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Research Article

Properties of steel fiber self-compacting concrete incorporating quarry dust fine powder

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ABSTRACT

Self-compacting concrete (SCC) has great potentials as it offers several environmental, economic and technical benefits. Moreover, the use of fibers extends its possibilities since fibers arrest cracks and retard their propagation. Incorporation of Quarry Dust (QD) in SCC help to reduce environmental hazards during the production of QD. This study evaluated the fresh and hardened properties of steel fiber self-compacting concrete (SFSCC) incorporating QD. The optimum fiber and QD contents with no adverse effects on fresh and hardened properties were determined. A comparative study on behavior of SCC and SFSCC mixtures in terms of workability, compressive strength, compressive strength development ratio, tensile, flexural and energy absorption capacity was carried out. Test results showed that compressive strength increased with increase in QD contents at fixed fiber content by mass of Portland cement (PC) and then decreased. Strength development ratio (C28/C7) for SCC was 1.13, while it was 1.06, 1.08, 1.10 and 1.01 after reinforcing with 0.10, 0.20 and 0.30 contents of fiber. The compressive, tensile, flexural and energy absorption capacity or Toughness of SFSCC increased with the inclusion of the aforementioned contents of steel fiber up to 0.20 % volume of total binder at constant QD content and then decreased when compared with control SCC values. From these results, optimum value for the variables studied was obtained from mix QD₂₀ + 0.2fr. Hence, steel fiber and QD could be successfully used in SCC production not minding the slight drawback on workability of SCC caused by inclusion of steel fiber, but with a modified dosage of super-plasticizer (SP), fresh and hardened properties, in accordance with specifications in relevant code(s) can be achieved.

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1. Introduction

Self-compacting concrete (SCC) is concrete that is self-compacted that flows under its own weight through restricted concrete sectional forms without segregation and bleeding. To achieve this, it should have a relatively low yield value to ensure high flow ability, a moderate viscosity to resist segregation and bleeding so as to maintain homogeneity during transportation, placing, curing to ensure adequate structural performance and long term durability. As stated by Alden (2014), SCC can be classified into three types: the powder type, viscosity agent type and the combination type. The powder type

SCC is characterized by the large amounts of powder (all material < 0.15) which is usually in the range of 550 to 650 kg/m³. This provides the plastic viscosity and hence the segregation resistance. The yield point is determined by the addition of SP. Due to the high content of powder, SCC may exhibit more plastic shrinkage of creep than ordinary concrete mixes, however, this can be overcome with the addition of fibers. For the viscosity type SCC, the powder content is lower (350 to 450 kg/m³). The segregation resistance is mainly controlled by viscosity modifying Admixture (VMA) and the yield point by addition of SP. In the combination type of SCC, the powder content is between 450 to 550 kg/m³ but in addition, the rheology

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is also influenced by a viscosity modifying agent (VMA) as well as an appropriate dosage of the SP (EFNARC, 2002). Introduced in the late 1980s by Japanese researchers, Nagamoto and Ozawa (1997), if well designed ensures a good balance between deformability and stability. This is achieved by careful mix proportioning of the constituents. The use of SCC has/off-set a growing shortage of skilled labor and has proved beneficial economically due to its advantages such as faster construction, reduction on site manpower, better surface finishes of structural elements, easier placing, improved durability, reduction in noise level and others just to mention but a few. It is obvious that, SCC, being a new type of concrete with two incompatible properties – high flow ability and segregation resistance and with an improved performance still has a draw-back yet to be addressed – weak in tension resistance. To overcome this, utilization of fibers essentially for bridging cracks has attracted the attention of researchers. Parameters such as fiber type, volume content, size, shape and aspect ratio affects properties of SCC and reduce its workability (Poon et al., 2007; Aydin, 2007). Steel fibers have a substantially high strength and high young modulus of elasticity compared with other types of fibers. This may lead to an enhanced flexural rigidity and has great potential for crack control despite its high volumetric density. Also, it is conductive in SCC constituents in terms of reduction in volume ratio of aggregate to cementitious material (Okamura, 1997), increase in paste volume and water-binder ratio, controlling maximum coarse aggregate particle size and the use of viscosity enhancing admixtures such as fly ash, silica fume, Limestone powder and others. These steps taken in conjunction with EFNARC specifications normally yield an acceptable SCC mix. Meyer et al. (1996) and Narayan and Nathan (2009) compared properties of SCC containing QD and that of conventional concrete so as to ascertain their performance which showed SCC with QD having better properties over that for conventional concrete. It is conductive in electric and magnetic fields and thus its content must be reduced to a certain level (Rao and Ravindra, 2010; Grunwald and Walraven, 2001). Their work on the effect of four types of steel at varying contents on the workability of two SCC mixes with and without fibers showed that the flow behavior of fiber reinforced mixtures differs from that of plain SCC. Their observations also indicated that when a homogeneous distribution is achieved with critical fiber content, the formation of its structure of the granular skeleton has a high measure of absorption energy during its deformation.

Toughness is a measure of the ability of the material to absorb energy during the deformation estimated using the area under the stress-strain curve or Load-deflection curve (Balaguru and Shah, 1992; Dawood and Ramli, 2009a). The toughness of plain concrete is low because it cannot absorb more energy without interruption. In other words, toughness is a property that consists of high resistance and ductility. A material that can absorb shaking and shocking without breakdown or failure is said to have a high toughness, unlike plain concrete which is brittle because of its low plastic deformation before failure (Hadian, 1966). Low failure toughness of plain concrete in service is a significant deficiency which

can be off-set by the incorporation of discontinuous fiber in the concrete. The main contribution of steel fiber to concrete is observed after matrix cracking. If a proper design is made, after the matrix cracking, randomly distributed, short fiber in the matrix arrest micro cracks, bridge these cracks, undergo a pull-out process and limit crack propagation (Banthia and Trottier, 1995; Kurihara et al., 2000); thus, improving the toughness of the concrete, Nataraja et al. (1999), Banthia and Sappakittipakorn (2007). Dawood and Ramli (2000b) studied the effect of steel fiber on toughness. The study showed that the toughness improves with the increasing content of fiber, the reason being the ability of fiber in arresting cracks at both micro and macro levels. At micro-level, fibers inhibit the initiation of cracks, while at macro-level fiber provide effective bridging and impart sources of toughness and ductility. The concrete brittle behavior is then replaced with a non-brittle fracture behavior related to a post-cracking bearing capacity. So, steel fibers are being increasingly employed in industrial floors, road pavements, tunneling and channels linings among other applications. Di Prisco et al. (2009) stated that one of main reasons for this increasing use of replacing entirely or partially conventional reinforcement in structural applications is due to its benefits such as savings in labour and crack control. The addition of steel fiber as reported by Nataraja et al. (1999), has significantly improved many of the engineering properties of mortar and concrete, notably impact strength and toughness. Flexural strength, fatigue strength, tensile strength and the ability to resist cracking, spalling are also enhanced. The work of Mohammadi et al. (2008) showed the influence of mixed aspect ratio of steel fibers on the fresh and hardened properties of fibrous concrete. Further studies on the use of fibers and pozzolans in SFSCC shows that the parameters influencing compressive strength differs from the one that influences flexural toughness of fibrous concrete.

The fresh and hardened properties of SFSCC aforementioned need to be assessed. European Federation for specialist construction chemicals and concrete system (2002) specified tests and test values to be used in assessing the fresh properties of SCC while relevant codes of practice for testing mechanical properties of concrete are available. These were used to test and obtain values in assessing the fresh and mechanical properties of the SFSCC. Researchers have reported results of investigations on various aspects of SCC and the use of fibers in concrete. Steel fiber used in this study was obtained from worn out tires littered at refuse dumps which is an eye sore. Their disposal is a challenge. Currently, the only means of disposal is by burning which pollutes the air constituting health hazards to all and sundry. The steel wires used as fibers for concrete reinforcement is the framework of a vehicle tire considered as waste but utilized more purposeful thus reducing the health hazard caused by burning them. Furthermore, at Quarry sites, heaps of QD powder can be seen here and there and in most cases end up in landfills. Apart from the heaps being an environmental threat to health, its dumping as landfills is a breach of the ecosystem. To overcome this problem, QD is introduced as a partial replacement of PC in SFSCC. This is an attempt to further reduce cost with

less PC used in the study. It is therefore imperative to investigate the fresh as well as hardened properties of SFSCC containing QD powder, highlighting the benefits of SFSCC incorporating QD both in the fresh and hardened states as well as its potentiality as a structural load resisting element. Recent works by researchers to enhance mechanical properties of fiber has attracted attention world- wide; but yet to be extended to SCC. This is not unconnected with the negative effect inclusion of fiber in concrete has on its workability which can be offset using SP. The incorporation of a pozzolana in SFSCC and how it affects its properties is also another perspective to be fully explored. However, it is not clear if SCC blended with QD and steel fiber will behave like plain SCC? Thus, the study evaluated the fresh and mechanical properties of steel fiber (obtained from worn out tires) self-compacting concrete incorporating QD with a view to obtaining optimum content for steel fiber and QD and its effect on the fresh and hardened properties.

2. Materials and Method

2.1. Materials

2.1.1. Cement

Portland cement used in all mixtures is a locally available product of grade 41 N manufactured by Dangote Cement Company Plc Nigeria which conformed to BS EN 196-6; 1997 as tested during the study. Physical properties of PC and QD are shown in Table 1.

Table 1. Properties of cement and quarry dust (QD).

Property	PC	QD
Colour	gray	off-white
Specific gravity	3.15	2.39
Spec surface area (cm ² /g)	3000	4580
Soundness	4.8	
Setting times (mins)		
Initial	161	172
Final	202	230

2.1.2. Steel fiber

Steel fiber used for the study was obtained from worn out tires, circular section, mean diameter of 0.5 mm and cut into length of 20 mm was in four proportions of 0%, 0.1%, 0.2% and 0.3% by mass of binder with an aspect ratio of 40 (Fig. 1). It was optimized by conducting slump flow tests for various mixes in accordance with EFNARC specifications. The SFSCC mix with a volume fraction of steel content and a slump flow with at least 600 mm spread diameter with homogeneous distribution of the fibers was considered as the optimum volume fraction of the steel fiber.



Fig. 1. Steel fibers from worn out tires.

Crushed granite of 10 mm and 19 mm maximum nominal sizes with specific gravity of 2.60 and 2.66 with average fineness modulus of 6.86 was used. Locally available sharp river sand with a specific gravity of 2.39, a fineness modulus of 2.75 with particle size distribution (PSD), (Fig. 2) was used. QD obtained from a local quarry site in Minna, Niger state, Nigeria used for the study was sieved with a 150 μ m sieve. A Poly-carboxylic ether (master Glenium ACE 456) based SP which conforms with ASTM C 494, type F was used.

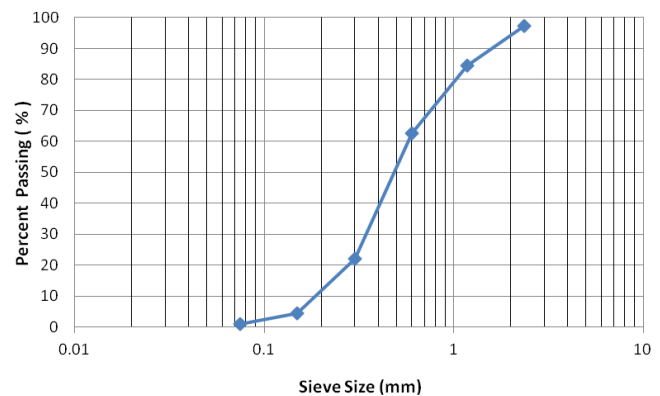


Fig. 2. Particle size distribution for sand.

2.2. Mix proportion

Selection and proportion of SFSCC constituents were based on principles and EFNARC guidelines. Furthermore, the rational approach proposed by Okamura and Ouchi (1998) for mix design was adopted. In other words, solid materials contents and water/binder ratio were fixed while SP dosage was adjusted so as to achieve optimum viscosity and flow ability. Mix proportions for SFSCC containing QD was also based on packing density approach. Maximum packing density was achieved by aggregate blending (19 mm and 10 mm) and the powder phase (PC + QD) using slump flow cone studies in accordance with EFNARC specifications, 2002. A total of nine (9) concrete mixtures were used with cementitious ma-

materials content, cement and QD replacement of PC at different levels and the mixes evaluated using slump flow tests. The bulk specific gravity, dry- rodded unit weight (DRUW) of the coarse aggregates blending 30:70 (19 mm and 10 mm) attained in accordance with EFNARC

specifications is 1477.33 kg/m³ and with percentage blending of coarse aggregate is 50.14. Volume of sand used for the study is 42% of mortar volume. A constant W/B ratio of 0.40 was used for all the mixes. Details of mix proportions is as shown in Table 2.

Table 2. Mix proportions.

Mix ID	Total Binder	PC	QD	CA	FA	Water	SP	W/B
SCC	400	400	xx	741	848	160	2.0	0.4
QD ₁₀ +0.1fr	400	360	40	741	848	160	2.2	0.4
QD ₂₀ +0.1fr	400	320	80	741	848	160	2.22	0.4
QD ₃₀ +0.1fr	400	280	120	741	848	160	2.25	0.4
QD ₁₀ +0.2fr	400	360	40	741	848	160	2.30	0.4
QD ₂₀ +0.2fr	400	320	80	741	848	160	2.32	0.4
QD ₃₀ +0.2fr	400	280	120	741	848	160	2.33	0.4
QD ₁₀ +0.3fr	400	360	40	741	848	160	2.35	0.4
QD ₂₀ +0.3fr	400	320	80	741	848	160	2.36	0.4
QD ₃₀ +0.3fr	400	280	120	741	848	160	2.37	0.4

N.B All materials are in kg/m³, SP = superplasticizer

2.3. Method

2.3.1. Fresh property tests

The study was conducted in two phases; based on mix proportions (Table 2), each mix constituents were measured and dry-mixed thoroughly. Seventy percent (70%) of water content was gradually added and then mixed for another three minutes. The remaining 30% water content was mixed with SP and gradually added to the mix and the mixing continued for another three minutes leading to the attainment of a homogeneous mix. Fresh property tests such as slump flow, T₅₀ and J-Ring were conducted to determine the filling and passing abilities of the mixtures in accordance with ASTM C 642-13 and EFNARC specifications. The slump cone placed on a flat base was filled with fresh SFSCC mix and the excess flushed with a straight edge. The cone was then lifted vertically off the base. The horizontal diameter flow of the fresh mix is a measure of the filling ability of the mix (Fig. 6). The measurement was done twice perpendicular to each other and an average value taken. The test was carried out for each mix. The T₅₀ test is the time taken (secs) the SFSCC mix takes to spread horizontally to 500 mm diameter which is a secondary indicator of the flow ability of the mix. For the J-Ring test, the slump cone was filled with fresh SFSCC, and the excess flushed with a straight edge. The cone is vertically lifted out of the J-ring earlier placed on the flat base and the fresh mix flows through the ring which acts as an obstruction to the flow (Fig. 7). The flow spread is also measured twice perpendicular to each other and the average value is a measure of the passing ability of the mix. The test was carried out for each mix.

2.3.2. Tests for hardened properties

On completion of tests for fresh properties, the SFSCC specimens were cast in steel cube mould (100x100x100 mm) for compressive strength test, cylindrical 150x150x150 mm in diameter for splitting tensile strength test and 100x100x500 mm prisms for flexural strength and absorption capacity. The test specimens were de-molded after 24 hours and cured in water for 7, 14, 28, 56 and 70 days respectively and then tested for the aforementioned variables. The tests were conducted in accordance with BS 811, ASTM C 496/C496M -04, ASTM C 293/C293M-10 and ASTM C 1609/C1609M respectively. For the compressive strength test, a compression testing machine of 2000kN capacity was used and each test result was an average of three cube specimens (Fig. 3).



Fig. 3. Compressive strength test.

For the tensile strength test, since it is difficult to determine the tensile strength directly, it was determined indirectly. The cylindrical concrete specimen (150x150 mm) was placed horizontally between the loading surfaces of the compression testing machine (Fig. 4). A compression load, P was applied diametrically and uniformly along the length of the cylindrical specimen until failure occurred along the vertical diameter. The tensile strength was determined using Eq. (1).

$$F_t = \frac{2PL}{DL} \quad (1)$$

where F_t is tensile strength, P is failure load, D is the diameter of cylindrical specimen and L is length of cylindrical specimen. The test was conducted in accordance with ASTM C 496 (2011).



Fig. 4. Tensile strength test.

For the flexural strength, when a structural element is subjected to a bending load, its resistance to the bending load is the flexural strength. The prisms were tested under third-point loading gradually to failure (Fig. 5).



Fig. 5. SFSCC prisms tested for flexural strength.

The flexural strength is expressed as the load acting on the cross section of the prism until failure as expressed in Eq. (2).

$$F_r = \frac{PL}{bd^2} \quad (2)$$

where F_r is flexural strength, P is failure load, L is span, b and d are width and depth of section of prism. Test result is an average value of two prisms.

SCC reinforced with steel fibers gives it additional ability to absorb larger amount of energy to withstand large deformations prior to failure. The absorbed energy is called its energy absorption capacity of the material or flexural toughness which is represented by the area under the load-deflection curve for a concrete prism subjected to a four-point or three-point loading test. This energy is a reflection to what extent the SFSCC can withstand loading conditions until failure. Sample prisms (100x100x500 mm) cast (2 nos) from each mix, on attainment of curing durations of 28 and 56 days were subjected to a third-point loading test set up for the evaluation of flexural toughness. A load-deflection curve is plotted for each prism and the area under the curve is the flexural toughness. The flexural performance of SFSCC prism incorporating QD was evaluated from the area under each curve in accordance with the works of Low and Beaudoin (1994), Jastrzebski (1997) and ASTM C 1609/C 1609M respectively as shown in Fig. 6.

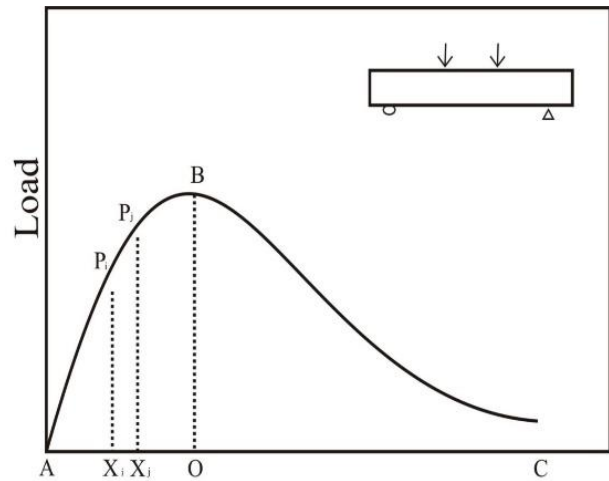


Fig. 6. Theoretical load-deflection curve.

Flexural toughness was determined as the entire area under the load-deflection curve derived from deflections and Loads. It is the sum of all finite area increments over the entire region under the ABC curve, which is work done leading to fracture which represents the flexural toughness of the test specimen. It was determined for each specimen using Eq. (3):

$$T = \sum \frac{[P_i + P_j]}{2} [x_j - x_i] \quad (3)$$

where P_i and P_j are two different loads applied for a finite increment and $[x_j - x_i]$ is the change in deflection within the finite increment, T is flexural toughness

3. Test Results and Discussion

3.1. Fresh properties

The results of fresh properties for all the mixes used for the study are shown in Table 3.

From the results in Table 3, slump flow values for SCC and SFSCC differs significantly. The SCC value of 730 mm when compared with that for mix QD10+0.1fr decreased to 722 mm (1.10%). This led to an increase in flow time from 3.72 secs to 3.93 secs (6.18%). For mix (QD10+0.2fr compared with control value experienced a flow spread decrease of 1.18%. This also led to a flow time increase of 8.87%. The trend continued with the rest mixes when compared with control values. This indicates that the addition of steel fiber and QD decreased workability

of SFSCC (filling and passing ability). It is an established fact that fibers reduce workability and with the addition of QD with high specific surface area (Blaine value=4580), Table 1 compared with that for cement (3000) absorbs large amount of water on the surface of the particles which produced negative effect on flow ability of fresh SFSCC. Results of J-Ring spread flow and T₅₀ for J-ring spread flow also followed the same trend with that of the slump flow spread. All the mixes except QD30+0.3fr met EFNARC and Brite EuRAM specifications. The result has shown that for SFSCC mixes, QD and steel fiber additions to SCC resulting to a decrease of flow spread greater than 25% of control value and an increase in flow time of at least 50% of control value does not meet EFNARC and Brite-EuRAM specifications.

Table 3. Slump flow, J-Ring and T₅₀ test results.

S/no	Mix ID	% Decrease in SF	Slump flow (mm)	T ₅₀ (secs)
1	SCC		730	3.72
2	QD10 + 0.1fr	(1.11)	722	3.95
3	QD20 + 0.1fr	(1.18)	721.50	4.05
4	QD30 + 0.1fr	(1.28)	720.80	4.42
5	QD10 + 0.2fr	(2.10)	715	4.80
6	QD20 + 0.2fr	(2.96)	709	5.21
7	QD30 + 0.2fr	(13.71)	642	6.85
8	QD10 + 0.3fr	(21.67)	600	8.76
9	QD20 + 0.3fr	(26.30)	578	10.52
10	QD30 + 0.3fr	(47.47)	490	12.85



Fig. 7. J-Ring flow spread.



Fig. 8. Slump flow spread.

3.2. Hardened properties

3.2.1. Compressive strength

The compressive strength for 7, 14, 28, 56 and 70 days for all mixes are shown in Table 4.

It can be observed that there is increase in compressive strength for all mixes with increase in age of curing. However, when control mix values are compared with other mixes for 7 and 14 days, compressive strength of SCC is higher than that of other mixes. This is because strength development is mainly due to PC hydration.

Also, for mixes other than SCC, PC content is lower because it has been replaced with QD thereby lowering its mineral content such as C_3S responsible for faster hydration. But at 28 days, the compressive strength of other mixes picks up, equals or even surpasses the control values. This is due to the fact that in addition to PC hydration, the QD which replaced part of PC reacts with calcium Hydroxide ($Ca(OH)_2$), a by-product from the PC hydration to form more Calcium silicate hydrate which accounts for the increase in strength. The results also indicate that compressive strength increased with increase in QD replacement level of PC up to 20% by mass of PC and addition of steel fiber up to 0.20% by mass of PC and then decreased for all curing ages and for all mixes. The highest strength development occurred at 20 % replacement level and thereafter decreased. This is because at 0 to 20% replacement level, more hydration products are

formed as hydration continues leading to the formation of more cementing materials like C-S-H and C-S-A-H hydrates which give rise to increase in compressive strength for all the mixes (Taha et al., 1981). However, at 30 % replacement level, there is more QD volume content and less $Ca(OH)_2$; which reduce pH level of the solution and hinders further hydration, leading to reduction in strength. This is called the dilution effect. Also, with high content of QD and its high specific surface area, will require more water but since water binder ratio is constant, there is less water for hydration which leads to less hydration and decrease in strength. Furthermore, at the 30% replacement level, there is high fiber content, non-uniform distribution, balling and high specific surface area leading to interference in strength development. Optimum fiber content of 0.20fr by volume had already being attained at mix design process.

Table 4. Compressive strength for mixes.

Age (Days)	7	14	28	56	70
mix ID	Compressive strength (N/mm ²)				
SCC	19.87	20.52	24.78	25.86	30.63
QD10+0.1fr	17.95	19.42	24.98	26.38	30.83
QD20+0.1fr	18.67	20.71	24.93	29.80	31.14
QD30+0.1fr	16.93	17.13	23.02	23.75	24.48
QD10+0.2fr	18.24	19.46	24.57	25.02	26.53
QD20+0.2fr	19.19	20.22	24.95	26.12	36.78
QD30+0.2fr	16.93	17.13	21.02	23.75	24.48
QD10+0.3fr	15.72	16.44	20.49	22.11	23.62
QD20+0.3fr	15.58	16.01	19.51	21.48	22.49
QD30+0.3fr	15.25	16.72	20.88	21.88	23.72

3.2.2. Strength development – fiber content relationship

A comparison between strength development with time was examined so as to establish if steel fiber content enhances strength gain. The effect of fiber content on the maturity of SFSCC could be traced through the rate of strength gain. Fig. 9 shows the strength gain ratios for all the mixes in terms of fiber content. Strength development for SCC was 1.03 for C14/C7 and became 1.08, 1.10, 1.06 and 1.01 after reinforcing with steel fiber content of 0.1, 0.20 and 0.30 respectively. Compared with SCC value (1.03), the strength development values for SFSCC increased to a maximum and then decreased after attainment of optimum fiber content. This is the effect of fiber used. The same trend can be seen for series C28/C7, C56/C7 and C70/C7. It can be deduced that the tested SFSCC mixtures with different constituents did not show a pronounced negative effect on the maturity of concrete but rather led to higher strength gain. The same contribution to strength gain by steel fiber content for the mixes can be observed in Table 4.

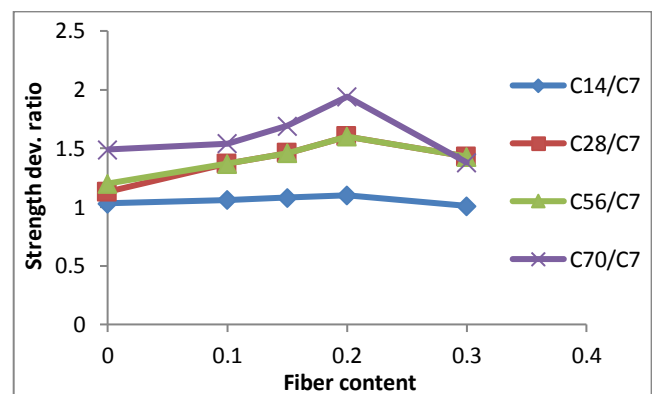


Fig. 9. Strength development ratio against fiber content.

The tensile strength of QD blended SFSCC is in Table 5. The results followed the same trend with that of compressive strength. This is because tensile strength is closely related to compressive strength and there is a theoretical relationship between them already established in literature.

Table 5. Tensile strength test results.

Age (Days)	7	14	28	56	70
mix ID	Tensile strength (N/mm ²)				
SCC	2.80	2.94	3.09	3.20	3.98
QD10+0.1fr	2.68	2.82	3.25	3.58	3.95
QD20+0.1fr	2.69	2.86	3.37	3.60	3.85
QD30+0.1fr	2.70	2.76	3.32	3.46	3.79
QD10+0.2fr	3.34	3.92	3.95	4.50	4.55
QD20+0.2fr	3.47	3.82	3.98	4.83	4.97
QD30+0.2fr	3.30	3.72	3.81	4.01	4.12
QD10+0.3fr	2.89	2.99	3.61	3.69	3.72
QD20+0.3fr	2.74	2.88	3.71	3.76	3.80
QD30+0.3fr	2.88	3.01	3.72	3.85	3.88

3.2.3. Flexural strength

Table 6 below shows the results of the flexural strength of the tested prisms. Trend in the result values is similar to that for tensile and compressive strength values aforementioned.

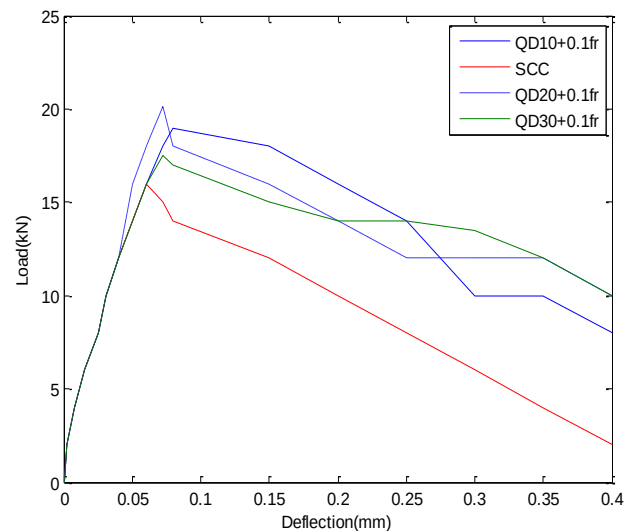
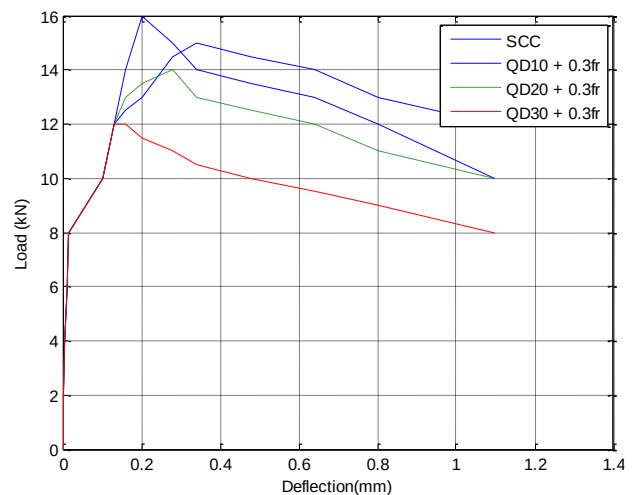
When the values of other mixes are compared with SCC values, they increase marginally with age of curing; For example, QD10+0.1fr increased by 0.77% at 28 days and 0.66% at 56 days and for QD20+0.2fr, there was an increase of 32% at 28 days and 32.34% at 56 days which is the optimum. These changes are due to same reasons given earlier for compressive strength values.

3.2.4. Energy absorption capacity

The load–deflection response of the miniature beams (prisms) tested for energy absorption capacity is as shown in Figs. 10-12.

Table 6. Flexural strength for mixes.

Age (Days)	28	56
mix ID	Flexural strength (N/mm ²)	
SCC	9.06	9.10
QD10+0.1fr	9.13	9.16
QD20+0.1fr	9.22	9.35
QD30+0.1fr	9.12	9.16
QD10+0.2fr	10.25	10.35
QD20+0.2fr	13.30	13.39
QD30+0.2fr	12.07	12.32
QD10+0.3fr	10.12	10.19
QD20+0.3fr	9.42	9.78
QD30+0.3fr	9.12	9.25

**Fig. 10.** Load-deflection response for prisms (SCC, QD10+0.1fr, QD20+0.1fr and QD30+0.1fr).**Fig. 11.** Load-deflection response for prisms (SCC, QD10+0.3fr, QD20+0.3fr and QD30+0.3fr).

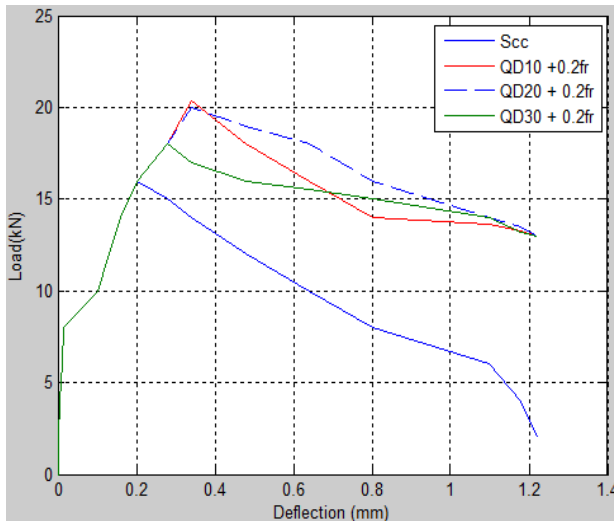


Fig. 12. Load-deflection response for prisms (SCC, QD10+0.2fr, QD20+0.2fr and QD30+0.2fr).

The flexural absorption capacity for each prism was calculated from their Load-deflection curves using Eq. (3) derived from Fig. 5 as stated in the works of Low and Beaudion (1994), and Jastrzeski (1997). These values are provided in Table 7.

Table 7. Flexural absorption capacity (T_f).

mix ID	Flexural absorption capacity (Nmm)
SCC	9.06
QD10+0.1fr	9.13
QD20+0.1fr	9.22
QD30+0.1fr	9.12
QD10+0.2fr	10.25
QD20+0.2fr	13.30
QD30+0.2fr	12.07
QD10+0.3fr	10.12
QD20+0.3fr	9.42
QD30+0.3fr	9.12

From Table 7, the energy absorption capacity, (T_f), for SCC is 3231 Nmm due to its low tensile strength and strain capacity. The SCC prism failed immediately on attainment of load capacity. Prisms of other mixes also failed abruptly, though, after an appreciable increase in load. None of the prisms failed in a ductile manner. Compared with the values of other mixes indicated an increase in T_f (enhanced by addition of steel fiber and QD) of 34% for Mix QD10+0.1fr, 38% for QD20+0.1fr and 35% for QD30+0.1fr. For Mixes with constant 0.2% fiber content and a stepwise increase of 10% QD up to 30%, increased by 81%, 85% and 77% while for mixes with 0.3% constant fiber content with stepwise increase 10% QD up to 30 %, increased in T_f by 76%, 73% and 69% respectively. In the same vein, at constant contents of QD and varying percentage fiber contents, similar results for

T_f were obtained for all the mixes, with Mix QD20+0.2fr again attaining the highest percentage increase (85%) in T_f . Changes in the mixes giving rise to increase in T_f can be attributed to nucleating action and pozzolanic reactions of QD on one hand and the bridging ability of steel fibers which delay crack occurrence on the other hand; and for the decrease in T_f in some mixes can also be attributed to dilution effect of QD, balling, non-uniform distribution and poor orientation of the steel fibers (Altun et al., 2007).

4. Conclusions

SCC concrete with slump flow of 730 mm, SFSCC mixtures containing QD with varying slump flow with a constant w/b ratio of (0.40), slightly modified SP content with cement content 280 to 400 kg/m³ was studied. Results and discussion of the study indicated that:

- Proportioning of constituents of SFSCC containing QD and steel fiber led to the development of SFSCC with high fresh and hardened properties. The inclusion of steel fibers slightly affected the fresh properties as evidenced in the values measured for both filling and passing abilities of SFSCC (Table 3).
- Compressive strength of SFSCC increased with increase in fiber content up 0.20% and then decreased. This is further strengthened as evidenced in the strength development – fiber content relationship.
- The inclusion of steel fibers enhanced the mechanical behavior of SFSCC as it improved the compressive strength which ranged from 1 to 10% at 14 days, 18 to 22% at 28 days, 5 to 16% at 56 days and 5 to 29% at 70 days with varying contents of steel fiber.
- Inclusion of QD improved the strength of SFSCC especially for values at latter ages (28 to 70 days thus contributing to reduction in cost of cement.
- There was no indication of a pronounced negative effect on maturity of concrete from the tested SFSCC mixtures, but rather led to a higher strength gain. The strength development factor for control mixture was 1.13 for C14/C7, and decreased to 1.06, 1.08, 1.10 and 1.01 after being reinforced with 0.10, 0.20 and 0.30 contents of fiber. It improved and ranged between 1.37 to 1.43 and 1.54 to 1.69 for C56/C7 and C70/C7 respectively.
- In the production of SCC mixtures beside steel fiber, the use of QD extends their technical and environmental benefits as it minimized air pollution and health hazards resulting from its production.
- Optimum mix obtained from the study is QD20+0.2fr which yielded highest percentage increase in nearly all the variables studied.
- Flexural energy absorption capacity, T_f , increased from 34% to 85% of SCC.
- The study was carried out at a constant w/b ratio (0.40) with a modified SP dosage, without considering effect of fiber orientation and even distribution. Further investigation is required to consider effect of aforementioned variables on the behavior of SFSCC incorporating QD.

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Research Article

Destructive and non-destructive testing of bronze-waste tire-concrete composites

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ABSTRACT

In this study, the effects of finely-milled bronze and waste tire on the mechanical properties of concrete have been investigated. Approximately 2.5% and 5% by weight for each additive (bronze sawdust and waste tire) were added to dry concrete. The open porosity, density, compressive strength values of cured concrete have been determined. In addition, the Schmidt rebound hammer (SRH) and the ultrasonic pulse velocity (UPV) tests, which are non-destructive test methods, were applied. The microstructure and fracture surfaces of these materials were characterized by scanning electron microscopy (SEM). It was observed that the density of pure concrete was 2.35 g/cm³ while the density was 2.19 g/cm³ for a C+5%B+5%T material. Similarly, pure concrete had an almost three times better compressive strength and a two times better SRH value than those of the C+5%B+5%T material. The density and mechanical properties of concrete materials containing bronze and waste tire decreased due to micro crack formations, weak bonding and deep cracks forming especially between the concrete and additives.

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1. Introduction

Concrete is the most widely used construction material around the world. The demand for concrete was rapidly increased as a result of the population growth. This leads to increase the demand for the construction raw materials consist of two or more materials with improved performance and the research on innovative structural designs. The mechanical properties and damage tolerance of brittle material can be improved by using interlocking elements as structural components. The interfaces of the elements also prevent catastrophic crack propagation (Mohammed and Najim, 2020; Javan et al., 2020).

Different types of concrete are needed to meet different engineering environment. And many recent innovations in advanced concrete materials in order to produce concrete with exceptional performance characteristics (Shang et al., 2014).

There are several factors that make cement and cement-reinforced structures unsuitable, such as the poor resistance of the steel reinforcement to deicing salts and marine environments and corrosion in the natural environment. Similarly, poorly-designed concrete structures are unable to withstand natural disasters. As a result, various alternatives to reduce the environmental impact and make the construction industry sustainable and environmentally friendly have been investigated. Traditional fibers, such as steel, glass, carbon, polypropylene, polyvinyl alcohol, and other fibers have also been studied as potential reinforcements. Fibrous reinforcements provide concrete with improved mechanical performance and/or resistance to environmental conditions. The fibers make concrete more flexible and able to withstand stresses (Anandamurthy et al., 2017).

The toughness was greater in concrete where waste was used because the cracks were controlled due to the

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bridging of the fibers, with the waste becoming more ductile. The fiber waste gave better mechanical results than waste used in the form of pieces (Garrick, 2005).

The concrete that was produced at 10-15% by weight of granulated tire/rubber waste material had an approximately 60% better freeze-thaw resistance at 300 cycle performance than that of concrete produced by an admixture of 20-30% waste (Savas et al., 1996).

In this study, the mechanical properties of bronze and waste tire containing concrete were investigated.

2. Materials and Methods

In this study, C30 class ready-dry concrete, finely ground CuAl₁₀Ni bronze sawdust and waste tire were used. Then pure concrete, concrete incorporating 2.5% bronze sawdust and 2.5% waste tire (by wt.), and concrete incorporating 5% bronze sawdust and 5% waste

tire (by wt.) were produced in the shape of 5cm³ cubes. After this, all the concrete was cured for 28 days.

Densities and open porosities were determined by the Archimedes principle, according to the ASTM C642 test standard. A compressive test (ASTM C109/C109M), which is a destructive test method, the Schmidt method, which is the most common method used for non-destructive testing of concrete (ASTM C805), and a UPV test, which is another non-destructive test method (ASTM C597), were performed.

3. Results and Discussion

3.1. Physical and mechanical properties

Density, open porosity and strength values of pure concrete and concrete incorporating waste tire and bronze are given in Table 1.

Table 1. Density, open porosity, compressive strength values and SRH and UPV results of bronze-waste tire-concrete composites (C: pure concrete, B: Bronze sawdust, T: waste tire).

Material	Density (g/cm ³)	Open porosity (%)	Compressive strength (MPa)	SRH (MPa)	UPV (m/s)
C	2.35	8.78	63.64	53.57	4567
C+2.5%B+2.5%T	2.25	11.22	35.24	40.30	4165
C+5%B+5%T	2.19	11.18	20.47	29.92	3904

According to Table 1, the density values decreased and the amount of open porosities increased. Therefore, the compressive strength values decreased through the incorporation of waste tire and bronze due to weak bonding and deep cracks forming between the tire, concrete and bronze grains, as well as large amounts of micro cracks forming, even though the tire and bronze material were ductile and had a high capacity for absorbing plastic energy under compressive loading (Hernandez et al., 2003; Topcu, 1995; Papakonstantinou et al., 2006; Snelson et al., 2009; Ali et al., 2008). Waste tire grains could absorb some water during curing, tire grains could swell, and then cause micro cracks to form. In addition, ceramic concrete, metal bronze additive and polymer waste tire had different shrinkage behavior, which could also cause cracks to form and a decrease in density and strength values.

Additionally, according to the non-destructive tests results, strength obtained from the SRH method and the speed of the sound wave results (UPV) indicate that the decrease in strength was more significant as the amount of waste tire increased and, consequently, the speed of sound decreased due to the greater amount of porosity. The UPV method is a non-destructive testing method based on measuring the velocity of compression stress waves (P-waves). The ultrasonic pulse velocity depends on the density of the material being tested; the higher the density of the concrete means the higher the pulse velocity (Manish, 2006).

3.2. Microstructural analysis results by SEM

In a microstructure of C+5%B+5%T composite material according to 1000x significance of SEM result (Fig. 1), porosities, micro cracks, and deep cracks due to weak bonding between the concrete matrix, bronze and waste tire additives grains were observed. These defects caused a decrease in strength, SRH and UPV values (Table 1). It was observed that concrete grains (gray color) had a more compacted structure, as seen on the left of the microstructure image (Fig. 1-a) than those of regions where the bronze and waste tire additives were located (as seen on the right of microstructure image).

According to an SEM-mapping analysis, it was observed that additives were non-homogenously distributed in concrete, mainly Ca, Si and Al elements that was contained in the cement phase (gray color grains). The C element that came from waste tire was seen as black. The distributions of Cu, Al, and Ni elements were in the same regions, which indicate that these grains were bronze seen as white (Fig. 1-c-n).

According to the fracture surface images of the concrete materials (Fig. 2), trans-granular (indicated as blue arrow) and inter-granular (indicated as red arrow) fracture types were both seen in pure concrete (Fig. 2-a), while a dominantly trans-granular fracture type was observed in the C+5%B+5%T concrete material (Fig. 2-b). This transition of fracture type is thought to be the reason for the decrease in mechanical properties.

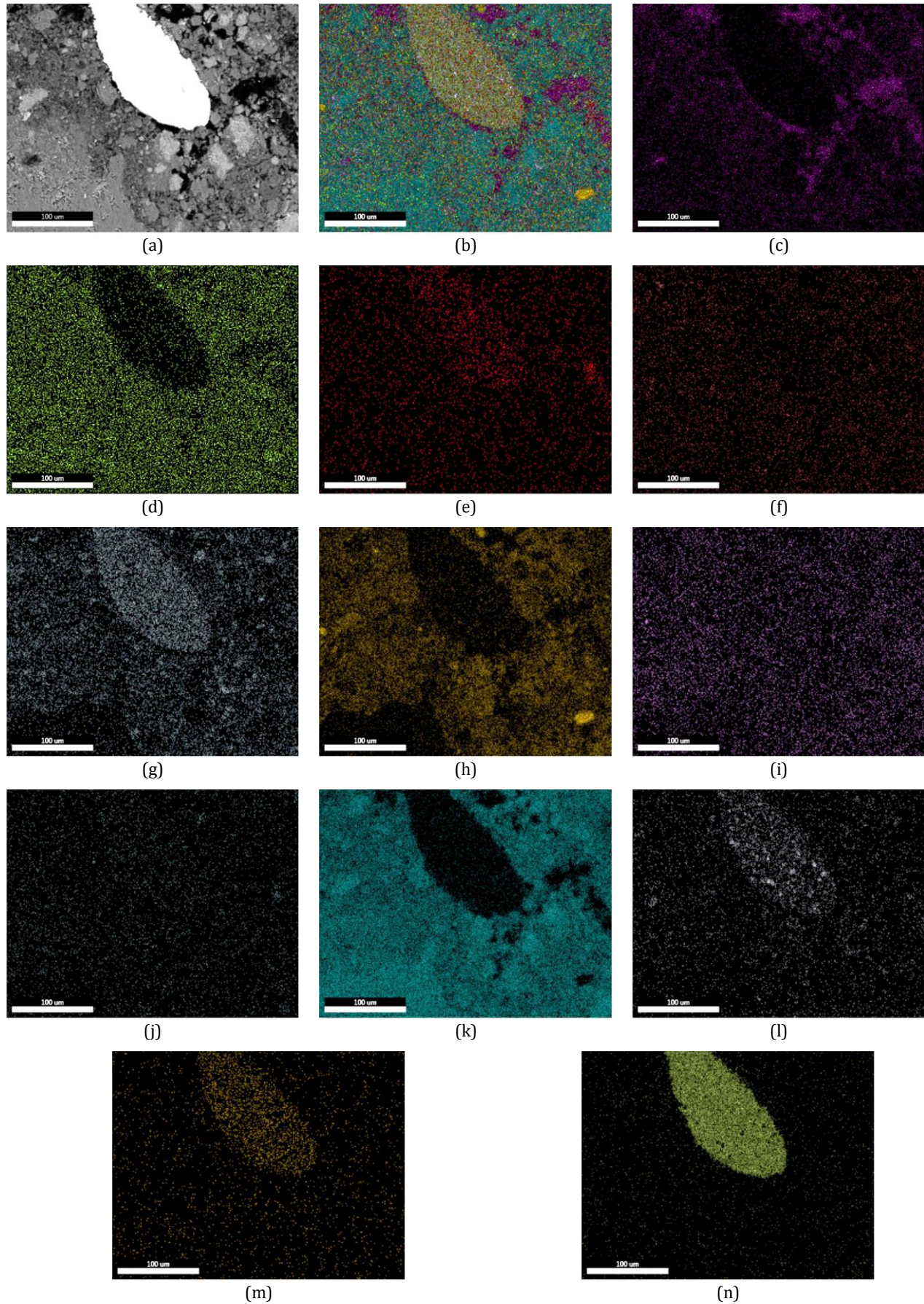


Fig. 1. 1000X-SEM back scattered electron image:

a) Microstructure; b) Colored microstructure of C+5%B+5%T composite and distribution of elements;
c) C; d) O; e) Na; f) Mg; g) Al; h) Si; i) S; j) K; k) Ca; l) Fe; m) Ni; n) Cu.

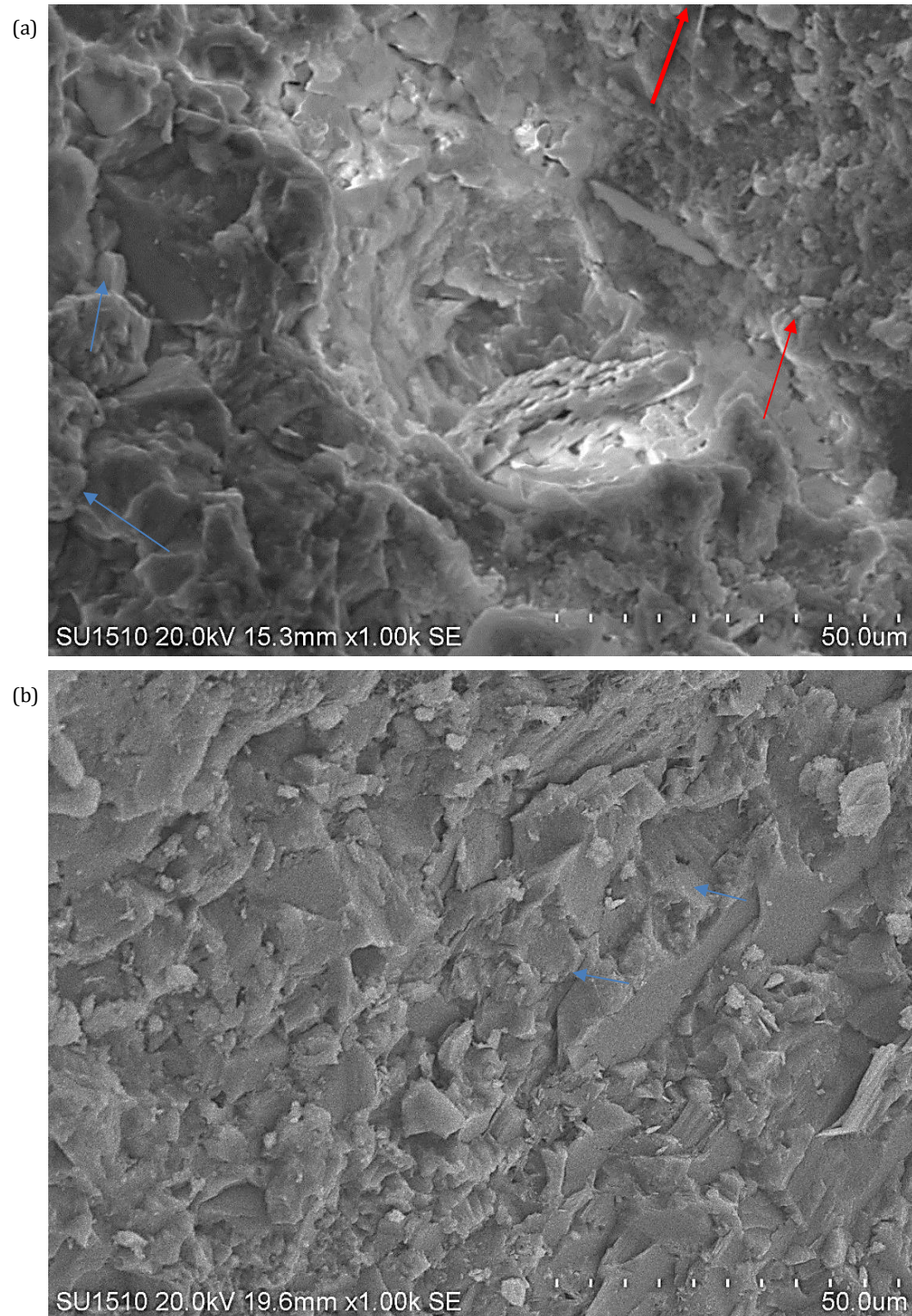


Fig. 2. Fracture surface images of a) C and b) C+5%B+5%T composite materials.

4. Conclusions

Concrete, incorporating bronze and especially waste tire, has lower density and compressive strength, SRH and UPV values than pure concrete due to weak bonding between the tire, concrete and bronze grains.

Transition from trans-granular fracture type to inter-granular fracture type is effective in an increase of mechanical properties.

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Research Article

Exploring optimum percentage of fly-ash as a replacement of cement for enhancement of concrete properties

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ABSTRACT

Researchers and decision makers are continuously looking out to determine the potential and effectiveness of fly-ash as a partial replacement of cement in concrete. The current study is carried out to check the optimum or nearly optimum quantity of fly-ash with which cement should be replaced to get most of the properties of concrete enhanced and to give the idea about the quantities of fly-ash that can be used in a better way and better cause so that a proper management scheme of its usage and disposal can be implied. Further, a comparison is given between normal concrete and fly-ash concrete to show the properties which can be enhanced by proper utilization of fly-ash as a partial replacement of cement. After carrying out the lab experiments, it has been seen that the replacement of fly-ash in concrete has resulted in general increase in compressive strength, flexural strength and splitting tensile strength up to 15% replacement and after then the strength is decreased considerably than that of normal concrete. Addition of fly-ash in concrete has resulted in decrease in the water absorption of concrete and hence decreases in permeability of concrete. There is a progressive increase in workability with increase in percentage of fly-ash in concrete. The current study has led to a conclusion that in order to achieve best results in use of fly-ash concrete, the fly-ash used for replacing cement in concrete should have the required properties as specified by the standards and proper techniques of processing fly-ash as well as mixing of fly-ash with cement must be employed.

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1. Introduction

First use of fly ash in concrete started in the United States in the early 1930's at University of California by Davis (1937). The major breakthrough in using fly ash in concrete was the construction of Hungry Horse Dam in USA in 1948, utilizing 120,000 metric tons of fly ash by Thomas (2010). This decision by the U.S. Bureau of Reclamation paved the way for using fly ash in concrete constructions. In India, Fly-ash was first used in the construction of Rihand Dam in Uttar Pradesh in 1962, replacing cement up to 15% showed in literature review by Bendapudi and Saha (2011). The Indian Standards IS:3812-1981 define, "Fly ash as a finely divided residue

resulting from the combustion of ground or powdered bituminous coal or sub-bituminous coal (lignite) and transported by the flue gases of boilers fired by pulverized coal or lignite." A pozzolana is a siliceous or siliceous/ aluminous material which possesses little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically reacts with Calcium Hydroxide liberated on hydration at ordinary temperature, to form compounds possessing cementitious properties explained by Gray and Lin (1972). Fly ash is best known and one of the most commonly used pozzolans in the world. We are using fly ash in almost all the activities where cement is involved like cement concreting, brick manufacturing, cement mortar for brick work & plastering.

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Fly-ash is one of the residues generated in the combustion of coal. Fly ash is generally captured from the chimneys of coal-fired power plants and is one of two types of ash that jointly is known as coal ash; the other bottom ash is removed from the bottom of coal furnaces. Depending upon the source and makeup of the coal being burned, the components of fly ash vary considerably but all fly ash include substantial amounts of silicon dioxide (SiO_2) (both amorphous and crystalline) and calcium oxide (CaO). Toxic constituents include arsenic, beryllium, boron, cadmium, chromium, chromium VI, cobalt, lead, manganese, mercury, molybdenum, selenium, strontium, thallium, and vanadium, along with dioxins and PAH (Poly-Aromatic Hydrocarbons) compounds. Fig. 1 shows the photomicrograph showing fly-ash particles.

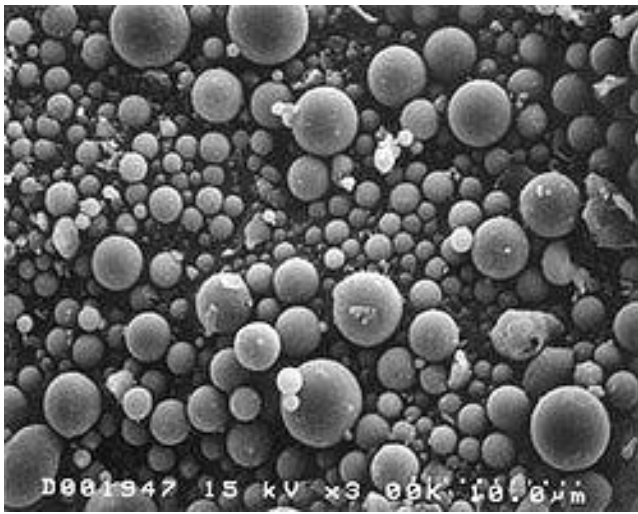


Fig. 1. Photomicrograph made with a Scanning Electron Microscope (SEM): Fly-ash particles at 3,000 x magnification (Credit: United States Department of Transportation - Federal Highway Administration).

In past fly ash was generally released into the atmosphere but pollution control equipment mandated in recent decades now requires that it be captured prior to release. In United States, fly ash is generally stored at coal power plants or placed in landfills explained by Subramani and Sakthivel (2016). About 43 percent is recycled, often used to supplement Portland cement in

concrete production. It is increasingly finding use in the synthesis of geo-polymers and zeolites. Fly-ash can also be used as a supplementary material for soil stabilization shown in literature review of Brooks (2009) and reclamation of saline soils shown by Ors et al. (2015). It is expected that use of fly-ash instead of lime in agriculture can reduce net CO_2 emission, thus reduce global warming also explained by Basu et al. (2009). High-Volume fly-ash concrete (HVFA) concrete is more suitable for the construction of rural, regional, and the national network of roads in India explained by Desai (2004). Studies show that partial replacement of concrete by fly-ash improves the concrete properties by considerable percentage explained by Marthong and Agrawal (2012).

2. Objectives of the Current Research

In this study, quantity of cement in concrete was replaced by 5%, 10%, 15%, 20%, 25 and 30% by weight of fly-ash and a comparative study was carried out:

1. To find-out the potential of fly-ash as a possible partial replacement of cement in terms of its binding property.
2. To check the optimum or nearly optimum quantity of fly-ash with which cement should be replaced to get most of the properties of concrete enhanced.
3. To compare the short-term and long-term strengths of normal concrete and fly-ash concrete.

3. Materials and Methods

Materials obtained for making concrete (cement, sand and coarse aggregate) were tested for their respective properties according to the respective IS codes. The size of the specimens and specifications of materials used are shown in Tables 1 and 2 respectively.

Table 1. Size of specimens used for testing of concrete.

S. No	Specimen	Size
1	Cube	(150 × 150 × 150) mm ³
2	Beam	(100 × 100 × 500) mm ³
3	Cylinder	(150mm ø, 300 mm height)

Table 2. Specifications of materials used in testing.

S. No	Material	Specifications
1	Cement	43 Grade (Khyber Cement)
2	Fly-ash	Class F-type obtained from JK Cements Khrew Kashmir
3	Coarse aggregate	20mm (Crushed) obtained from Crusher plant Ganderbal Kashmir
7	Fine Aggregate	Grading zone-1 procured from River Sindh Ganderbal Kashmir
5	Mix of Concrete	M20 (1:1.5:3) Nominal Mix
6	Water-Cement Ratio	0.38

Fly-ash procured was sieved to remove any impurities and then pulverized in a pulverizer to increase its fineness to make it suitable for replacing cement in concrete. The fly-ash was pulverized to a limit so that it freely passes 90 μm sieve. The specimens (cubes, cylinders, beams) have been tested for compressive, split cyl-

inder and flexural tests. The tests were carried after curing period of 7 and 28 days with the help of UTM (Universal Testing Machine) and Compression Testing Machine (CTM) in NIT Srinagar. Tests were conducted on the procured materials whose results are summarized in Table 3.

Table 3. Test results of procured materials.

S. No	Material	Test Name	Test results obtained from samples (Average value)	Standard values as recommended by IS codes
1	Cement	Fineness	0.547%	Should be less than 10% of wt of cement particles larger than 90 μm. (micron) as per IS:4031-1988
		Standard consistency	30%	Should be about 30% by weight of cement as per IS:4031-1988
		Setting time	Initial setting time = 2 hrs. and 55 min	Should not be less than 30 minutes as per IS:4031-1988
			Final setting time = 5 hrs. and 35 min	Should not be more than 10 hours as per IS:4031-1988
		Soundness (Average expansion value)	1.67mm	Should not be more than 10mm as per IS:4031-1988
		Compressive strength	7 day strength=32.38	Should be more than 30.1N/mm ² as per IS:4031-1988
28 day Strength=45.47 N/mm ²	Should be more than 43 N/mm ² as per IS:4031-1988			
2	Coarse aggregate	Gradation (Fineness modulus)	3.047	Should not be more than 7 as per IS:383-1970
		Aggregate crushing value	16.08%	Aggregate Crushing Value shall not exceed 30% as per IS:383-1970
3	Fine aggregates	Sieve analysis	Grading zone = Zone-1 (Well graded Sand)	Curve conforming to IS:383-1970

Similarly physical and chemical properties of the fly-ash concrete were obtained from the database of JK cements Khrew Kashmir and are summarized in Table 4.

4. Test Results of Normal Concrete

Ten specimens of cube, cylinder and beam were cast with slump of normal concrete as 25 mm and compaction factor as 0.893.

Five casted specimens of cube, cylinder and beam were tested after 7 days and 28 days. The specimens were tested for compressive strength, flexural strength and splitting tensile strength.

Compressive strength test is conducted on cubes casted which were loaded on their opposite faces in a Compression Testing Machine (CTM). The load at which first crack appears is considered as the failure load and the compressive strength is calculated corresponding to this particular value of load. The compressive strength is calculated as:

$$S_C = L_P / C_A \quad (1)$$

where S_C = compressive strength, L_P = load at failure, C_A = cross sectional area.

Flexural strength test of concrete is performed on beam specimens. The loading applied on the beam is a two point loading in which loads are applied at $(1/3)^{\text{rd}}$ points of the beam. The beam is placed in the testing machine in such a way that the load points are 13.3 cm apart from each other as well as from each support. The load is increased until the specimen fails and this load is noted as failure load. The flexural strength is calculated as:

$$S_F = 2PL / BD^2 \quad (2)$$

where S_F = flexural strength, P = load at failure/2, L = length of beam between supports, B = breadth of beam, D = depth of beam.

Splitting tensile strength test is carried out by placing a cylindrical specimen horizontally between the loading

surfaces of a compression testing machine and the load is applied until failure of the cylinder along the vertical diameter. The loading condition produces a high compressive stress below the two surfaces to which the load is applied and the larger portion corresponding to depth is subjected to a uniform tensile stress acting horizontally. It is estimated that the compressive stress is acting for about $(1/6)^{\text{th}}$ depth and the remaining $(5/6)^{\text{th}}$ depth

is subjected to tension. The splitting tensile strength is calculated as:

$$S_H = 2P/\pi HD \quad (3)$$

where S_H = horizontal tensile stress, P = load at failure, H = height of cylinder, D = diameter of cylinder

The test results obtained are the average results summarized in Table 5.

Table 4. Properties of fly-ash.

S. No	Characteristic	Results Obtained	Standard values as recommended by IS:3812-1981	
			Grade - I	Grade - II
PHYSICAL PROPERTIES				
1	Fineness (m²/kg)	361.3	Should not be less than 320	Should not be less than 250
2	Lime reactivity (MPa)	4.92	Should not be less than 4	Should not be less than 3
3	Compressive strength (MPa)	81	Not less than 80% of the strength of corresponding cement mortar cubes	
4	Drying shrinkage (%)	0.011	Should not be more than 0.15	Should not be more than 0.10
5	Autoclave expansion (%)	0.01	Should not be more than 0.8	Should not be more than 0.8
CHEMICAL PROPERTIES				
6	SiO₂+Al₂O₃+Fe₂O₃ %	91.95	Should not be less than 70	Should not be less than 70
7	SiO₂ %	58.66	Should not be less than 35	Should not be less than 35
8	Al₂O₃ %	28.76	-	-
9	Fe₂O₃ %	4.53	-	-
10	CaO %	1.58	-	-
11	MgO %	0.48	Should not be more than 5	Should not be more than 5
12	SO₃ %	0.18	Should not be more than 2.75	Should not be more than 2.75
13	Alkalis (Na₂O _{eq})	0.78	Should not be more than 1.5	Should not be more than 1.5
14	Na₂O %	0.14	-	-
15	K₂O %	0.97	-	-
16	LOI (loss on ignition) %	2.00	Should not be more than 12	Should not be more than 12

Table 5. Average value of results obtained from testing of normal concrete M20.

S. No	Test name	Specimen	Results
1	Compressive Strength	Cube	Average 7 days compressive strength = 17.23 N/mm ²
			Average 28 days compressive strength = 25.21 N/mm ²
2	Flexural Strength	Beam	Average 7 days Flexural strength = 3.87 N/mm ²
			Average 28 days Flexural strength = 5.22 N/mm ²
3	Splitting Tensile Strength	Cylinder	Average 7 days splitting tensile strength = 1.71 N/mm ²
			Average 28 days splitting tensile strength = 2.43 N/mm ²
4	Permeability	Beam	Average percentage weight of water absorbed = 0.79%
		Cylinder	Average percentage weight of water absorbed = 0.82%

5. Test Results of Fly-Ash Concrete

Fly ash was used as 5%, 10%, 15%, 20%, 25 and 30% replacement of cement by weight after pulverizing and sieving it to remove any impurities. The fly ash was thoroughly mixed with cement manually before using for making concrete for casting fly ash concrete specimens.

12 specimens were casted for each trial consisting of 4 cubes, 4 beams and 4 cylinders. 6 specimens, two from each category, were cured for 7 days and rest 6 for 28 days. In-situ tests were carried out for each trial.

The test results for 5%, 10%, 15%, 20%, 25% and 30% trails are summarized in Tables 6-11, respectively.

Table 6. Test results for 5% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.834	26mm	Average 7 days compressive strength = 23.22 N/mm ² Average 28 days compressive strength= 32.58 N/mm ²
2	Flexural Strength	Beam	0.834	26mm	Average 7 days flexural strength= 4.12 N/mm ² Average 28 days flexural strength= 8.87 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.834	26mm	Average 7 days splitting tensile strength= 1.87 N/mm ² Average 28 days splitting tensile strength = 2.76 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed =0.68% Average %age weight of water absorbed =0.72%

Table 7. Test results for 10% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.872	26mm	Average 7 days compressive strength = 23.20 N/mm ² Average 28 days compressive strength = 32.26 N/mm ²
2	Flexural Strength	Beam	0.872	26mm	Average 7 days flexural strength= 4.10 N/mm ² Average 28 days flexural strength= 8.25 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.872	26mm	Average 7 days splitting tensile strength = 1.96 N/mm ² Average 28 days splitting tensile strength = 2.86 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed =0.59% Average %age weight of water absorbed =0.51%

Table 8. Test results for 15% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.876	27mm	Average 7 days compressive strength = 21.22 N/mm ² Average 28 days compressive strength = 31.90 N/mm ²
2	Flexural Strength	Beam	0.876	27mm	Average 7 days flexural strength = 4.02 N/mm ² Average 28 days flexural strength = 8.16 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.876	27mm	Average 7 days splitting tensile strength = 1.82 N/mm ² Average 28 days splitting tensile strength = 2.69 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed =0.58% Average %age weight of water absorbed =0.57%

Table 9. Test results for 20% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.882	27mm	Average 7 days compressive strength = 17.12 N/mm ² Average 28 days compressive strength = 27.18 N/mm ²
2	Flexural Strength	Beam	0.882	27mm	Average 7 days flexural strength = 3.27 N/mm ² Average 28 days flexural strength = 7.46 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.882	27mm	Average 7 days splitting tensile strength = 1.51 N/mm ² Average 28 days splitting tensile strength = 2.39 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed =0.60% Average %age weight of water absorbed =0.61%

Table 10. Test results for 25% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.892	28mm	Average 7 days compressive strength = 15.12 N/mm ² Average 28 days compressive strength = 24.18 N/mm ²
2	Flexural Strength	Beam	0.892	28mm	Average 7 days flexural strength = 2.89 N/mm ² Average 28 days flexural strength = 6.26 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.892	28mm	Average 7 days splitting tensile strength = 1.23 N/mm ² Average 28 days splitting tensile strength = 2.12 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed = 0.62% Average %age weight of water absorbed = 0.68%

Table 11. Test results for 30% replacement trail.

S. No	Test Name	Specimen	Compacting factor	Slump(mm)	Results
1	Compressive Strength	Cube	0.898	30mm	Average 7 days compressive strength = 11.19 N/mm ² Average 28 days compressive strength = 23.28 N/mm ²
2	Flexural Strength	Beam	0.898	30mm	Average 7 days flexural strength = 2.81 N/mm ² Average 28 days flexural strength = 4.21 N/mm ²
3	Splitting Tensile Strength	Cylinder	0.898	30mm	Average 7 days splitting tensile strength = 1.20 N/mm ² Average 28 days splitting tensile strength = 2.07 N/mm ²
4	Permeability	Beam Cylinder			Average %age weight of water absorbed =0.68% Average %age weight of water absorbed =0.76%

6. Comparative Analysis

A comparative study was performed to check the effectiveness of different percentages of fly-ash and bar-charts were prepared as shown in Figs 1-6.

Further Tables 12 and 13 show the strength comparison of normal concrete and different trials of fly-ash concrete.

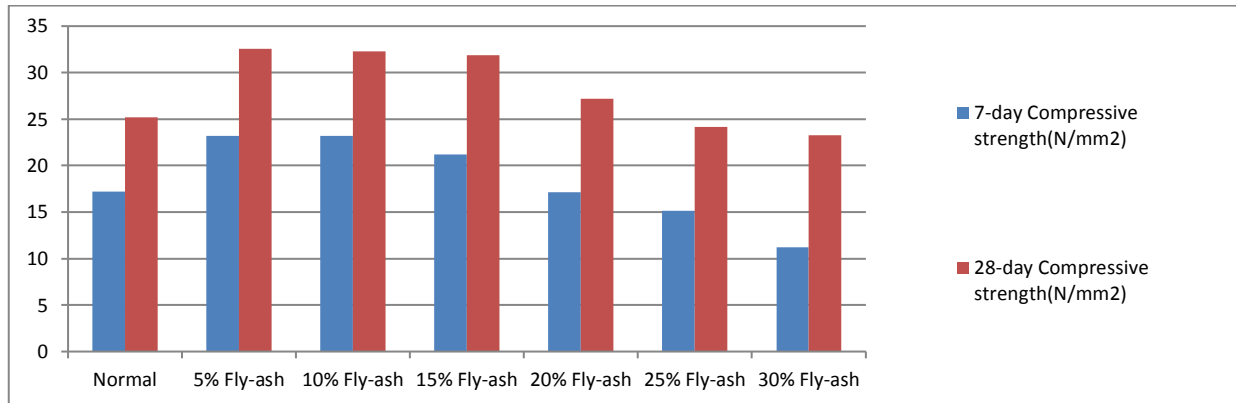


Fig. 1. Comparison of 7 days and 28 days compressive strength of normal and fly-ash concrete.

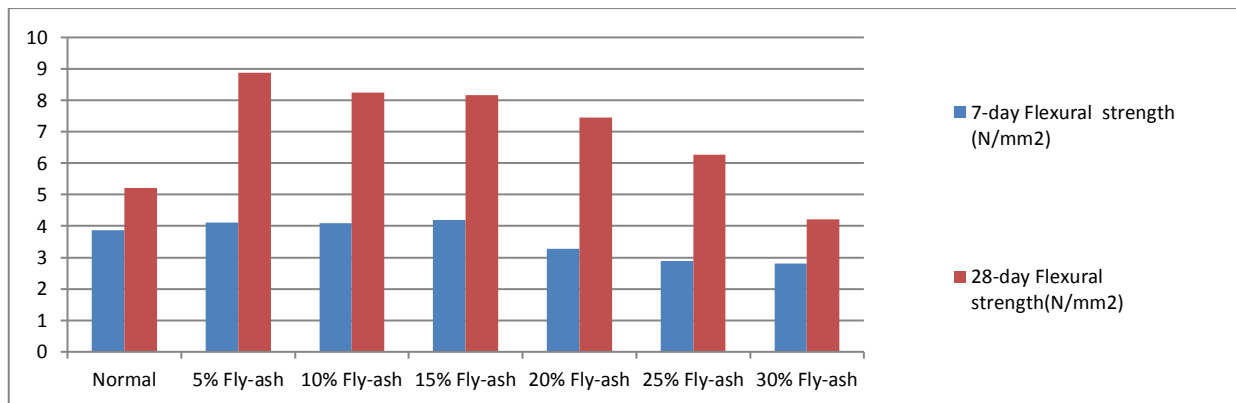


Fig. 2. Comparison of 7 days and 28 days flexural strength of normal and fly-ash concrete.

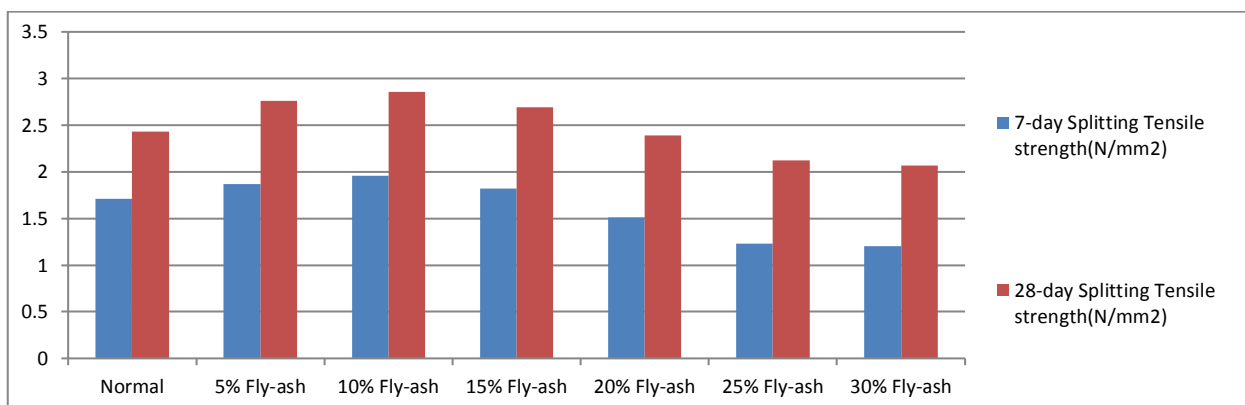


Fig. 3. Comparison of 7 days and 28 days splitting tensile strength of normal and fly-ash concrete.

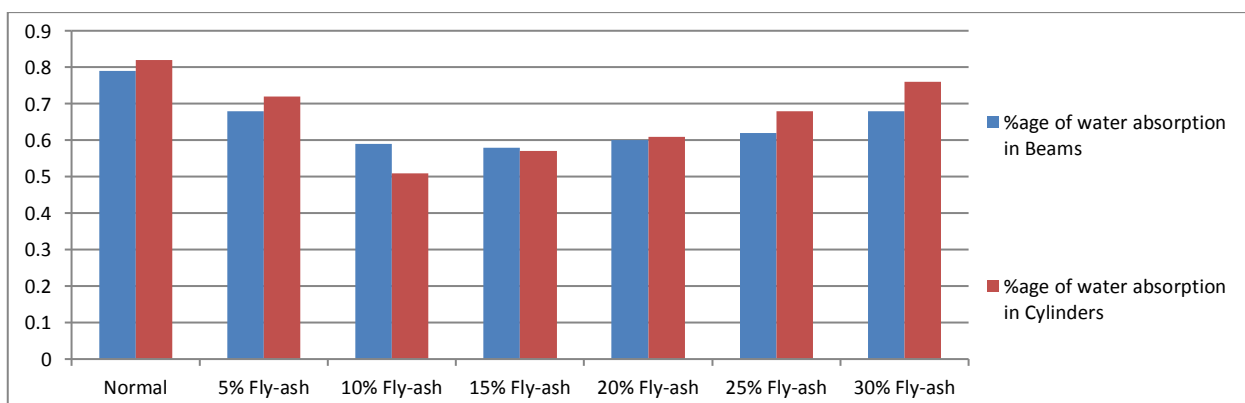


Fig. 4. Comparison of percentage of water absorption of normal and fly-ash concrete.

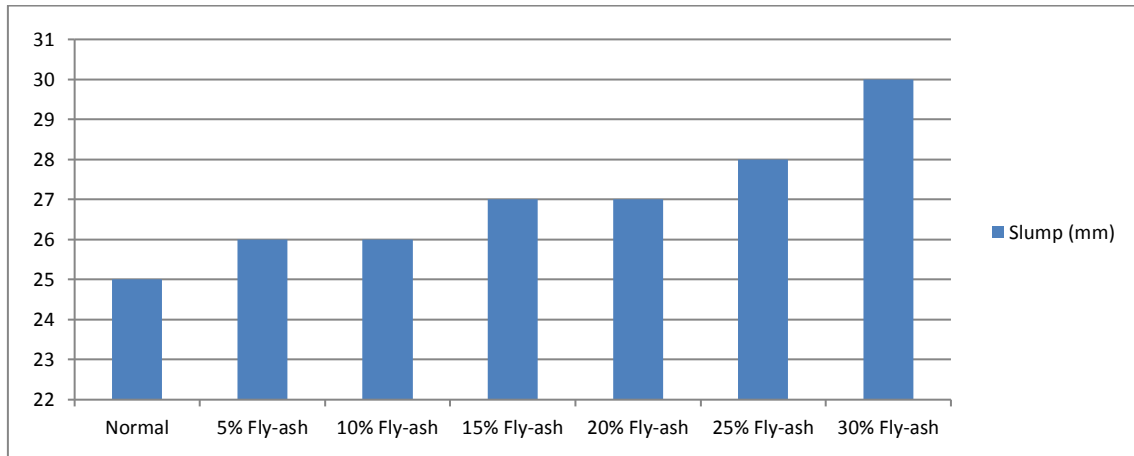


Fig. 5. Comparison of slump of normal and fly-ash concrete.

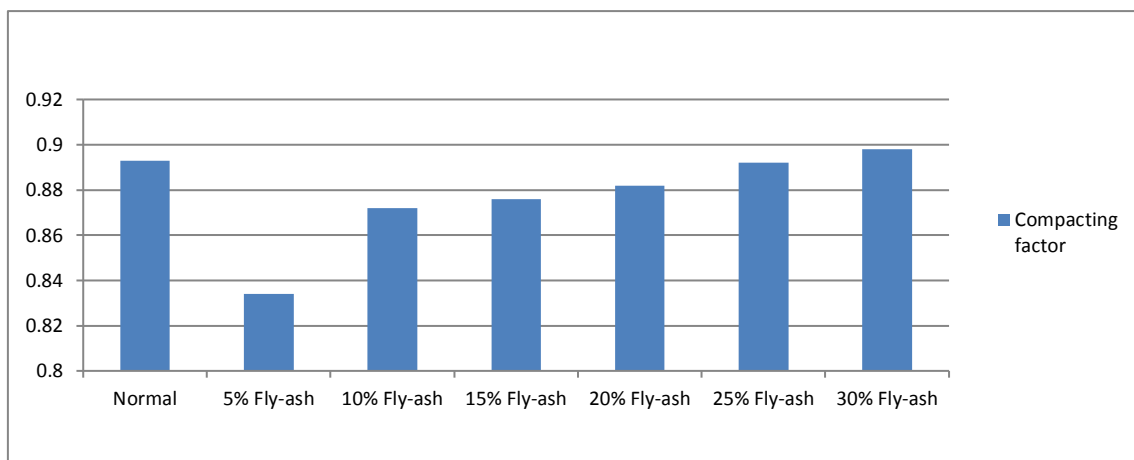


Fig. 6. Comparison of compacting factor of normal and fly-ash concrete.

Table 12. 7 days strength comparison of normal and fly-ash concretes.

Type of concrete	7 days strength (N/mm ²)					
	Compressive strength	%age increase/decrease	Flexural strength	%age increase/decrease	Splitting tensile strength	%age increase/decrease
Normal concrete	17.23	-	3.87	-	1.71	-
5% fly-ash concrete	23.22	34.76	4.12	6.45	1.87	9.35
10% fly-ash concrete	23.20	34.64	4.10	5.94	1.96	14.61
15% fly-ash concrete	21.22	23.15	4.02	3.87	1.82	6.43
20% fly-ash concrete	17.12	-0.63	3.27	-15.50	1.51	-11.69
25% fly-ash concrete	15.12	-12.24	2.89	-25.32	1.23	-28.07
30% fly-ash concrete	11.19	-35.05	2.81	-27.39	1.20	-29.82

Table 13. 28 days strength comparison of normal and fly-ash concretes.

Type of concrete	28 days strength (N/mm ²)					
	Compressive strength	%age increase/decrease	Flexural strength	%age increase/decrease	Splitting tensile strength	%age increase/decrease
Normal concrete	25.21	-	5.22	-	2.43	-
5% fly-ash concrete	32.58	29.23	8.87	69.92	2.76	13.58
10% fly-ash concrete	32.26	27.96	8.25	58.04	2.86	17.69
15% fly-ash concrete	31.90	26.53	8.16	56.32	2.76	13.58
20% fly-ash concrete	27.18	7.81	7.46	42.91	2.39	-1.64
25% fly-ash concrete	24.18	-4.08	6.26	19.92	2.12	-12.75
30% fly-ash concrete	23.28	-7.65	4.21	-19.34	2.07	-14.81

7. Economic Analysis

In this economic analysis the cost of fly-ash has been taken as zero as it has been procured free of cost from the JK cements factory Khrew Kashmir. Since the transportation cost and the cost of sand and aggregates is same for both normal concrete and fly-ash concrete, so they are not taken into account. Hence this economic analysis has been conducted on the basis of quantity of cement used in both the cases only. Fly-ash obtained was pulverized but its cost has been neglected for the present economic analysis.

Normal Concrete (1:1.5:3)

Volume of one cube = $3.375 \times 10^{-3} \text{ m}^3$

Cement used in one cube = 1.473 kg

Cost of 1 kg of cement = Rs. 5.00

Cost of cement per cube = Rs. 7.365

Cost of cement used in 1 m³ of concrete = Rs. 2182.17

Fly-Ash Concrete

The economic analysis of fly-ash concrete for different trails is summarized in Table 14.

Table 14. Economic analysis for different trails of fly-ash concrete.

S. No	Fly-ash replacement	Quantity of cement replaced per cube	Cost of cement replaced per cube (Rupees)	Cost of concrete per cube cost of cement replaced in 1 m ³ of concrete (Rupees)
1	5%	0.07365	0.368	109.03
2	10%	0.1473	0.7365	218.21
3	15%	0.22095	1.10475	327.32
4	20%	0.2946	1.473	436.43
5	25%	0.36825	1.84125	545.54
6	30%	0.4419	2.2095	654.65

8. Conclusions

After experimentation and testing of various specimens, the following conclusions were drawn:

- The short-term compressive, flexural and splitting tensile strengths (7-day) of Fly-ash concrete increased to some extent up to partial replacement of 15% and then after it considerably decreased.
- The water absorption percentage of fly-ash showed a considerable decrease giving an idea about the decrease in permeability by partial replacement of cement by fly-ash. However the percentage absorption showed an increasing trend after the trails of 25% fly-ash replacements.
- Test comparisons show that partial replacement of cement between 10% to 15 % show a maximum

efficiency in increasing the strength and other properties of concrete and hence it can be concluded that the optimum percentage of fly-ash by which cement should be replaced lies between 10% to 15%.

- The workability showed a linear increase with an increasing slump and compacting factors with increase in percentage of fly-ash.
- Economic analysis shows that Partial replacement of cement by fly-ash can boost the economy by reducing the overall cost of concrete.
- Utilization of fly-ash in concrete has provided an excellent means of disposal of fly-ash which has adverse environmental impacts.

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