition, compressive, flexural and splitting tensile strengths of JFRCCs were tested at 7, 14 and 28 days conforming to relevant EN standards. It was found that, by combining GGBS with PC as matrix, JFRCC achieved higher compressive strength, tensile strength, fracture toughness, critical strain energy release rate, and critical stress intensity factor than those with combination of PFA and PC as matrix. Impact tests, however, indicated that JFRCC mortar panels with PFA/PC matrix possessed higher impact resistance, absorbed more impact energy and survived more impact blows upon failure than those with GGBS/PC matrix at the ages of 14 and 28 days. JFRCC mortar panels did not shatter into pieces and demonstrated a ductile failure while the plain mortar ones behaved very brittle and shattered into pieces. Upon impact failure, fibre pull-out was observed in JFRCC mortar panels with PFA/PC matrix while fibre fracture in those with GGBS/PC matrix. Besides, the impact resistance, in terms of the number of impact blows survived and the total energy absorbed upon failure, of JFRCC mortar panels decreased with age.



## 1. Introduction

Nowadays one of the main challenges of construction industry is to improve their image in terms of sustainability. Therefore using sustainable materials to the best of their properties is one of the key strategies to achieve sustainable construction. Unreinforced cementitious materials are characterised by low tensile strength, low fracture toughness, and low tensile strain capacities. The inclusion of short discrete fibres, however, in concrete, mortar and/or cement paste can largely enhance their many engineering properties, such as fracture toughness, tensile strength, flexural strength, resistance to fatigue, impact, and thermal shock [1].

Economics and other related factors in many developing countries, where natural fibres of various origins are abundantly available, demand engineers to employ appropriate technology to utilise natural fibres and local materials as effectively, economically and much as possible to produce good quality but low-cost fibre-reinforced cementitious composites (FRCCs) for housing and other needs. Synthetic fibres, such as Polyvinyl Alcohol (PVA) fibres, could be much more expensive, in terms of cost per unit weight, than other ingredients for making FRCCs. Therefore a potential saving can come from replacing synthetic fibres by natural fibres which possess many advantages, such as: (1) abundance and therefore low cost, (2) biodegradability, (3) flexibility and soft during processing and therefore less machine wear, (4) minimal health hazards, (5) low density, (6) desirable fibre aspect ratio, and (7) relatively high tensile and flexural modulus [2]. If natural fibres in a relatively brittle cement matrix are to achieve and maintain toughness and ductility of the composite, the durability of such fibres in a highly alkaline cement matrix must be taken into consideration and ensured by effective modifications made to fibre surface and/or to matrix compositions to overcome the inherent problem i.e. ‘embrittlement’, of natural fibres as evident from the pioneering work done by Gram [3].

Most of the developments with FRCCs so far involve the use of Portland cement (PC) as matrix. However, high alumina cement, gypsum, and a variety of special low carbon and low energy supplementary cementitious materials have also been used to produce FRCCs, which may improve the durability of the composites, and/or

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reduce chemical interactions between fibres and cementitious matrix. Natural fibres are prospective reinforcing materials and their applications as reinforcement in cementitious composites have been catching more and more attentions from construction industry recently.

During its service life, there is a wide variety of extreme environmental and/or dynamic loads that an infrastructure may experience. Severe structural damage or even catastrophic failures can occur due to these extreme environment events and/or dynamic loads. Hence there is a need to design civil infrastructure resilient to seismic, impact, and blast loading to enhance public safety [4]. The behaviour of a structure to extreme environmental events and dynamic loads, however, largely depends on the materials which the structure is made of.

As well known, cracking may impair the durability of concrete by allowing ingress of aggressive agents. In case of natural fibre reinforcement, it is essential to reduce the cracking within the composite as it accelerates the deterioration of fibres once certain width of crack is formed. It is thus important to investigate the fracture properties of natural fibre reinforced cementitious composites for infrastructure applications. However, there is very limited research published on fracture and impact behaviour of natural fibres reinforced cementitious composites in scientific literature. It is generally believed that the inclusion of natural fibres improves the fracture toughness and impact resistance of cementitious materials. Al-Oraimi and Seibi [5] reported that using even a low percentage of natural fibres improves the mechanical properties and the impact resistance of concrete making it demonstrate similar performance compared to synthetic fibre reinforced concrete. However, Silva and Rodrigues [6] found that the addition of sisal fibres into concrete reduced its compressive strength which they claimed due to its low workability making its microstructure not as dense as that without fibre reinforcement.

Ramakrishna and Sundararajan [7] tested sisal, coir, jute and hibiscus cannebinus (kenaf) fibres reinforced cement mortars with different fibre lengths and fibre dosages. They found that the impact strength of mortars with fibre reinforcement is always higher than that of those without fibre reinforcement. In some cases, the impact resistance of the former is 18 times higher than that of the latter. Savastano et al. [8] compared the mechanical performance of cement composites reinforced with sisal, banana and eucalyptus fibres. They found that those cement composites reinforced by sisal and banana fibres, with the length of 1.65 or 1.95 mm, exhibit more stable fracture behaviour than those reinforced by eucalyptus fibres with the length of 0.66 mm which confirms that fibre length influences the process by which load is transferred from cement matrix to fibres. Li et al. [9] investigated both dry and wet mixing methods in order to yield homogeneous dispersion of hemp fibres in cement matrix and it was concluded that wet mixing method results better dispersion and has positive impact on the flexural properties of fibre-reinforced concrete. Kundu et al. [10] reported a cost effective process methodology for manufacturing jute fibre reinforced concrete sewage pipe. In that study, jute fibres were chopped and treated by chemicals in order to achieve homogeneous dispersion of jute fibres into cement matrix. It was found that the load bearing capacity of jute fibre-reinforced sewage pipes was significantly increased as compared to the concrete pipes made without fibre reinforcement, indicating that natural fibres, such as jute fibres, could be reasonably good reinforcement for cement-based materials. However using chemicals to treat jute fibres obviously increases the cost and decreases the sustainable score of the final FRCC products. Ali et al. [11] investigated the effect of embedment length, diameter and pre-treatment condition on bond strength between coconut fibre and concrete through experiment. In their study, coconut fibres were loosed and soaked in tap water for 30 min. Then they were washed and soaked again for 30 min for three times followed by straightening and drying till most moisture is removed. The soaked fibres were then treated either (1) with boiling water and washed with tap water; or (2) with chemicals, in that case, first in 0.25% Sodium Alginate ($NaC_6H_7O_6$) solution for 30 min followed by in 1% Calcium Chloride ($CaCl_2$) solution for 90 min. They found that fibre tensile strength, fibre toughness and fibre-concrete bond strength can be increased by 34%, 55% and 184%, respectively, when fibres are boiled and washed. In comparison, chemical pre-treatment causes decrease in bond strength and tensile strength by 25% and 23%, respectively. This study suggests that simple treatment of natural fibres using boiling water might be a good way to increase the bond between fibres and cement matrix.

The long term performance of natural fibre reinforced cement composites can be affected by two features of natural fibres: length changes which fibres may become longer than when they were originally incorporated into cementitious systems because of their hygroscopic nature; and variations in mechanical properties which may be associated with reduced strength and toughness of FRCCs. These two effects are independent, but they both may lead to undesirable performance such as increased sensitivity to cracking. However, in properly designed components, and adequately formulated and treated composites, these effects can be minimised or even eliminated [12].

High alkali environment of PC dissolves the lignin and hemicellulose phases thus weakening the fibre structure [6] which could be a potential obstacle for promoting natural fibre reinforced cementitious materials. In order to reduce the high alkali environment in PC, pozzolanic materials has been employed to wholly or partially replace PC. These pozzolanic materials include high alumina cement, silica fume, pulverised fly ash (PFA), ground granulated blast furnace slag (GGBS), and natural pozzolanas such as rice husk ash, pumice and diatomite. On the other hand, using these pozzolanic materials to replace PC can help to improve the sustainability image of cement industry which produces the world's second most used material after water. Production of PC is an energy intensive process and also there is huge amount of $CO_2$ released associated with production process. On average 900 kg of carbon dioxide $CO_2$ is emitted for every 1000 kg of PC produced. Overall the cement production industry contributes approximately 5–8% of the global man-made carbon emissions. In many countries, legislation is now in place that specifies targets to reduce carbon emissions. Construction industry has been looking for alternative binding materials/mineral admixtures, such as those pozzolanic materials like GGBS and PFA, to replace PC so that to reduce its negative environment impact for decades. Moreover, some pozzolanic materials are able to improve durability and quality of concrete [13]. The usage of these mineral admixtures eventually leads to economic benefit as most of them are industrial by-products.

PFA itself is dust-like fine powder of mainly spherical and glassy particles. It has pozzolanic properties and consists essentially of $SiO_2$ and $Al_2O_3$ with the content of reactive $SiO_2$ being at least 25% by mass in order that it can be used as a type II addition for production of concrete conforming to EN 206-1 [14]. PFA has been used particularly in mass concrete applications and large volume placement to control expansion due to its low heat of hydration and also helps in reducing cracking at early ages. The main disadvantage of using PFA in concrete is that its strength development is significantly lower than that of PC resulting in a relatively low early strength. On the other hand, GGBS is a by-product from blast-furnaces of iron-manufacturing industry. It is a mixture of lime, silica, and alumina, the same oxides that make up PC, but not in the same proportion as PC. Though the compositions of GGBS may vary depending on the ores and other supplementary materials used in iron manufacturing, silicon, calcium, aluminium, magnesium, and oxygen constitute typically 95% or more of GGBS.

EN 15167-1 [15] specifies that, as a type II concrete addition, the chemical compositions of GGBS shall consist of at least 2/3 by mass of the sum of calcium oxide (CaO), magnesium oxide (MgO) and silicon dioxide ($SiO_2$) with the ratio by mass (CaO + MgO)/($SiO_2$) exceeding 1.0. The reminder shall be mainly aluminium oxide ($Al_2O_3$). Concrete made with GGBS has many advantages, including improved durability, workability and economic benefits. Similar to PFA, the drawback in the use of GGBS concrete is that its strength development is slower than that of PC concrete under 20 °C curing, although the ultimate strength may become higher than PC concrete for the same water-to-binder ratio [16].

Jute is abundantly grown in Bangladesh, China, India, Thailand and UK. Jute fibres are extracted from the fibrous bark of the jute plants which grow as tall as 2.5 m with a diameter of the stem at the base of around 25 mm. The matured plants are cut down, tied into bundles and submerged in water for about four weeks during which the bark is completely decomposed and fibres are exposed. The fibres are then stripped off manually from the stems, washed and sun dried [17]. As the natural fibres are agriculture waste, engineering natural materials and products are consequently an economic option for the construction industry. Kundu et al. [10] found that jute fibres are about seven times lighter than steel fibres but with reasonably high tensile strength in the range of 250–300 MPa. Ramaswamy et al. [18] tested tensile-breaking strength and tensile elongation ratios of jute fibres in natural air dry state and also in an alkaline environment by immersion up to 28 days in sodium hydroxide solution with pH value 11. They found that the breaking tensile strength of jute fibre is quite high and that the loss of strength when immersed in an alkaline medium varies from 5% to 32%. In comparison, the fibres embedded in cement concrete showed only marginal loss of strength [18].

## 2. Theories and experiment

### 2.1. Raw materials

CEM II PC conforming to EN 197-1 [19] used for this study was purchased from LAFARGE Cement (UK). PFA for this research came from HCCP Hargreaves Coal Combustion Products Limited (UK) which is compliant with EN 450-1 [20] for use as a type II addition in the production of concrete. GGBS was obtained from Hanson Heidelberg Cement Group (UK) which is compliant with EN 15167-1 [15] for use as a type II addition in the production of concrete. The specific gravity density and Blaine fineness of the PC, PFA and GGBS used for this study were tested conforming to EN 196-6 [21] and the results are shown in Table 1. It can be seen that PFA particles are the finest among the three and PFA also possess the lowest gravity density. The chemical compositions of the three binding materials were obtained through SEM–EDX analysis with the results shown in Table 2a in terms of elements and Table 2b in the terms of oxides, respectively. For PFA, the sum of the contents of $SiO_2$ and $Al_2O_3$ is 76.34% by mass, the total content of alkali calculated as $Na_2O$ is 3.94% by mass and the content of MgO is 1.29% by mass which all satisfy the relevant requirement stipulated in EN 450-1 [20]. But it should be noted that the content of sulphuric anhydride, $SO_3$, is 3.21% by mass which does slightly exceed the limit, 3%, specified in EN 450-1 [20]. For GGBS, the contents of CaO, MgO and $SiO_2$ together are 84.96% by mass and the ratio by mass (CaO + MgO)/($SiO_2$) is equal to 1.57 which both satisfy the relevant requirements specified in EN 15167-1 [15]. Therefore, both the PFA and the GGBS used for this study can be regarded as type II addition of concrete, i.e., pozzolanic or cementitious materials, as per EN standards.

River sand with 2-mm nominal maximum grain size was used as fine aggregate for preparing cement mortars and concretes. Its grading was tested through sieve analysis and its fineness modulus was calculated as 2.64 both conforming to EN 12620 [22]. Gravel stone with 10-mm nominal maximum size was used as coarse aggregate for preparing concretes. Both sand and coarse aggregates were pre-heated in an oven with the temperature of 105 °C for 24 h and then cooled down in air for a few hours before they were mixed with other ingredients for making cement mortars or concretes.

**Table 1**
Gravity density and Blaine fineness of PC, PFA and GGBS.

| | Gravity density | Blaine fineness ($m^2$/kg) |
|---|---|---|
| PC | 2.94 | 453 |
| PFA | 2.18 | 619 |
| GGBS | 2.93 | 512 |

**Table 2a**
Chemical compositions (elements) of PC, PFA and GGBS (% by weight).

| | O | Si | Al | Ca | S | Na | Mg | Fe | K | Mn | Ti |
|---|---|---|---|---|---|---|---|---|---|---|---|
| PC | 34.62 | 7.38 | 1.97 | 50.76 | 2.11 | 0.42 | 0.49 | 1.55 | 0.78 | – | – |
| PFA | 46.10 | 24.39 | 12.79 | 2.48 | 1.29 | 1.09 | 0.78 | 7.42 | 3.11 | – | 0.68 |
| GGBS | 40.06 | 15.45 | 5.47 | 32.37 | 0.96 | 0.23 | 3.99 | – | 0.59 | 0.52 | 0.40 |

**Table 2b**
Chemical compositions (oxides) of PC, PFA and GGBS (% by weight).

| | CaO | $SiO_2$ | $Al_2O_3$ | FeO | $K_2O$ | $Na_2O$ | MgO | $SO_3$ | $TiO_2$ |
|---|---|---|---|---|---|---|---|---|---|
| PC | 71.02 | 15.78 | 3.72 | 1.99 | 0.94 | 0.56 | 0.81 | 5.27 | – |
| PFA | 3.47 | 52.18 | 24.16 | 9.55 | 3.75 | 1.47 | 1.29 | 3.21 | 1.14 |
| GGBS | 45.29 | 33.06 | 10.34 | – | 0.71 | 0.31 | 6.61 | 2.39 | 0.67 |

The commercially available jute fibre was in the form of twine (see Fig. 1) and it was cut by scissors to the desirable length of 20 mm. The manual separation of fibres from the chopped bunches was laborious and time consuming. Several fibre disentangling and dispersion methods were tried to achieve best dispersion of short discrete jute fibres in cement matrix and it was finally found a wet mixing method, similar to that proposed by Li et al. [9] and Ali et al. [11], led to homogeneous dispersion of jute fibres in concrete and mortar. The final fibre separating and dispersion method adopted in this research was as follows: chopped jute fibre bunches and sand were first mixed with water for 3 min before other ingredients were added into mortar or concrete mixtures. It was found that, by doing so, the jute fibre bunches were separated into discrete fibres and dispersed reasonably well in cement matrix.

### 2.2. Sample preparation

The basic mix proportion for concrete was Binder: Sand: Aggregate = 1:1.5:2.5 by weight with the water-to-binder (W/B) ratio equal to 0.65. Here the binder includes PC, GGBS and/or PFA whatever was presented in the mixture. If jute fibres were presented in concrete mixture, its volume ratio was 0.5%. For mortars the mix proportion was Binder: Sand = 1: 1.5 by weight with the W/B ratio also equal to 0.65. Again, the binder includes PC, GGBS and/or PFA whatever was presented in the mixture. However, the volume ratio for jute fibre was increased to 1%. The binder for making mortars and concretes was consisted of PC and GGBS or PFA at 50%: 50%-based by weight.

When preparing fresh JFRCC mortars/concretes, chopped jute fibres bunches were mixed with sand and water for 3 min in a mixer to separate them into discrete fibres. Then cementitious materials,


**Fig. 1.** Jute twine and chopped fibre bunches with the length of 20 mm.

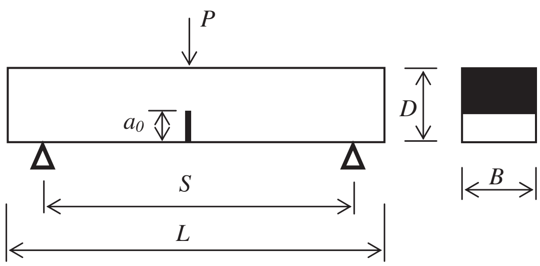



**Fig. 2.** Concrete beams with precast notch.

in this case PC, PFA and/or GGBS, and aggregates were added for another 6 min mixing. The freshly blended JFRCC mortar was then cast into $40 \times 40 \times 160\ \mathrm{mm}^3$ prismatic moulds and compacted using a vibrating table for 60 s conforming to EN 196-1 [23]. The freshly prepared concrete was transferred into 100 mm diameter $\times$ 200 mm length cylindrical moulds and $100 \times 100\ \mathrm{mm}^2$ cross-section $\times$ 500 mm length beam moulds with a notch in the mid-span (see Fig. 2). The depth of the notch was 1/3 of that of the beam. All concrete specimens were compacted using a vibrating table conforming to EN 12390-2 [24]. After that all the specimens were immediately covered with plastic sheets to prevent moisture loss with water spraying on the top surface of the plastic sheet to keep a moisture environment for 24 h. Then they were demoulded and moved into a well-controlled curing chamber with the temperature of 20 ± 1 °C and relatively humidity of 95% till the age of testing. The mixer used for making mortars was a bench-top mortar mixer while that for making concretes was a drum-type concrete mixer.

### 2.3. Compression and splitting tensile tests of concrete

Compressive and splitting tensile strengths of concrete were tested conforming to EN 12390-3 [25] and EN 12390-6 [26], respectively, from cylindrical specimens at the ages of 7, 14 and 28 days. The loading rate for compression and splitting tensile tests were 3 and 1.2 KN/s, respectively. Three cylinders were tested at each age for compressive and splitting tensile strength, respectively, to ensure repeatability. The average was presented in this paper as the compressive or splitting tensile strength of concrete at that age.

### 2.4. Fracture test

Cement-based materials exhibit pre-peak crack growth, therefore linear elastic fracture mechanics (LEFMs) cannot be directly applied to these materials. Over the last decades, several experimental and theoretical approaches have been developed to determine reliable parameters that can represent fracture properties of cementitious composites which are able to account for the development of the fracture process zone [27–31]. One, probably the most cited, fracture model which has been developed to account for the pre-critical crack growth for cement-based materials is the two-parameter fracture model (TPFM), proposed by Jenq and Shah [29], which is based on the simple premise that a change in specimen compliance can be correlated to the length of the effective crack at the point when the critical (i.e. peak) load is reached.

For concrete and other cement-based materials, linear elastic response normally goes up to a load corresponding approximately to $P_{max}/2$ in fracture test where $P_{max}$ is the maximum load in fracture test, which means that the induced $K_I$ is less than $K_{IC}^S/2$ where $K_I$ is the stress intensity factor and $K_{IC}^S$ the critical stress intensity factor. During this stage the *CTOD* (crack tip opening displacement) is zero as predicted by LEFM. During the second stage, when the applied load $P$ is greater than $P_{max}/2$, cement-based materials behave in a nonlinear mode. This is caused by the formation of the fracture process zone ahead of the crack tip, which is the existing crack being pre-notched or precast not the result of some prior crack nucleation/extension, for which a process zone first has to be developed. This process zone formation has also been referred as slow crack growth [8]. As a result of this micro-cracking, the crack tip starts to open in a fashion similar to the blunting of sharp cracks in metals due to yielding. At the peak load, there are two conditions which are simultaneously satisfied:

$$K_I = K_{IC}^S \tag{1}$$

and

$$CTOD = CTOD_C \tag{2}$$

where the critical stress intensity factor, $K_{IC}^S$, is actually the fracture toughness, *CTOD* is the crack tip opening displacement and $CTOD_C$ is the critical crack tip opening displacement. The parameters on the right hand side of Eqs. (1) and (2) are material properties. The results of the fracture test interpreted by this model are independent of specimen size. Hence the critical values, $K_{IC}^S$ and $CTOD_C$, are size independent which is one of the major advantages of using TPFM to determine fracture properties of concrete and other cement-based materials.

According to TPFM, the critical stress intensity factor $K_{IC}^S$, the critical crack tip opening displacement $CTOD_C$, the modulus of elasticity (Tensile modulus) $E$, and the critical strain energy release rate $G_{IC}^S$ can be calculated by the following equations [29].

$$K_{IC}^S = \frac{3\left(P_{\max} + \frac{0.5WS}{L}\right)S}{2DB^2}\sqrt{\pi a_e}F(\alpha) \tag{3}$$

$$CTOD_C = \frac{6\left(P_{\max} + \frac{0.5WS}{L}\right)Sa_e}{D^2BE}V_1(\alpha)\{(1-\beta)^2 + (-1.149\alpha + 1.081)(\beta - \beta^2)\}^{0.5} \tag{4}$$

$$E = \frac{6Sa_eV_1(\alpha)}{C_iD^2B} \tag{5}$$

$$G_{IC}^S = K_{IC}^{S^2}/E \tag{6}$$


**Fig. 3.** Concrete beam under three-point bending fracture test.

where in Eqs. (3)–(6) $W$ is self-weight of the notched beam; $S$ is the span of the beam; $L$ is the length of the beam; $a_e$ is critical effective crack length; $D$ is the depth of the beam; $B$ is the width of the beam; $C_i$ is initial loading compliance; $C_u$ is unloading compliance; $F(\alpha)$ is a shape function about $\alpha$ for calculating $K_{IC}^S$, and $V_1(\alpha)$ is a shape function about for calculating $CTOD_C$ and $E$ where $\alpha = a_e/D$; and $P_{max}$ is the maximum load.

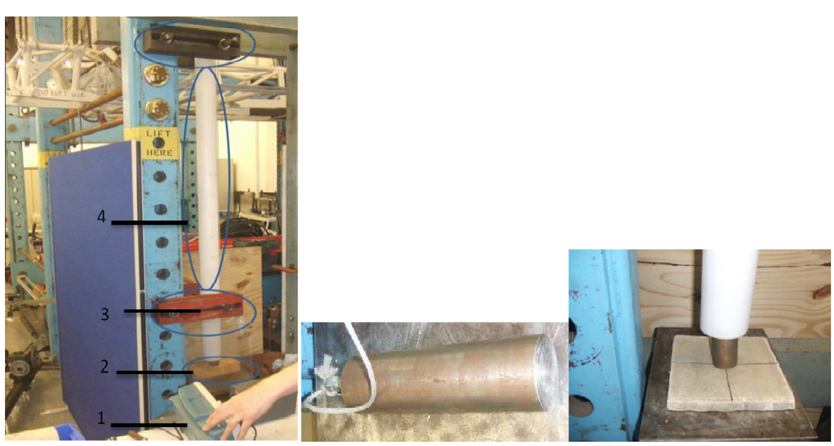
To implement this model into characterizing fracture properties of concrete, the load with respect to *CMOD* (crack mouth opening displacement) of a notched beam is needed. Therefore, in this study, fracture test was conducted on $100 \times 100 \times 500$ mm$^3$ centrally notched beam, with the span of 400 mm and depth 100 mm, under three-point bending. To ensure stability, the test was carried out under crack mouth opening displacement (*CMOD*) control mode using an Instron 2670 series crack opening displacement (*COD*) gauge with a *CMOD* rate of 0.0075 mm/min (see Fig. 3). Two notched JRFCC beams were tested at each age to ensure repeatability. The fracture test was conducted in accordance with the RILEM recommendations [30] associated with the TPFM to obtain the elastic modulus ($E$), critical stress intensity factor ($K_{IC}^S$), critical crack tip opening displacement ($CTOD_C$), and critical strain energy release rate ($G_{IC}^S$) of various JFRCC concretes.

### 2.5. Impact test

Impact resistance of fibre reinforced composite can be measured by a number of test methods, which can be broadly grouped into the following categories: (i) dropping weight single or repeated impact test; (ii) weighted pendulum type impact test; (iii) projectile impact test; (iv) explosion-impact test; (v) constant strain rate test; (vi) split Hopkinson bar test; and (vii) instrumented pendulum impact test [32]. The impact resistance of a composite material is measured using one of the following criteria, such as: (i) energy needed to fracture the specimen; (ii) number of blows to achieve a specified distress level (in a repeated impact test); and (iii) the size of the damage (i.e. crater size, perforation) or the size and velocity of spall after the specimen is subjected to a surface blast loading [33]. Impact test seems to be simple, but quantitative interpretation of the test results to derive inherent physical material parameters can be difficult. Therefore impact test can also be divided into three categories: (1) qualitative, (2) semi quantitative and (3) quantitative, depending on the property measured, rather than on the method by which the impact test is conducted [12].

In this research, the impact resistance of mortar panels was determined by dropping a steel rod in a vertical guide tube from a fixed height of 0.5 m and repeating this till failure. The steel rod used for impact test had a mass of 2 kg with a cylindrical body diameter of 4 cm and a height of 17 cm. Its front head had a spherical shape (see Fig. 4). The guide tube had an inner diameter greater than that of the ball so that it can be reasonably assuming that there is no friction between the ball and the tube inner wall when the rod is falling along the guide tube. The steel rod was projected at exactly the centre of the mortar panel which was resting on a base plate. An oscilloscope was employed to monitor the response of the base plate during impact tests. To do so, an accelerometer was mounted underneath the centre of the base plate and connected to the oscilloscope. With such set-up, in a continuous impact test for a series of mortar panels, only mortar panel needs to be replaced when moving to next test.

The assumption used to conduct the semi-quantitative analysis for the impact resistance of mortar panels are explained as follows. The potential energy of the steel rod was converted into kinetic en-




**1: oscilloscope 2: base plate**
**3: locking the tube 4: Tube to direct the balll**

**(a)** **(b)** **(c)**

**Fig. 4.** Set-up of impact test: (a) the overall set-up; (b) the rod; and (c) the moment when the front head of the rod was impacting a mortar panel resting on the base plate.

ergy which was further converted into a signal and picked up by the oscilloscope. In the reference test, a reference steel panel with the dimensions of 200 × 200 × 20 mm$^3$, same as the JFRCC mortar panels, resting on the base plate was impacted by the steel rod from the height of 0.5 m. It was found that there was very small deflection at the central of the reference steel panel which can be neglected. However, when the reference steel panel was replaced by a JFRCC or plain mortar panel, the deformation of the mortar panel caused by impact was much greater and cracks appeared on its surface.

The total potential energy of the steel rod was consumed by not only cracking the mortar panel but also driving the base plate downwards. Hence a drop in voltage was detected by the oscilloscope. In comparison, when the reference steel panel was impacted by the steel rod from the same height, 0.5 m, the voltage measured by the oscilloscope was always 140 V which was confirmed by several trial tests. Therefore, this value of 140 V was used as the reference voltage. The voltages recorded by the accelerometer during impact tests of mortar panels resting on the base plate were then analysed by comparing it with the reference value. It is the interest of this research that the energy absorbed by various mortar panels are semi-quantitatively figured out after each impact blow so that the impact resistance of various JFRCCs can be assessed qualitatively and semi-quantitatively and their fracture resistance can be relatively compared.

Impact tests were conducted on JFRCC and plain mortar panels at the ages of 7, 14 and 28 days. In addition, flexural and compression strengths of JFRCC mortars were measured at 7, 14 and 28 days conforming to EN 196-1 [23] to monitor the strength development of the JFRCC mortars with age. Three prismatic mortar specimens with the dimensions of 40 × 40 × 160 mm$^3$ were tested for flexural strength for each mixture at each age. Consequently, six mortar cubes with the loading area of 40 × 40 mm$^2$ were tested for each mortar mixture at each age.

## 3. Results and discussion

### *3.1. Compressive, flexural and/or splitting tensile strengths of concretes and mortars*

The compressive and splitting tensile strengths of various JFRCC concretes at 7, 14 and 28 days are shown in Figs. 5 and 6, respectively. It can be seen that, GGBS concrete consistently exhibited a higher strength than PFA ones in both compression and tension. Due to low pozzolanic reaction, the strength of PFA concretes grew very slowly and it never reached as high as that of GGBS mixture up to the age of 28 days. Fibre reinforcement did increase the splitting tensile strength of concrete with it rising to 1/6 of the corresponding compression value for PFA concrete comparing with the value of 1/10 usually quoted as the ratio between the tensile and the compression strength for plain concrete. This value was 1/7 for GGBS mixtures at 14 and 28 days.

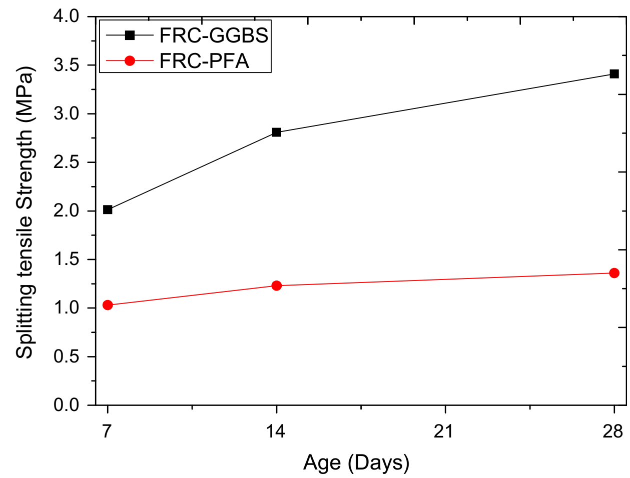



**Fig. 6.** Splitting tensile strength of JFRCC concretes at various ages.

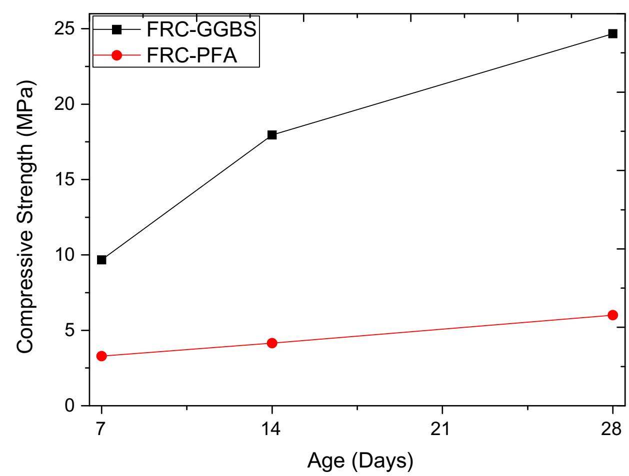



**Fig. 7.** Compressive strength of JFRCC mortars at various ages.

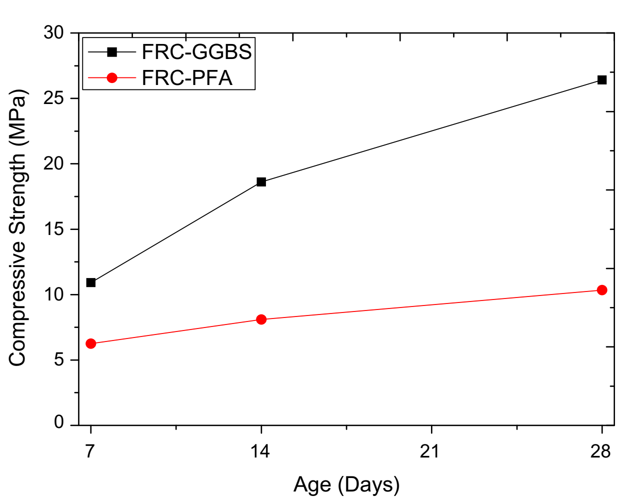



**Fig. 5.** Compressive strength of JFRCC concretes at various ages.

The compressive and flexural strength of JFRCC mortars progressed quite rapidly from early to later ages for both mortar mixtures, i.e. PFA/PC and GGBS/PC. However, the GGBS/PC mortar mixture exhibited much higher compressive and flexural strength when compared to PFA/PC one. Overall, strength of JRFCC with GGBS/PC matrix is higher than that of JFRCC with PFA/PC matrix (See Figs. 5–8).

### *3.2. Hydration of PFA and its strength development in concrete/mortar*

The reason why cementitious composites with PFA as matrix possess lower mechanical properties than those with GGBS as matrix is due to the delay in hydration caused by PFA. The hydration products of PFA closely resemble C–S–H produced by the hydration of PC [34]. However the reaction does not start until certain age after mixing. In the case of PFA, this can be as long as one week or even later [35]. The reactivity of PFA is influenced by the alkali content of the PC with which the PFA is used with.

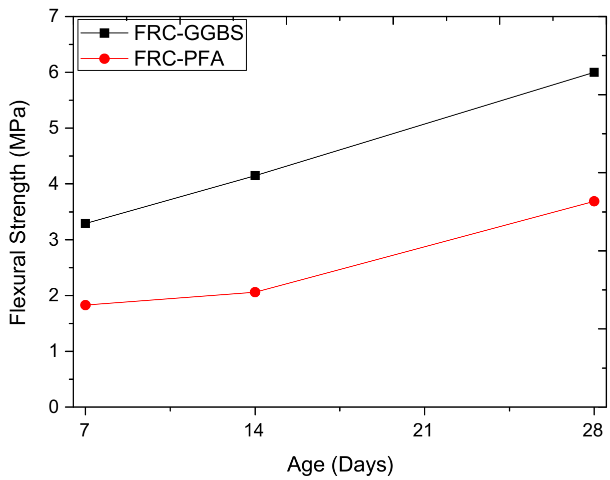



**Fig. 8.** Flexural strength of JFRCC mortars at various ages.

The hydration of PFA is also affected by PC when they are blended with water. Moreover, in addition to the effect of chemical reactions, PFA has a physical effect of improving the microstructure of the hydrated cement paste which is the packing effect of PFA particles at the interface between coarse aggregate and this packing effect is absent in mortars as there are no coarse aggregate [35]. The extent of packing effect depends on both the PFA and the PC used. Better packing is achieved with coarser PC and with finer PFA [36], but the main contribution of packing lies in a reduction in the volume of large capillary pores [35]. For these reasons, strength measurements do not adequately establish the contribution of PFA to the development of strength of a particular concrete/mortar in which PFA is incorporated.

### 3.3. Interfacial bond

The mechanical behaviour of fibre–cement composite is largely dependent on the bond between fibre and cement matrix which depends on many factors like the physical characteristics of the fibres such as geometry, type, and surface characteristics, fibre orientation, fibre volume ratio and fibre distribution, the chemical composition of the fibre, but also the treatment of the fibre and additives in the cement mixture. The interfacial bond may be chemical or physical or a combination of both. In general, organic fibres, such as natural fibres, are considered to be less compatible with inorganic matrix, such as cement matrix, in terms of chemical bond [37]. Poor bonding between natural fibres and cement matrix is often due to swelling of the fibres in the wet mix and subsequent shrinkage upon drying. As pointed out by Paramasivam et al. [38] and Cook et al. [39], the bond between natural fibres and cement matrix can be improved by applying a casting pressure resulting in an increase in strength. The main effect of the casting pressure is to reduce the voids and to densify the cementitious composite while, in this research, the usage of vibrating table to compact various cement mortars was very essential in obtaining JFRCC with good interfacial bond between jute fibres and various cement matrixes as it helped to reduce the voids in the mixtures. Pre-treating natural fibres can clean and chemically modify fibre surface, stop the moisture absorption process, and increase the surface roughness, all of which will influence the mechanical performance and properties of the natural fibre reinforced cementitious composites.

### 3.4. PC matrix partially replaced by PFA or GGBS

It has been reported that the use of ternary blends containing slag/metakaolin and silica fume are effective in preventing fibre degradation [40]. But in some cases the low alkalinity is not enough to prevent lignin from being decomposed [41]. Fast carbonation can also induce lower alkalinity [42]. This is confirmed by Tonoli et al. [43] who reported applying artificial carbonation to lignocellulosic fibre reinforced cementitious roofing tiles to obtain $CaCO_3$ from $Ca(OH)_2$ leading to an increased strength and reduced water absorption. D'Almeida et al. [44] used blended cement matrix where 50% PC by weight was replaced by metakaolin and produced a matrix totally free of calcium hydroxide that prevents migration of calcium hydroxide to the fibre lumen, middle lamella and cell walls and thus avoids brittle failure of natural fibre reinforced cement composites. The use of pozzolanic fillers, such as silica fume and GGBS, can reduce the alkalinity of the matrix as well as the content of calcium hydroxide, and thus slow down the processes which lead to degradation in the properties of JFRCC. In this research, replacing 50% by weight of PC by PFA and GGBS, respectively, was adopted to reduce the alkalinity of the matrix as mentioned before.

Partial replacement of PC by GGBS did not reduce the brittleness of cementitious composites as much as PFA did which was observed in impact tests where the JFRCC PC/GGBS mortar panel shattered into pieces with much less blows than the JRFCC PFA/PC mortar panel at ages of 14 and 28 days (see Table 4). The presence of GGBS in the mixture improves workability and makes it more mobile but cohesive which was one of the most important observations during preparing cementitious composites in this research and is the consequence of a better dispersion of the cementitious particles and of the surface characteristics of the GGBS particles, which are smooth and absorb little water during mixing [45]. Such phenomenon was obviously observed when making JFRCC with GGBS matrix. The proportions of GGBS and PC influence the development of strength of concrete. For the highest medium term strength, the proportions are about 1:1, that is 50% PC and 50% GGBS by weight in the cementitious composites [46] which was the recipe taken by this research when preparing cementitious mixtures.

### 3.5. Fracture toughness

The measured load versus *CMOD* curves for various notched beams under three-point bending fracture test are shown in Figs. 9a–c for the age of 7, 14, and 28 days, respectively, in which FRC-GGBS and FRC-PFA represents jute fibre reinforced concrete with GGBS and PC as matrix and that with PFA and PC as matrix, respectively, while UN-GGBS represents plain concrete with PC and GGBS

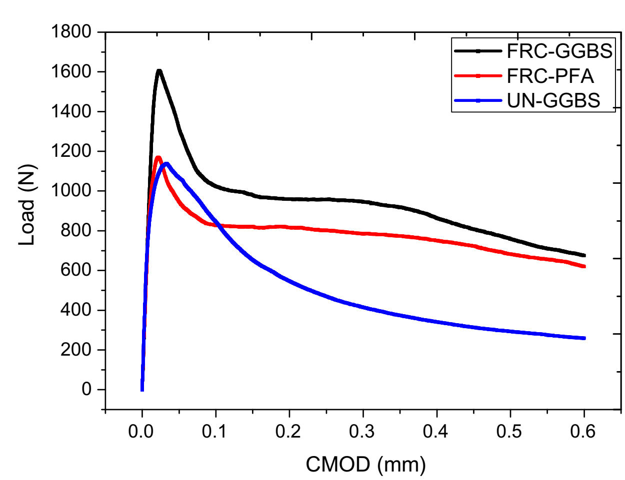



**Fig. 9a.** Load versus *CMOD* for various concrete beams at 7 days.

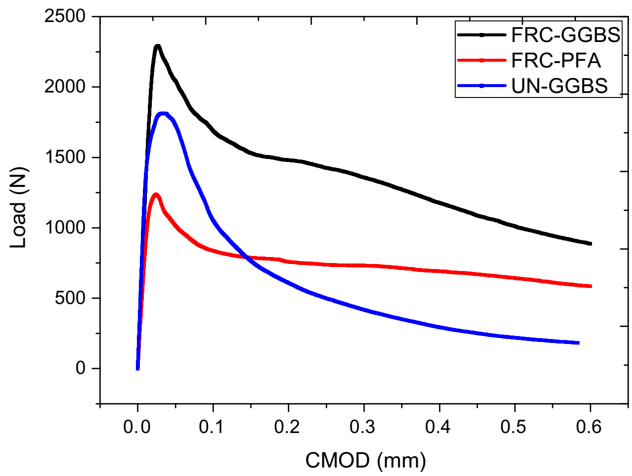



**Fig. 9b.** Load versus *CMOD* for various concrete beams at 14 days.

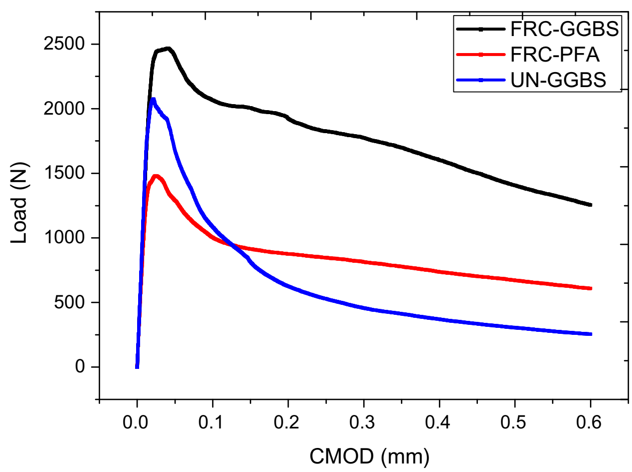



**Fig. 9c.** Load versus *CMOD* for various concrete beams at 28 days.

as matrix at 50%:50% based by weight. It can be found from Figs. 9a–c that the area under the load versus *CMOD* curve for FRC-GGBS is the largest at all ages suggesting that fracture toughness of JFRCCs increased with the addition of GGBS and fibres. Short discrete fibres arrest the macrocracks in concrete and hence it takes more energy for crack to propagate in concrete resulted in increased cracking resistance. In the case of plain concrete, there is a sharp drop in load after the peak load and that is due to the brittleness of the unreinforced concrete. Chakraborty et al. [47] investigated jute fibres as a reinforcing agent in improving the physical and mechanical properties of cement mortar and found that fracture toughness is significantly increased of jute fibre reinforced cement mortar up to 1% by weight, with respect to cement, jute fibre loading while, with the further increase of jute contents, the fracture toughness of cement mortar is gradually reduced. In this study, the jute fibre loading in concrete was 0.5% by volume, which is equivalent to 1.2% by weight, of cementitious binder close to the recommended fibre loading recommended by Chakraborty et al. [47]. It was found that, by this fibre loading, fracture toughness of JFRCC with GGBS and cement as matrix was largely increased compared with plain concrete at all the ages investigated.

The load against *CMOD* graphs presented in Figs. 9a–c also indicate that JFRCC concrete with GGBS, i.e. FRC-GGBS, has the highest fracture strength, indicated by it demonstrating the highest peak load among the three composites at all the three ages. For a crack to follow the path of least resistance in concrete, it should propagate along the relative weaker interface rather than through the relative tougher matrix. The interface in the fibre cement composites is relatively weak leading to preferential crack propagation along it rather than through the matrix. Under an applied load, distributed micro-cracks propagate and align themselves to produce macro-cracks. When loads are further increased and conditions of critical crack growth, i.e. Eqs. (1) and (2) are satisfied at the tips of the macro-cracks, unstable and catastrophic failure is thus reached.

### 3.6. Fracture parameters

Relevant fracture parameters, i.e. $C_i$, $C_u$, $K^S_{IC}$, $CTOD_C$, $G^S_{IC}$ and $E$, for various JFRCC and plain concretes were calculated from the fracture test results based on the TPFM. These parameters are shown in Tables 3a–c. TPFM considers the elastic–plastic deformations occurred ahead of the tip of a macrocrack induced by a notch. Unloading compliance ($C_u$) is measured in the unloading branch at 95% of the maximum load in the graph of load versus *CMOD*. $C_u$ is then used to determine other materials parameters including the critical stress intensity factor ($K^S_{IC}$), and the critical crack tip opening displacement ($CTOD_C$). The initial loading compliance ($C_i$) gives the module of elasticity ($E$). The parameters $K^S_{IC}$ and $CTOD_C$ were used to calculate the strain energy release rate $G^S_{IC}$.

Figs. 10–12 present the derived fracture parameters $G^S_{IC}$, $E$, and $K^S_{IC}$, respectively, graphically with respect to *CMOD* at various ages from fracture test results. Critical strain energy release rate $G^S_{IC}$ for JFRCC with GGBS and PC as matrix increases fastest with age among the three composites tested indicating that the combination of GGBS and PC results in very strong bond between matrix and fibres. This could also explain the fact that JFRCC with GGBS/PC matrix possessed the highest critical stress intensity as found from this study (see Fig. 12).

On the other hand, plain concrete with GGBS/PC matrix had the highest modulus of elasticity $E$ which may be due to the fact that plain concrete had better workability compared with JFRCC making its microstructure much denser thus a higher initial modulus of elasticity resulted. The beneficial effects of GGBS arise from the denser microstructure of hydrated cement paste. More of the pore space was filled with C–S–H in the blended matrix than in pastes with PC only [35] which can explain why JFRCC with GGBS and PC as matrix consistently exhibits greater critical stress intensity $K^S_{IC}$, critical strain energy release rate $G^S_{IC}$ and modulus of elasticity $E$ than JRFCC with PFA and PC as matrix as indicated by Table 3a and b.

### 3.7. Impact resistance

The repeated dropping weight impact test method, adopted in this research, was also used in other studies [48] which showed that the failure pattern of concrete slabs under impact involved the formation of a localised crater followed by the formation and eventual movement of a cone shaped plug of concrete. The samples tested in this research include: JFRCC mortar panels with GGBS and PC at 50%:50%-based by weight as matrix; JFRCC mortar panels with PFA and PC at 50%:50%-based by weight as matrix; and plain mortar panels with GGBS and PC at 50%:50%-based by weight as matrix.

With the aid of oscilloscope and judgement by eyes the mortar panels were analysed for their failure. Initial voltage picked at first impact was used as the bench mark. At each following blow the voltage varied, when it reached close to the bench mark, i.e. when it reached initial voltage ±10 V, the mortar panel was judged as satisfied one of the two failure criteria. At the same time, the judgement from eye observation was such that the crack must be

**Table 3a**
Fracture parameters of jute fibre reinforced PFA/PC concrete.

| Ages (day) | Peak load (N) | $C_i$ (mm/N) $\times 10^{-6}$ | $C_u$ (mm/N) $\times 10^{-5}$ | $K_{IC}^S$ (MPa mm$^{0.5}$) | $CTOD_C$ (mm) | $G_{IC}^S$ $\left(\frac{N^2}{mm^3 MPa}\right)$ | $\alpha_C$ (mm) | $E$ (GPa) |
|---|---|---|---|---|---|---|---|---|
| 7 | 1170.65 | 9.90 | 1.88 | 15.707 | 1.711 | 16.630 | 44.18 | 14.836 |
| 14 | 1238.37 | 9.81 | 1.94 | 16.717 | 1.962 | 18.657 | 44.94 | 14.978 |
| 28 | 1480.62 | 8.14 | 1.65 | 19.964 | 2.052 | 22.078 | 45.40 | 18.053 |

**Table 3b**
Fracture parameters of jute fibre reinforced GGBS/PC concrete.

| Ages (day) | Peak load (N) | $C_i$ (mm/N) $\times 10^{-6}$ | $C_u$ (mm/N) $\times 10^{-5}$ | $K_{IC}^S$ (MPa mm$^{0.5}$) | $CTOD_C$ (mm) | $G_{IC}^S$ $\left(\frac{N^2}{mm^3 MPa}\right)$ | $\alpha_C$ (mm) | $E$ (GPa) |
|---|---|---|---|---|---|---|---|---|
| 7 | 1607.27 | 9.51 | 1.42 | 20.265 | 1.181 | 26.588 | 39.71 | 15.446 |
| 14 | 2294.69 | 7.93 | 1.16 | 28.527 | 1.295 | 43.953 | 39.27 | 18.515 |
| 28 | 2467.72 | 7.13 | 1.65 | 33.666 | 3.858 | 55.048 | 47.77 | 20.589 |

**Table 3c**
Fracture parameters of plain GGBS/PC concrete.

| Ages (day) | Peak load (N) | $C_i$ (mm/N) $\times 10^{-6}$ | $C_u$ (mm/N) $\times 10^{-5}$ | $K_{IC}^S$ (MPa mm$^{0.5}$) | $CTOD_C$ (mm) | $G_{IC}S$ $\left(\frac{N^2}{mm^3 MPa}\right)$ | $\alpha_C$ (mm) | $E$ (GPa) |
|---|---|---|---|---|---|---|---|---|
| 7 | 1138.95 | 8.59 | 2.89 | 16.897 | 3.978 | 16.696 | 54.49 | 17.099 |
| 14 | 1814.35 | 6.86 | 1.79 | 25.462 | 3.379 | 30.270 | 49.98 | 21.419 |
| 28 | 2075.15 | 6.84 | 1.03 | 26.019 | 1.112 | 31.507 | 39.79 | 21.488 |

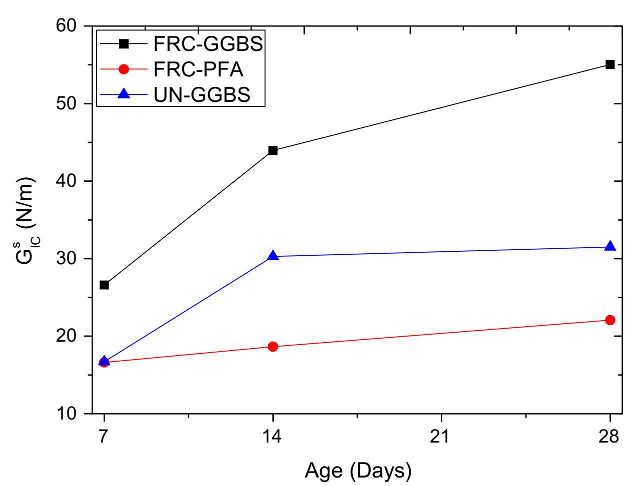



**Fig. 10.** $G_{IC}^S$ versus age for various concretes at various ages.

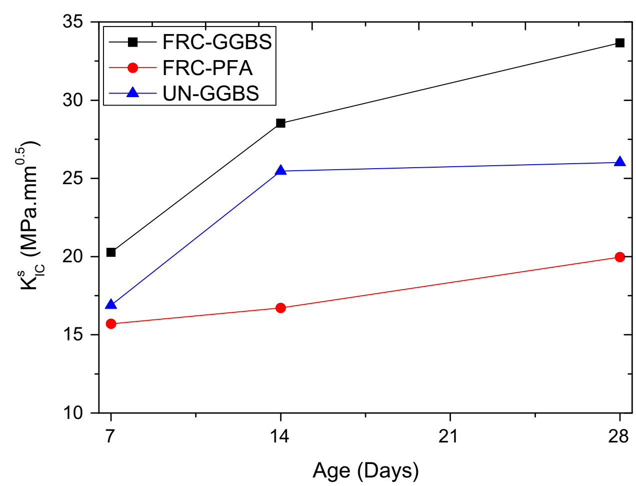



**Fig. 12.** $K_{IC}^S$ versus age for various concretes at various ages.

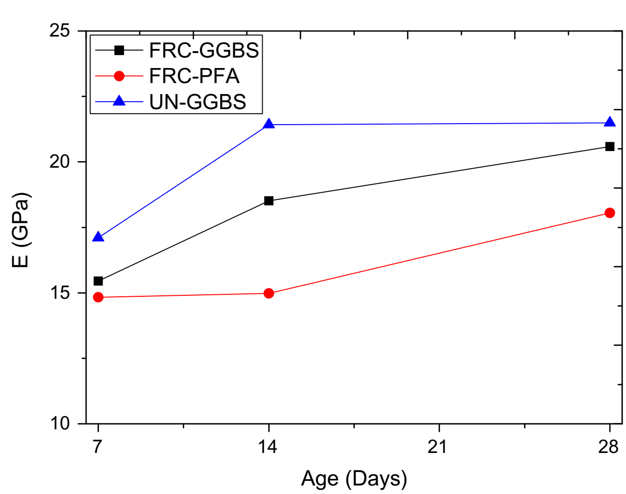



**Fig. 11.** E versus age for various concretes at various ages.

visible and propagate throughout the panel, i.e. at least one crack propagated throughout the panel reaching any two opposite edges of the square panel and throughout the depth of the panel as well which is the other of the two failure criteria for impact. Then the panel was judged as failed. To this point, the panel lost its integrity and cannot bear any more loads.

Fig. 13 shows how the cracks propagated until reaching at least two opposite edges of a square panel, which is one of the criteria of failure at the same time the voltage at that particular impact must be close to the initial voltage picked during the first blow. The impact test were carried on until both failure criteria were satisfied (see Fig. 14 indicating that the measured voltage firstly increased after 1st blow then decreased to close to the bench mark at the 4th blow) which the panel was judged as failed as it was simultaneously observed that a crack propagated to two opposite edges of the panel during this 4th blow. Based on this, the numbers of successive impacts for various mortar panels until reaching failure are presented in Table 4.

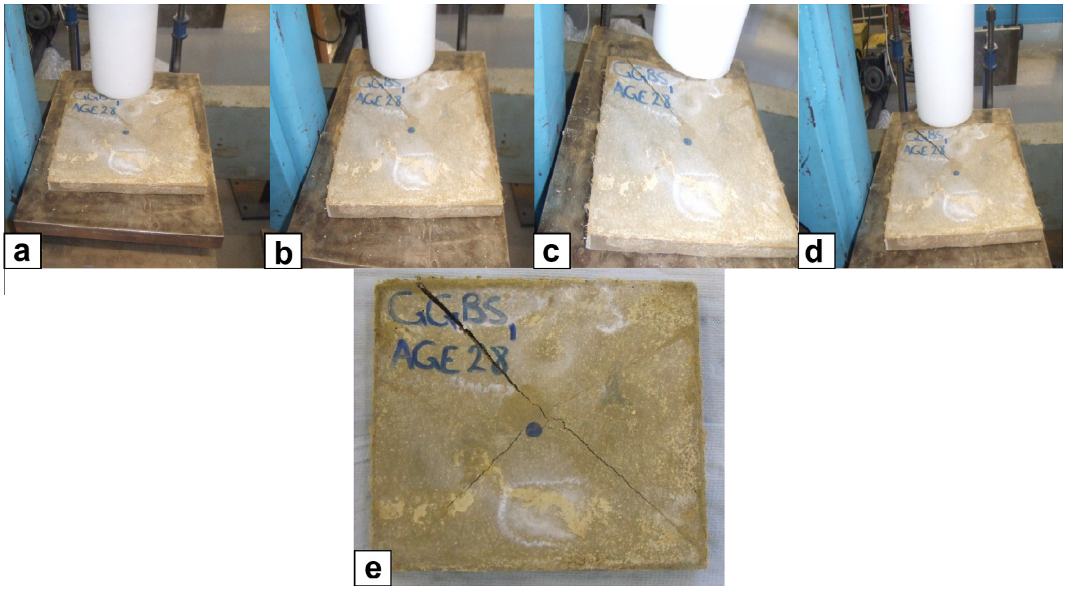



Fig. 13. Impact failure process of a JFRCC mortar panel with GGBS/PC matrix from the 1st impact (a) to the 4th impact (d) and a closer image of the final failure after the 4th impact (e).

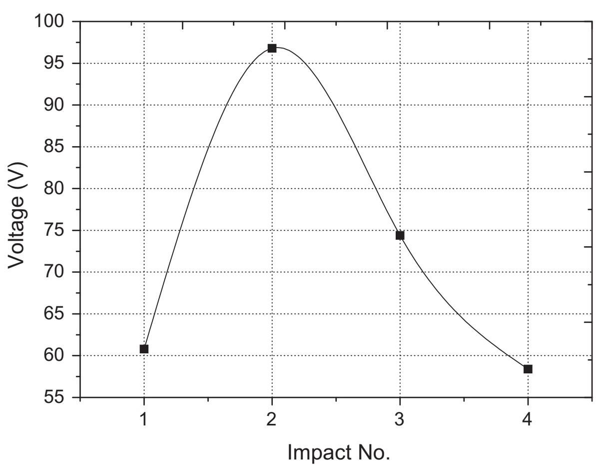



Fig. 14. Voltage changes from the 1st to the 4th impact of a FRC-GGBS panel at 28 days.

Table 4
Number of successive impact blows upon failure for various mortar panels.

| Type of mortar | Age | No. of successive impacts upon failure |
|---|---|---|
| FRC-GGBS mortar | 7 | 10 |
| | 14 | 4 |
| | 28 | 4 |
| FRC-PFA mortar | 7 | 10 |
| | 14 | 7 |
| | 28 | 6 |
| UN-GGBS mortar | 7 | 1 |
| | 14 | 1 |
| | 28 | 1 |

It can be seen from Table 4 that, with the increase in age, the number of successive blows to failure decreases which can be regarded as an indication to the increase in brittleness of JFRCC mortars with age. This means that, different from compressive strength, flexural strength and facture toughness which increased with age, the impact resistance of JFRCC mortar panel decreased with age. This phenomenon could be explained by the presence of moisture in the JFRCC mortar panels, meaning that the panels have internal pore water, due to low hydration process, at early stages of hydration of cement matrix. Thus they are more ductile than when they are at a later age.

Fig. 15 shows the ultimate failure of three types of mortar panels tested. Plain mortar panel shatters into pieces after first impact (see Fig. 15a) at all ages. JFRCC PFA/PC mortar panel at 28 days reached failure after six successive blows demonstrating great ductility and at the same age JFRCC GGBS/PC mortar panel failed after four successive blows. The projectile impact, conducted in this research, acted as a concentrated load at the centre of the mortar panel. The contact area between the steel rod and the mortar panel was compacted after each blow. It can be seen from Fig. 15b and c that larger compaction area was observed in the PFA/PC panel than in the GGBS/PC panel indicating that the PFA/PC mix was softer than GGBS/PC mix which is another indication of the slow hydration process of PFA/PC mix.

It can be seen from Fig. 16 that more fibres were found across the fracture surface of JFRCC mortar panel with PFA/PC matrix than that of JFRCC one with GGBS/PC matrix which can be ascribed to the fact that PFA/PC matrix provided a lower alkali environment to jute fibres than GGBS/PC matrix did which subsequently caused less deterioration to jute fibres as the alkali can react with the lignin in jute fibres causing them deteriorated and losing the function as reinforcement.

Considering the nature of failure, it was observed that the plain mortar panel with GGBS/PC matrix broke into pieces (see Fig. 15a) while the mortar panels reinforced by jute fibres had a number of multiple cracks and the panel remained certain integrity, i.e. in one piece, due to the presence of the short discrete fibres. This is consistent with the findings of Ramaswamy et al. [18] who reported that, in repeated dropping weight impact test, plain concrete panels exhibited total disintegration and shattering of the specimens while jute fibre reinforced concrete panels remained in one piece, thus retaining their shape and continuity. Moreover, at ultimate failure, fibre pull-out was observed from JFRCC mortar panels with PFA/PC matrix (see Fig. 16b and d) while fibre fracture was observed from JFRCC mortar panels with GGBS/PC matrix (see Fig. 16a and c).


**Fig. 15.** Ultimate impact failure of: (a) plain mortar panel with GGBS/PC matrix; (b) JFRCC mortar panel with PFA/PC matrix; and (c) JFRCC mortar panel with GGBS/PC matrix.

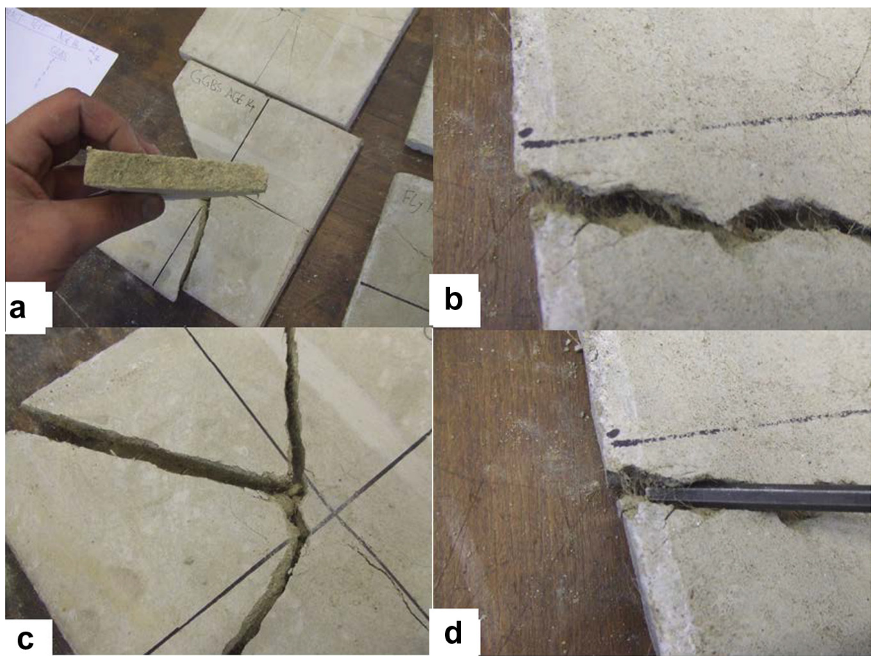



**Fig. 16.** Ultimate impact failure of a JFRCC mortar panels at 14 days: (a) and (c) JFRCC mortar panel with GGBS/PC matrix; (b) and (d): JFRCC mortar panel with PFA/PC matrix.

### *3.8. Impact energy absorbed by mortar panels*

According to the discussion in Section 2, the impact test results can be semi-quantitatively analysed. Assigning $V_1$ denoting the voltage measured at the reference impact test on the reference steel panel with the dimensions of $200 \times 200$ mm$^2$ in cross-section and 20 mm in depth, i.e. $V_1$ is the reference voltage equal to 140 V, and $V_2$ the voltage measured at certain blow when a mortar panel, with the same dimension as $200 \times 200 \times 20$ mm$^3$, replaced the reference steel panel during impact test, the energy absorbed by the mortar panel during that blow can then be calculated by the following formula.

$$\text{Energy Absorbed} = \frac{V_1 - V_2}{V_1} \times mgh \tag{7}$$

where $m$ is the mass of the steel rod equal to 2 kg and $h$ is the falling height of the steel rod equal to 0.5 m in this case. Based on this, the total energy absorbed by various mortar panels in impact tests upon failure is presented in Table 5a where the total energy absorbed by a mortar panel is a cumulative addition of energy absorbed during each blow until failure. It can be seen that at 7 days JFRCC mortar panels absorbed more energy upon failure when compared to them at 14 and 28 days. In the case of plain mortar panels with GGBS/PC matrix the energy absorbed at 7 and 14 days was very close however it decreased at 28 days. Besides, energy absorbed by plain mortar panels was much less than that by JFRCC mortar panels as they shattered into pieces after first impact. It can be found from Table 5a that the total energy absorbed by a mortar panel decreased with age which is consistent with the findings that number of impact blows survived by a mortar panel upon failure decreased with age.

**Table 5a**
Total impact energy absorbed by mortar panels upon failure.

| Age | FRC-GGBS | FRC-PFA | UN-GGBS |
|---|---|---|---|
| 7 | 47.668707 | 46.879396 | 3.153214 |
| 14 | 17.153487 | 31.335942 | 3.223286 |
| 28 | 13.173429 | 29.373943 | 2.662714 |

Total energy absorbed by JFRCC mortar panels with PFA/PC matrix at 14 and 28 days was considerably higher than those by JFRCC mortar panels with GGBS/PC matrix at the same ages which indicates that the PFA/PC matrix is more ductile and hence it can absorb more energy. Table 5b presents the energy absorbed by various mortar panels at the first blow. It can be seen that the JFRCC mortar panels absorbed much more energy than the plain mortar ones with the value of the former is more than twice of that of the latter at all the ages investigated. Energy absorbed was high-

**Table 5b**
Energy absorbed at 1st impact blow by mortar panels.

| Age | FRC-GGBS | FRC-PFA | UN-GGBS |
|---|---|---|---|
| 7 | 7.047414 | 6.399052 | 3.153214 |
| 14 | 6.698829 | 6.614743 | 3.223286 |
| 28 | 5.549657 | 7.679829 | 2.662714 |

er for JFRCC mortar panels with the combination of PFA and PC as matrix than those with the combination of GGBS and PC as matrix at 14 and 28 days but the values were very close. Rather, as aforementioned, the total energy absorbed by the JFRCC mortar panels with the combination of PFA and PC as matrix was considerably higher than those by the JFRCC ones with the combination of GGBS and PC as matrix at the same ages of 14 and 28 days.

## 4. Conclusions

Based on qualitative, semi-quantitative and quantitative analyses of fracture and impact test results of various JFRCC and plain concretes and mortars, the following conclusions can be drawn:

(1) JFRCC with GGBS/PC matrix achieved higher compressive strength, splitting tensile strength, and flexural strength than that with PFA/PC matrix. It also demonstrated higher fracture toughness, critical strain energy rate, and critical stress intensity factor than JFRCC with PFA/PC matrix and plain concrete with GGBS/PC matrix. But the plain concrete exhibited highest modulus of elasticity among the three concretes.

(2) Plain concrete exhibited higher fracture toughness, critical strain energy release rate, and critical stress intensity factor than JFRCC with PFA/PC matrix at early ages up to 28 days due to the contribution of GGBS replacing PFA in matrix.

(3) JFRCC mortar panels with PFA/PC matrix possessed higher impact resistance than those with GGBS/PC matrix. The former also absorbed more impact energy and survived more impact blows upon failure than the latter at ages of 14 and 28 days. But both of them exhibited much higher impact resistance, absorbed much more impact energy and survived more impact blows than the plain mortar panels.

(4) Jute fibres exhibited less deterioration in PFA/PC matrix than in GGBS/PC one. Fibre pull-out was observed in JFRCC mortar panels with PFA/PC matrix while fibre fracture in those with GGBS/PC matrix upon impact failure.

## Acknowledgement

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