Research Article

A Study on the Flexural Behaviour of Geopolymer Lightweight Eco-Friendly Concrete Using Coconut Shell as Coarse Aggregate

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This paper presents the flexural behaviour of concrete containing ground granulated blast furnace slag (GGBS) as a binder, manufactured sand (M-sand) as a fine aggregate, and coconut shell (CS) and crushed stone aggregate (CSA) as coarse aggregates. Alkaline activator sodium hydroxide with 10 molarity and sodium silicate were used in a weighing proportion of 1 : 2.5 to produce structural grade concrete. Out of 12 beams cast, 6 were used to study geopolymer coconut shell concrete (GPCSC) beam behaviour and 6 were used to study geopolymer conventional concrete (GPCC) beam behaviour. Data presented include cracking behaviour, ultimate moment capacities, deflection behaviour, ductility ratio, and end rotation of the beam. Laboratory investigations show encouraging results, and it can be summarized that coconut shell has good potential as a coarse aggregate for the production of structural grade geopolymer lightweight coconut shell concrete.

1. Introduction

Infrastructure development plays a major role in the country's economic growth, but also it leads to greenhouse effects. For infrastructure development, concrete is the most consumed building material. Conversely, it has been doing a notable impact on climate due to its large volumes and the energy consumption in the production of cement clinker, which has led to greenhouse gas emissions [1]. To reduce this impact, many initiatives were taken at academic, industrial, and government levels. The efforts mainly focus on the development, design, and execution of green materials that produce less waste, uses fewer resources, and consume less energy. The main aspect in achieving sustainable construction is to completely eliminate the use of virgin material to produce concrete.

To overcome this problem, geopolymer concrete has been introduced which can completely eliminate ordinary

Portland cement (OPC) with byproducts such as fly ash (FA), palm oil fuel ash (POFA), ground granulated blast furnace slag (GGBS), coconut shell powder (CSP), meta-kaolin (MK), and water with an alkaline activator. To produce geopolymer concrete, fine and coarse aggregate can be used as conventional concrete (CC). Many alternatives have been identified in natural and artificial aggregates to provide a partial replacement of conventional aggregates which lead to lot of research in the past decades for lightweight concrete (LWC).

Geopolymer production process can be divided into two major steps. The first one is mixing the components in proper proportion depending on the strength needed which includes mixing of binder, fine aggregate, coarse aggregate, and alkaline activators. The second step is heat curing. The heat curing process generally changes the internal structure of the geopolymer significantly. A mix containing binders such as fly ash or other slowly reacting raw material binders

requires heat treatment mainly to enhance the maturity of the geopolymers. But heat curing is not necessary for all binders. Especially for slag-rich, geopolymer composites achieve their desired properties within a few hours or days at room temperature without any heat. The overall mechanism of geopolymerisation is established by three major phases: (i) dissolution of aluminosilicate in a highly alkaline environment, (ii) coagulation and gelation of dissolved oxide minerals, and (iii) formation of the 3D network (aluminosilicate structures). The resulting chemical bond facilitates the three predominant structures found in the 3D aluminosilicate network. The predominant structures are poly sialate, poly sialate-siloxo, and poly sialate-disiloxo.

The use of geopolymer technology significantly reduces the carbon dioxide emissions resulting from the cement industry and it is a potential substitute for conventional concrete. The mineral polymers resulting from the reaction between the oxides of Si and Al in the presence of an alkali solution are referred to as geopolymer. The general formula for a geopolymer is given in the following equation:

where M is an alkali cation (such as sodium (Na^+) or potassium (K^+)), z is 1, 2, or 3, and w is the number of moles of water [2, 3]. Products such as FA, MK, and slag are the main sources of alumina silicate materials. The behaviour and properties of the geopolymer binder depend on many factors such as alkaline activator types and their compositions, as well as the chemical composition of the binder, its curing condition, and the water binder ratio used in the mix.

Many studies were done in the past two decades for partial and full replacement of natural river sand, for example, with sugarcane bagasse ash, slag limestone, quarry dust, and siliceous stone powder. It is epitomized that the quality is the major concern in using alternative material for fine aggregate which can be achieved using M-sand [4, 5].

Disposal of this abundantly available coconut shell (CS) generated from the coconut industries creates an undesirable effect on the land causing huge environmental impact. Due to its abundant availability in India, a lot of research is going on by using these CS as partial or full replacement of coarse aggregate since the unit weight of concrete produced with CS is smaller because of the unit mass of CS used in the mixture [6–15].

In the present work, a systematic study has been carried out to study the flexural behaviour of lightweight geopolymer coconut shell concrete (GPCSC) and the results were compared to the geopolymer conventional concrete (GPCC). Performance aspects, such as load-carrying capacity, crack width, ductility ratio, ultimate moments, deflections, strains, and end rotation of the beam different stages, were studied. The failure modes were also recorded for the beams. The paper compares the performance of lightweight geopolymer coconut shell concrete beams to conventional geopolymer concrete beams.

2. Experimental Programme

2.1. Constituent Materials. GGBS was used as a binder (a complete replacement for the cement) to accelerate the geopolymerisation reaction at ambient temperature. The chemical analysis of the GGBS (binder) was analysed by the x-ray photoelectron spectroscopy (XPS) using ULVAC-PHI, Inc; Model: PHI5000 Version Probe III. The chemical composition of the used GGBS is shown in Table 1, which contains a sum of CaO , SiO_2 , and MgO of around 82%, which represents more than two-thirds of the mass of the binder (Table 1). Manufactured sand (M-sand) (locally bought) was used as a fine aggregate, conforming to zone II as per IS 383: 2016 and also as per ASTM C 778 [16, 17], with a fineness modulus of 2.7 and a specific gravity of 2.6.

A coarse aggregate (CS) with a nominal size of 12.5 mm, crushed in a crusher machine with a fineness modulus of 6.1 and a specific gravity of 1.15, was used to prepare the lightweight geopolymer coconut shell concrete (GPCSC). The CS used were collected from a site near Erode, Tamil Nadu, India. Collected CS were crushed using crusher equipment. After crushing, the CS were sieved and passed through 12.5 mm sieves and used as coarse aggregates. The aggregates used in concrete can be wet, dry with a saturated surface, air-dried, or oven-dried, as in ACI 211.2-98 and ASTM C 127 [18, 19]. In the saturated surface dry (SSD) condition, the aggregate is saturated, and the surface is dry. To prepare the CS in the SSD condition, the CS was immersed in water for 24 h to prevent water absorption from occurring during the mixing process. Figure 1 shows the process of CS from raw to SSD. The chemical composition and mechanical properties of the ingredients used are shown in Table 1. For the control mix, CSA coarse aggregate of 12.5 mm nominal size with a fineness modulus of 7.1 and a specific gravity of 2.86 was used to prepare GPCC.

2.2. Alkaline Activator. A sodium silicate and sodium hydroxide solution were chosen as the alkaline activator to activate the source material. A sodium silicate solution with $\text{Na}_2\text{O} = 14.7\%$, $\text{SiO}_2 = 29.4\%$ and water = 55.9% by mass was used for the study. The sodium hydroxide pellets with 97%–98% purity were mixed with water to make a solution at 10 molarity (10 M) concentration. The sodium hydroxide solution was prepared in the laboratory 24 h before casting. The sodium hydroxide and sodium silicate solution were prepared in a weight proportion of 1 : 2.5 just before casting.

2.3. Mix Design. A minimum compressive strength of 30 MPa at 28 days was fixed as a target strength expected from the concrete mixes used so that the developed GPCC and GPCSC could be used for both non-structural and structural elements without any hesitation in practical implementation. The ingredients were chosen based on the trial mix since the ACI and Indian Standard methods could

TABLE 1: Chemical composition and mechanical properties of the materials used.

<i>Chemical composition of the GGBFS from the XPS study</i>													
CaO	SiO ₂	Al ₂ O ₃	MgO	Na ₂ O	SO ₃	P ₂ O ₅	K ₂ O	TiO ₂	MnO	Fe ₂ O ₃	SrO	Cl	LOI
45.83	32.52	13.7	3.27	0.25	1.8	0.04	0.48	0.73	0.35	0.76	0.08	0.02	0.60
<i>Mechanical properties of materials used</i>													
Properties	GGBS		M-sand		CS		CSA						
Bulk density-compacted (kg/m ³)	—		1520		650		1670						
Bulk density-loose (kg/m ³)	—		1500		550		1490						
Maximum size	12.3 μ m		4.75 mm		12.5 mm		12.5 mm						
Specific gravity	2.65		2.60		1.15		2.82						
Fineness modulus	—		2.70		6.11		6.93						
Specific surface area (cm ² /g)	4290		—		—		—						
Water adsorption (%)	—		2.25		22.00		0.50						
Shell thickness (mm)	—		—		8–10		—						
Aggregate impact value (AIV) (%)	—		—		8.15		6.23

Discarded CS at CS yard

Crushing equipment

Crushed CS

(a)

(b)

FIGURE 1: Continued.

FIGURE 1: Processing of CS (SSD condition). (a) Preparation of raw coconut shell aggregate. (b) Raw crushed CS 12.5 mm. (c) CS after immersion in water for 24 h. (d) CS in SSD condition.

not be applied to the mix design for the concrete with agro-waste materials due to its structural behaviour. Thus, all the trial mixes were prepared based on the volume proportions and then converted into the weight proportion. To prepare the control mix, the volume of the coconut shell coarse aggregate was compared to that of the crushed granite gravel and kept constant. The mix proportions (GGBS: M-sand: CS/CSA) for GPCSC and GPCC and the amount of the alkaline activator solution with water content are shown in Table 2.

2.4. Details of the Beam Specimen. The flexural behaviour of reinforced GPCSC and GPCC was studied in two different cases: one is singly reinforced (steel reinforcements provided in the tension zone of the beams only) (Figure 2) and the second one is doubly reinforced beams (steel reinforcements provided in both tension and compression zone of the beams) as shown in Figure 3. The section was determined as under-reinforced/over-reinforced by using the limit state design method as per IS 456: 2000 [20]. Totally twelve beams were tested, of which six beams were GPCSC and six beams

TABLE 2: Mix proportion of GPCSC and GPCC.

Mix types	Mix proportions	Binder	Fine aggregate	Coarse aggregate		Alkaline activator	
		GGBS (kg/m ³)	M-sand (kg/m ³)	CS (kg/m ³)	CSA (kg/m ³)	NaOH solution (ml/m ³)	Na ₂ SiO ₃ (ml/m ³)
GPCSC	1 : 2.53 : 1.05	420	1064	443	—	46	228
GPCC	1 : 4.15 : 3.56	282	1172	—	1004	31	152

ml: millilitre.

FIGURE 2: Reinforcement details and strain gauge location for SRB (S1, S2, and S3).

were GPCC. In addition to the beam specimen, the required number of cubes, cylinders, and prisms was cast and tested for its material characterizations. Cube specimen $100 \times 100 \times 100$ mm in size was used and tested as per IS 516: 1959 [21] for compressive strength of concrete at the rate of $1.4 \text{ kN/cm}^2/\text{min}$. In the design of concrete structural elements, concrete tensile strength also plays a vital role. However, it is somewhat complex in nature to determine the tensile strength of concrete under uniaxial tension test, and hence splitting tensile strength tests are generally conducted on cylindrical specimens though it is an indirect method. Therefore, splitting tensile strength was carried out to determine the indirect tensile strength of concrete study in this research. As per ASTM C496-90 [22], a 100 mm diameter and 200 mm long cylindrical specimen was used for the splitting tensile strength test which gives tensile strength of concrete indirectly. The rate of loading given for this test is 0.7 kN/min . The splitting tensile strength f_{st} of specimens was calculated using the following equation:

$$f_{st} = \frac{2P}{\pi DL} \text{ (MPa)}, \quad (2)$$

where P is the maximum applied load in Newton, D is the diameter of the cylinder in mm, and L is the length of the cylinder in mm.

A two-point load method was adopted to measure the flexural strength of the specimen as per ASTM C78-84 [23]. Prisms $100 \times 100 \times 500$ mm in size were used to find the

flexural strength and tested as per IS 516:1959 [21] at the rate of 180 kg/min . The flexural strength f_t of concrete specimens was calculated using the following equation:

$$f_t = \frac{PL}{BD^2} \text{ (MPa)}, \quad (3)$$

where P is the maximum applied load in Newton, L is the supported length of the prism in mm, B is the breadth of the prism in mm, and D is the depth of the prism in mm.

As per ASTM C469-02 [24], cylindrical specimens 150 mm in diameter and 300 mm in height were used for the determination of elastic modulus. This method involves applying compressive load longitudinally to the specimen to calculate the elastic modulus. For this purpose, deformation was measured using an extensometer attached to the cylindrical specimen for every increments of the load. For every increment of applied load, stresses and strains were calculated and the stress-strain graph was drawn. From the stress-strain graph, the mean modulus of elasticity was calculated [7, 11] using the following equation:

$$E = \frac{(S2 - S1)}{(\epsilon_2 - 0.000050)} \text{ (MPa)}, \quad (4)$$

where E is chord modulus of elasticity in MPa, $S2$ is stress corresponding to 40% of the ultimate load in MPa, $S1$ is stress corresponding to a longitudinal strain of 50×10^{-6} in MPa, and ϵ_2 is longitudinal strain produced by $S2$.

Doubly reinforced beam

All dimensions are in mm
Nos. - numbers

FIGURE 3: Reinforcement details and strain gauge location for DRB (D1, D2, and D3).

The determined properties of the concrete tested are given in Table 3. A schematic diagram of beam testing is shown in Figure 4. Singly reinforced beam (SRB) was represented as S1, S2, and S3 (Figure 3) and doubly reinforced beam (DRB) was represented as D1, D2, and D3 (Figure 4). The beam details with their corresponding ID and the percentage of used reinforcement and also total tension and compression area of reinforcements are given in Table 4.

All 12 specimens were cast with the overall dimension of length (3000 mm), 150 mm (width), and overall depth (230 mm). The span to effective depth ratio was chosen as 15 to ensure the failure of the beam with flexural nature as per IS 456:2000 [20]. No shear reinforcement was provided in the pure bending region for a length of 934 mm as shown in Figures 3 and 4. Also, sufficient shear links were provided at a spacing of 150 mm in the remaining distance. Fe500 grade of steel reinforcements (having Young's modulus $2.1 \times 10^5 \text{ N/mm}^2$ and yield strength 518 N/mm^2) conforming with the IS 1786: 2008 [25] was used in all the beams both in main reinforcement and in stirrups as well.

Strain gauges of length 10 mm were fixed at midspan in all the bars on both the compression zone and the tension zone to measure the strain in the provided reinforcement bar. The gauges on the surface of the bars were ground

smooth with sandpaper of approximate length 20 mm to facilitate the fixing of the 10 mm strain gauge, with a gauge resistance of 120 ohms, and the wire that was extended was covered with a fiberglass sleeve material. The strain gauges were fixed with gloves applied in the reinforcing bar and the surface of the strain gauges covered with waterproofing tape to prevent water entry and damage during casting. The functioning of the fixed strain gauge in reinforcement must be cross-checked by measuring the output resistance of 120 ohms using a multimeter to avoid damage while fixing and protecting. Figure 5 presents the procedure to fix the strain gauge in the reinforcement and the technique to check its functioning of strain gauge resistance using a multimeter.

2.5. Casting and Curing of the Beam Specimens. For casting all the specimens, the ingredients were weighed and mixed in a concrete mixture as per the trial mix proportions mentioned in Table 2. Wooden mould was used for casting the beam specimens. The bottom and sides of the moulds were bolted properly to avoid air gap and also for easy demoulding. The designed reinforcement is placed inside the mould leaving sufficient cover at the sides and at the bottom using cover blocks (cement masonry cover block). The ingredients were dry-mixed for 3-4 min and alkaline activators

TABLE 3: Properties of geopolymers concrete.

Mechanical properties	GPCSC			GPCC		
	Results	No. of samples	Standard deviation	Results	No. of samples	Standard deviation
28-day density (kg/m^3)	1980		2.52	2440		9.07
Compressive strength (MPa)	32.6		0.32	35.2		0.21
Flexural strength (MPa)	4.1	3	0.21	3.6	3	0.21
Split tensile strength (MPa)	3.0		0.15	3.1		0.15
Modulus of elasticity (GPa)	7.3		0.10	12.9		0.35

FIGURE 4: Schematic diagram of beam testing.

of designed proportion were mixed gradually and the wet mix was allowed in the mixture machine for 5 min. The wet mix was placed in the mould in three layers and compaction was done using a needle vibrator. Proper care was taken for uniform compaction and surface finish throughout the beam. Immediately after casting, the specimens were covered with a polyethene sheet and they were demoulded after 24 h for concealed curing as per ASTM C171-16 [26] to prevent evaporation of water.

2.6. Specimen Preparation and Instrumentations. After 28 days of concealed curing, the beams were painted with white powder to observe cracks at the time of the test. A day before the specimens attained their age of 28 days, the beams were carefully removed from the polythene cover and painted with whitewash powder. The beam was placed on the loading frame and supported at 100 mm from each beam end that

produced a beam clear span of 2800 mm (centre to centre distance from the support). Two symmetrical point loads were acting at one-third of the beam clear span as shown in Figure 6. The source of the load was applied through a hydraulic jack connected with a load cell of 1000 kN maximum capacity with the tolerance limit $\pm 2\%$. A dial gauge with a least count of 0.001 mm was used to measure the deflection of the beam under the loads. Crack widths were measured with a crack microscope that reads to an accuracy of 0.02 mm on the surface of the beams. The proving ring was set to zero before the readings were taken. Prior to the load application on the setup, all the equipment was properly monitored to ensure that they were in their correct position. The beam specimens were then loaded gradually, with an incremental stage. At the time of the test, the loads were applied incrementally at a rate of 1 kN until failure. The tensile and compressive strains of both reinforcements and concrete were measured through electrical resistance gauges.

TABLE 4: Beam details (width, 150 mm; effective depth, 200 mm; and overall length, 3000 mm).

Beam ID	Nominal reinforcement	Compression reinforcement	Tension reinforcement	Shear reinforcement	Area of tensile reinforcement A_{st} (mm ²)	$P = A_{st}/bd$ (%)
GPCSC-S1	2 nos. of 8 mm ϕ	—	2 nos. of 10 mm ϕ	6 mm ϕ @ 150 mm c/c	157.079	0.52
GPCSC-S2	2 nos. of 8 mm ϕ	—	2 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	226.195	0.75
GPCSC-S3	2 nos. of 8 mm ϕ	—	3 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	339.292	1.13
GPCC-S1	2 nos. of 8 mm ϕ	—	2 nos. of 10 mm ϕ	6 mm ϕ @ 150 mm c/c	157.079	0.52
GPCC-S2	2 nos. of 8 mm ϕ	—	2 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	226.195	0.75
GPCC-S3	2 nos. of 8 mm ϕ	—	3 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	339.292	1.13
GPCSC-D1	—	2 nos. of 10 mm ϕ	3 nos. of 16 mm ϕ	6 mm ϕ @ 150 mm c/c	603.185	2.01
GPCSC-D2	—	2 nos. of 16 mm ϕ & 1 no of 12 mm ϕ	3 nos. of 20 mm ϕ	6 mm ϕ @ 150 mm c/c	942.478	3.14
GPCSC-D3	—	2 nos. of 20 mm ϕ & 1 nos. of 10 mm ϕ	3 nos. of 20 mm ϕ & 2 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	1168.672	3.90
GPCC-D1	—	2 nos. of 10 mm ϕ	3 nos. of 16 mm ϕ	6 mm ϕ @ 150 mm c/c	603.185	2.01
GPCC-D2	—	2 nos. of 16 mm ϕ & 1 no of 12 mm ϕ	3 nos. of 20 mm ϕ	6 mm ϕ @ 150 mm c/c	942.478	3.14
GPCC-D3	—	2 nos. of 20 mm ϕ & 1 no of 10 mm ϕ	3 nos. of 20 mm ϕ & 2 nos. of 12 mm ϕ	6 mm ϕ @ 150 mm c/c	1168.672	3.90

FIGURE 5: Procedure to fix strain gauge.

FIGURE 6: Test setup in the loading frame for the beam.

All the strains were recorded using a 32 channel CALPlex data logger. A theodolite and a levelling staff were used to measure the end rotation on the incremental.

3. Results and Discussion

3.1. General Observations of the Tested Beams. All beams (GPCSC and GPCC) showed typical behaviour in flexure. Since the concave and convex surfaces of the coconut shell aggregate are smooth, bond failure may occur during testing in GPCSC concrete [10]. However, no horizontal cracks were observed at the level of the reinforcement, which indicated that there were no occurrences of bond failure. Vertical flexural cracks were observed in the constant moment region and sufficient warning was observed with a significant amount of ultimate deflection. The end rotation of the GPCSC beam was higher when compared to the GPCC beam. Similar findings were reported in oil palm shell concrete [27, 28].

3.2. Crack Pattern and Failure Mode. For all the tested beams, a few fine vertical flexural cracks first developed within the pure bending-moment zone, when the bending-moment exceeded the cracking moment of the beams. Once the cracks are formed at mid-span, their propagation is more parallel to each other and normal to the axis of the beam. The crack patterns at the failure of GPCSC and GPCC beams are shown in Figures 7–10. The crack patterns and modes of failure were dissimilar due to the value of reinforcement ratio and the diameter of reinforcement used in the tension and compression zone tests, for both GPCSC and GPCC. In general, this crack spacing mechanism is described as a

function of a bond between the reinforcement and the concrete [10, 27, 28].

Figures 7 and 8 depict the crack pattern up to peak load for all the tested singly reinforced beams (GPCSC and GPCC). The number of cracks developed along the span of GPCC was smaller than that of GPCSC, especially in the pure bending zone. The possible reason may be that of higher compressive strength and modulus of elasticity of GPCC compared to GPCSC as summarized in Table 3. But the cracks were wider and mostly concentrated at mid-span in the pure bending zone, at the time of yielding steel reinforcements in this region for both GPCSC and GPCC. Once the cracks are formed in the pure bending zone, the formed cracks progressed both lengthwise and widthwise since in that pure bending zone shear reinforcements are not provided intentionally to study the behaviour of beam under flexure. Also, it was found that the vertical cracks formed initially in both the end of the shear spans are further inclined at higher loads. This was observed in both GPCC and GPCSC beams. Therefore, it can be stated that the behaviour of GPCSC is very similar to GPCC beams. At the final loading stage, very few inclined cracks reached the compression zone of the geopolymer concrete. Based on Figures 7 and 8, the cracks were almost uniformly distributed along the beam span with an average spacing of 98 mm for GPCSC and 156 mm for GPCC, respectively.

Figures 9 and 10 depict the crack pattern at peak load for all the tested doubly reinforced beams (GPCSC and GPCC). The number and width of cracks developed along the span for DRB were higher when compared to SRB for both GPCSC and GPCC. The vertical cracks in the shear span were inclined due to the shear stress similar to SRB. At the final loading stage, many inclined cracks reached the

GPCSC_SRB_S1/S2/S3

All dimensions are in mm

All loads are in kN

FIGURE 7: The crack pattern of GPCSC for singly reinforced beam (SRB).

GPCC_SRB_S1/S2/S3

All dimensions are in mm

All loads are in kN

FIGURE 8: Crack pattern of GPCC for singly reinforced beam (SRB).

FIGURE 9: Crack pattern of GPCSC for doubly reinforced beam (DRB).

FIGURE 10: The crack pattern of GPCC for doubly reinforced beam (DRB).

compression zone of the geopolymer concrete in DRB when compared with SRB. Based on Figures 9 and 10, the cracks were almost uniformly distributed along the beam span with the spacing of approximately 75 mm for both GPCSC and GPCC.

Overall, this observation of crack patterns demonstrated that a composite action existed between the reinforcing bars and the geopolymer concrete. This study reveals that proper bonding exists between the reinforcements and both GPCC and GPCSC concretes and hence integration among them are also a perfect indication from the crack patterns observed. The number of cracks formed in the DRB was more compared to SRB for both GPCC and GPCSC beams. The reason for more cracks in the DRB is due to higher moment of resistance due to the higher reinforcement ratio compared to SRB. However, it was found that the maximum crack width of crack in the case of DRB (3.02 mm, 2.45 mm, and 1.98 mm) is less compared to SRB (3.59 mm, 3.14 mm, and 2.98 mm) and the reason behind this is the higher modulus of elasticity of the GPCC than the GPCSC as shown in Table 3. The total number of cracks that developed in GPCSC and GPCC is given in Table 5. It was observed that the width of the cracks was considerably reduced for DRB when compared with SRB for both GPCSC and GPCC. Similar findings were reported in oil palm shell concrete [27, 28].

3.3. Influence of the Reinforcement Ratio. The flexural stiffness of the beams of GPCSC and GPCC increases as the reinforcement ratio increases. Thus, it can be deduced from the experimental results that the serviceability (within permissible limits of deflection and crack width for the size of beam used) performance of a geopolymer concrete beam can be enhanced by increasing the amount of longitudinal reinforcement. This improvement can be clearly understood by assuming the bars as parallel springs. As the number of bars increased, the overall stiffness also increased, thereby lowering the deflection after cracking, and limiting the crack width (Table 5) [29, 30]. The flexural stiffness increased with increasing longitudinal rigidity and suggested that the reinforcement ratio should be increased in order to effectively control crack widths [30]. The test results clearly show that the deflection of the geopolymer beams (both GPCSC, Figures 11 and 12, and GPCC, Figures 13 and 14) decreased for the same load level with an increase in the reinforcement ratio. The theoretical design moment of the beams was calculated using the stress block analysis as recommended by IS 456:2000.

It was found that the ultimate moment obtained from the experiments was approximately 5%–15% higher for singly reinforced beams when compared to the theoretical moments. It is mainly due to the lower reinforcement ratio (0.52%–1.13%) (Table 6, singly reinforced beam) and geopolymer concrete impact. But for beams with a high reinforcement ratio (2.01%–3.89%) (Table 6, doubly reinforced beam), the experimental ultimate moment was only 1%–7% higher for both doubly reinforced geopolymer beams. The

capacity ratio is taken as the ratio of an experimental ultimate moment to the theoretical ultimate moment. The capacity ratio varies between 1.05 and 1.15 for SRB and 0.98–1.07 for DRB indicates the virtuous resisting capacity of the section of the beam tested. Moments calculated using the standard IS 456:2000 can be conservative and can be used to predict the moment capacity for both GPCC and GPCSC beams. Also, the results obtained in this study are compared with the findings of other researchers on lightweight concrete beams and presented in Table 7. It can be observed that the current research is consistent with the findings of other similar research [29–33].

3.4. Mid-Span Deflection Behaviour. Generally, all beams exhibited a similar load-deflection pattern. Tables 8 and 9 show the results of deflection of beams at service stage, ultimate stage, and permissible deflection as per IS456:2000 [20] and also as per ASTM C 778 [23]. Typical experimental moment deflection curves for the singly reinforced GPCSC and GPCC beams are shown in Figures 11 and 12, respectively. The moment deflection curves for doubly reinforced beams of GPCSC and GPCC are shown in Figures 13 and 14, respectively.

The slope of the moment-deflection curve was steep and mostly linear before cracking occurred in all geopolymer beams. Here, the stiffness of the member is initially provided by the total concrete and steel area. A change in slope of the moment-deflection curve was observed once flexural cracks formed due to the reduction in stiffness in both GPCSC and GPCC. The formation of flexural cracks in GPCSC was earlier due to the porous nature of the coconut shell and it also has low density and modulus of elasticity which directly influences the stiffness of the aggregate. At the latter stage, the moment-deflection curve indicated large increases in deflection with a small increase in load. It can be observed that GPCSC beams exhibit a behaviour similar to that of GPCC beams.

Although CSC has a low modulus of elasticity, the deflection under the design service loads for the singly reinforced beams was acceptable as the span-deflection ratios ranged from 96.59 mm to 235.89 mm and were larger than the allowable limit provided by IS456:2000 [20]. In doubly reinforced beams, the span-deflection ratio ranged from 52.37 mm to 111.02 mm and this was larger than the allowable limit provided by IS 456:2000 [20]. It must be noted that the current experiments only involve short-term deflection and do not allow for evaluating shrinkage and associated creep.

Also, it can be noted that there is no code and standards are available for LWC especially for CSC and in combination with geopolymer; IS 456:2000 reference is taken only for guidance in this deflection parameter since this IS 456:2000 is established for concrete produced with traditional materials. Therefore, though permissible deflection as per IS 456:2000 is exceeding for both GPCC and GPCSC, it could be studied further to control the deflection within

TABLE 5: Crack width, reinforcement ratio, and number of cracks.

Beam ID	Neutral axis depth (x_u) (mm)	Reinforcement ratio (%P) $P = (A_{st}/bd)$ in (%)	Crack width (mm)	Number of cracks
<i>Singly reinforced beam (SRB)</i>				
GPCSC-S1	48.60	0.52	3.59	25
GPCSC-S2	70.63	0.75	3.14	22
GPCSC-S3	103.11	1.13	2.98	21
GPCC-S1	44.00	0.52	2.81	22
GPCC-S2	63.71	0.75	2.73	19
GPCC-S3	94.46	1.13	2.61	18
<i>Doubly reinforced beam (DRB)</i>				
GPCSC-D1	119.15	2.01	3.02	53
GPCSC-D2	120.75	3.14	2.45	49
GPCSC-D3	126.73	3.89	1.98	48
GPCC-D1	114.93	2.01	2.64	50
GPCC-D2	115.18	3.14	2.42	46
GPCC-D3	118.54	3.89	1.45	38

A_{st} : area of reinforcement; b : breadth of beam; d : effective depth of a beam.

FIGURE 11: Load-deflection behaviour of GPCSC (SRB).

FIGURE 13: Load-deflection behaviour of GPCC (SRB).

FIGURE 12: Load-deflection behaviour of GPCSC (DRB).

FIGURE 14: Load-deflection behaviour of GPCC (DRB).

TABLE 6: Comparison of test results at service stage and ultimate stage.

Beam ID	Service load (kN)	Ultimate load (kN)	1st crack load (kN)	Cracking moment (mcr-exp) (kN·m)	Service moment (M_s -exp) (kN·m)	Ultimate moment (M_u -exp) (kN·m)	Ultimate moment (M_u -theo) (kN·m)	Capacity ratio
<i>Singly reinforced beam</i>								
GPCSC-S1	18.67	28	9	4.20	8.71	13.07	12.24	1.07
GPCSC-S2	25.33	38	10	4.67	11.82	17.73	16.88	1.05
GPCSC-S3	36.00	54	11	5.13	16.80	25.20	22.69	1.11
GPCC-S1	20.00	30	10	4.66	9.33	14.00	12.41	1.13
GPCC-S2	26.67	40	12	5.60	12.45	18.67	17.17	1.09
GPCC-S3	38.67	58	14	6.53	18.05	27.07	23.58	1.15
<i>Doubly reinforced beam</i>								
GPCSC-D1	54.67	82	12	5.60	25.51	38.27	35.69	1.07
GPCSC-D2	88.00	132	13	6.07	41.07	61.60	59.94	1.03
GPCSC-D3	102.67	154	16	7.47	47.91	71.87	73.71	0.98
GPCC-D1	56.00	84	14	6.53	26.13	39.20	37.65	1.04
GPCC-D2	90.67	136	15	7.00	42.31	63.47	61.59	1.03
GPCC-D3	108.00	162	18	8.40	50.40	75.60	75.07	1.01

TABLE 7: Comparison of the capacity ratio of LWC with others.

Coarse aggregate	Code	Mean	Standard deviation	Author
Coconut shell (full replacement)	IS 456 [20]	1.26	0.048	Current study
Coconut shell (partial replacement)	IS 456 [20]	1.35	0.049	Parkash et al. [14]
Coconut shell (full replacement)	IS 456 [20]	1.09	0.000	Parkash et al. [14]
Coconut shell	IS 456 [20]	1.19	0.148	Gunasekaran et al. [10]
Oil palm shell	BS8110 [34]	1.16	0.115	Teo et al. [35, 36]

TABLE 8: Deflection at service stage and ultimate stage.

Beam ID	Service stage		Ultimate stage		Ductility index (Δ_U)/(Δ_Y)
	Load (kN)	Displacement (Δy) (mm)	Load (kN)	Displacement (Δu) (mm)	
<i>Singly reinforcement beam</i>					
GPCSC-S1	18.67	7.56	28	16.82	2.22
GPCSC-S2	25.33	9.02	38	18.99	2.11
GPCSC-S3	36.00	10.67	54	28.99	2.72
GPCC-S1	20.00	3.21	30	11.23	3.50
GPCC-S2	26.67	3.45	40	12.34	3.58
GPCC-S3	38.67	7.89	58	25.99	3.29
<i>Doubly reinforcement beam</i>					
GPCSC-D1	54.67	10.45	82	29.45	2.82
GPCSC-D2	88.00	15.67	132	43.44	2.77
GPCSC-D3	102.67	18.56	154	53.45	2.88
GPCC-D1	56.00	7.11	84	25.22	3.55
GPCC-D2	90.67	9.99	136	35.45	3.55
GPCC-D3	108.00	15.66	162	45.45	2.90

TABLE 9: Comparison of deflection with the Indian standard code.

Beam ID	Experimental deflection (Δ_{exp})	Permissible deflection (Δ_{per}) as per IS465-2000	$(\Delta_{exp})/(\Delta_{per})$	Span (Δ_{exp})
<i>Singly reinforcement beam</i>				
GPCSC-S1	16.82	11.67	1.44	166.47
GPCSC-S2	18.99	11.67	1.63	147.45
GPCSC-S3	28.99	11.67	2.48	96.59
GPCC-S1	11.87	11.67	0.96	235.89
GPCC-S2	12.34	11.67	1.06	226.90
GPCC-S3	25.99	11.67	2.23	107.73
<i>Doubly reinforcement beam</i>				
GPCSC-D1	29.45	11.67	2.52	95.08
GPCSC-D2	43.44	11.67	3.72	64.46
GPCSC-D3	53.45	11.67	4.58	52.39
GPCC-D1	25.22	11.67	2.16	111.02
GPCC-D2	35.45	11.67	3.04	78.98
GPCC-D3	45.45	11.67	3.89	61.61

permissible limits of some other standards or else study should be taken in the future to establish the control deflection for both GPCC and GPCSC beams under flexure.

3.5. Ductility Behaviour. The ratio of total deflection at ultimate load to the deflection at service load is known as the ductility index. The ductility indices of all the beams tested are presented in Table 8. Since coconut shell aggregate possesses good toughness and shock absorbance [10], it contributes to improving the ductility index in all the beams. In this study, the ductility indices vary from 2.11 to 2.88 for GPCSC and from 2.90 to 3.59 for GPCC. The lowest value is recorded in beam GPCSC-S2 and the highest value is recorded in beam GPCC-S2. Similar results have also been reported in previous research [33, 34].

3.6. Strain Measurement. The main objective of measuring the strain on the longitudinal steel bar is to find whether the beam reaches the yielding point during the test. The steel strain was measured with strain gauges attached to the tension steel bar. The concrete compressive strain was measured with strain gauges attached in the compression zone (Figures 2 and 3).

3.6.1. Steel Bar in Tensile and Concrete in the Compression Zone (SRB). The actual moment-strain behaviour of singly reinforced beam for GPCSC and GPCC is given in Figures 15 and 16. It can be seen that the load-strain curvature pattern is similar in both GPCC and GPCSC beams and it also happens in both bottom tensile reinforcement in the tension zone and geopolymer concrete in the compression zone. The strain in the tensile reinforcement at the ultimate stage was SRB S1-3987 μm , SRB S2-4447 μm , and SRB S3-4823 μm for GPCSC. Also, the strain in the geopolymer concrete in the compression zone at the ultimate stage was SRB S1-2565 μm , SRB S2-3656 μm , and SRB S3-4565 μm for GPCSC. Similarly, the strain in the tensile reinforcement at the ultimate stage was SRB S1-2343 μm , SRB S2-2675 μm , and SRB S3-4213 μm for GPCC. Also, the strain in the geopolymer concrete in the compression zone at the

FIGURE 15: Load-strain behaviour of GPCSC (SRB).

ultimate stage was SRB S1-1897 μm , SRB S2-2008 μm , and SRB S3-3012 μm for GPCC.

The tested beam specimens of SRB shared almost the same small increase of linear strain segment in both GPCSC and GPCC beams until initial crack load. A linear and nonlinear increase in strain was observed beyond the initial crack load. The failure mode of GPCSC beams is akin to GPCC beams and also GPCSC beams achieved their

FIGURE 16: Load-strain behaviour of GPCC (SRB).

maximum strain like GPCC. Therefore, it can be stated that the GPCSC beam behaviour is concurrent with the GPCC beams. For GPCSC-S1 and GPCSC-S2, the strain in tension-steel bars is less than the strains in GPCC which indicates a tension failure mode because in both the cases the same amount of steel was provided but GPCSC steels strains are less due to less CSC strength compatibility compared to CC strength and hence tension failure happens. For GPCSC-S3 beam, the strain in concrete is almost equal to the ultimate strain in steel, which indicates a balanced failure mode; i.e., in this case, concrete and steel strains almost reach simultaneously. But, for GPCC-S1, S2, and S3, the strain in tension-steel bars is less than the strains in geopolymer concrete, indicating a tension failure mode. Overall, the effect of the process of crack formation can be noticed on the moment-strain curves in both singly reinforced geopolymer beams.

3.6.2. Steel Bar in Tensile and Concrete in the Compression Zone (DRB). The actual moment-strain behaviour of a doubly reinforced beam for GPCSC and GPCC is given in Figures 17 and 18. Load-strain curvature observed among the bottom tensile reinforcement in tension zone, geopolymer concrete, and compression reinforcement in

FIGURE 17: Load-strain behaviour of GPCSC (DRB).

compression zone is analogous in both GPCSC and GPCC. The strain in the tensile loading at the ultimate stage was DRB D1-4323 μm , DRB D2-6678 μm , and DRB D3-7898 μm for GPCSC. Similarly, the strain in the tensile reinforcement at the ultimate stage was: DRB D1-3676 μm , DRB D2-5004 μm , and DRB D3-5976 μm for GPCC. The strain in the compression reinforcement and compression zone concrete for GPCSC at the ultimate stage was: DRB D1-498 μm and 777 μm , DRB D2-1545 μm and 1100 μm , and DRB D3-2457 μm and 1586 μm , respectively. The strain in the compression reinforcement and compression zone concrete for GPCC at the ultimate stage was DRB D1-400 μm and 678 μm , DRB D2-856 μm and 1009 μm , and DRB D3-1408 μm and 1678 μm , respectively. All the geopolymer beams were subjected to ductile failure with an increased strain percentage in GPCC when compared to GPCSC. The tested beam specimens of a doubly reinforced beam shared almost the same small linear increase in strain segment in both GPCSC and GPCC beam until initial crack load. A rapid increase of linear and nonlinear strain was observed beyond the initial crack load. All doubly reinforced geopolymer beams (GPCSC-D1, D2, and D3 and GPCC-D1, D2, and D3) show a tension mode failure where the strain in the tension steel bars was greater than that of the strain in

FIGURE 18: Load-strain behaviour of GPCC (DRB).

geopolymer concrete and compression steel. Overall, the effect of the process of crack formation can be noticed on the moment-strain curves in both geopolymer DRBs.

3.6.3. End Rotation. The end rotation of all geopolymer beams is given in Figures 19 (SRB) and 20 (DRB). GPCSC is subjected to a larger rotation when compared to GPCC. The moment rotation curve follows a linear behaviour up to the yield point and beyond that, there was a rapid increase in rotation. The obtained results are consistent with the work of the other researchers. The end rotation curvature pattern of GPCSC is similar to GPCC, and also they are conservative with the similar work carried out using coconut shell concrete [10].

There are some advantages and disadvantages of the GPCSC mix compared to the GPCC mix. The advantage is GPCSC density is less compared to GPCC because of coconut shell density ($550\text{--}650\text{ kg/m}^3$) compared to the conventional stone aggregate density ($1600\text{--}1800\text{ kg/m}^3$). Also, due to the fibrous nature of coconut shell aggregate compared to conventional stone aggregate, naturally, the ductility of GPCSC is more compared to GPCC and it is more advantageous especially in the case of seismic resistance. A disadvantage of using coconut shells in urban areas is transportation cost. Therefore, it is most advantageous that

FIGURE 19: End rotation of SRB.

FIGURE 20: End rotation of DRB.

the coconut shell is used in rural areas where it is dumped as waste.

4. Conclusions

The methods reviewed in this article demonstrate that it is possible to obtain crushed coconut shell as an effective replacement of coarse aggregate and ground granulated blast furnace slag can be completely used as a replacement for conventional OPC to produce lightweight geopolymer concrete.

Based on the experimental results, the following conclusions were made:

- The comparative assessment clearly shows the competitiveness between lightweight geopolymer coconut shell concrete (GPCSC) and geopolymer conventional concrete (GPCC) systems from an environmental point of view.

- (ii) The superior properties of GPCSC demonstrate a promising perspective for the concrete industry in the future. The highest compressive strength of about 32.60 MPa was achieved at 28 days for the mix with 100% CS replacement as coarse aggregate and it is mainly based on the binder, molarity, alkaline binder ratio, and alkaline activator used in the GPCSC mix.
- (iii) The hardened density of GPCSC was found to be 1980 kg/m^3 , satisfying the criteria of lightweight aggregate concrete.
- (iv) Nearly similar cracking pattern, load-deflection behaviour, the ultimate moment of resistance, deflection capacities, end rotation of the beam, and strain readings were obtained from all the geopolymer tested beams.
- (v) The cracked response of the beams at the initial stage was relatively comparable since the strength of the beam depends on the geopolymer concrete strength.
- (vi) The increase in reinforcement ratio had a significant effect on first cracking load and ultimate load, for both GPCSC and GPCC beams.
- (vii) Beams with lower reinforcement ratios (both SRB and DRB) experienced a fewer number of cracks and a higher value of crack width for GPCSC and GPCC beam, while the beams with higher reinforcement ratio (both SRB and DRB) experienced numerous cracks and less value of crack width. But the number and width of the crack of GPCSC were larger when compared with GPCC for both singly reinforced and doubly reinforced geopolymer beams. The average increase in crack width was 31% for singly reinforced and 12% doubly reinforced geopolymer coconut shell concrete beam (GPCSC) when compared with geopolymer conventional concrete beam (GPCC).
- (viii) The measured maximum deflections of beams underestimate the predicted deflections by calculation using the provisions of IS 456:2000 and conventional RC theory shows fair agreement but call for improved prediction for geopolymer beams.
- (ix) The experimental moment is compared with the theoretical moment calculated using the stress block analysis of IS 456:2000 and found to be 3% to 15% higher than the theoretical moment. The experimental moment capacity of the GPCC beams investigated in the study was found to be more than that of the GPCSC beams because of their higher compressive strength.
- (x) The ultimate moment carrying capacity of the test beams calculated using the conventional reinforced concrete principles and strain compatibility

approach showed a good correlation between the test and calculated values as per codal provision. The studies showed that the computational methods used for evaluating the performance parameters of the conventional concrete beams at different stages can also be extended to geopolymer beams.

- (xi) The ductility index of all the geopolymer concrete beams is found to be in the range of 2.11 to 3.58, due to good ductility behaviour and it provides sufficient warning before failure. It reveals that both GPCSC and GPCC have good toughness characteristics.
- (xii) Both single and doubly reinforced geopolymer beams (GPCSC and GPCC) undergo failure by the yielding of steel reinforcement in the tension zone, followed by crushing of concrete in the compression zone. A large deflection is observed in all GPCSC which indicates that the coconut shell concrete fails in a ductile manner.
- (xiii) The end rotation of the GPCSC beams investigated in the study was found to be more than that of the GPCC beams because of the good flexural behaviour of coconut shell concrete.

Overall, the geopolymer coconut shell concrete with GGBS is suitable to be utilized as a sustainable eco-friendly construction material as the coconut shell is a renewable and naturally available resource, while GGBS is an industrial waste.

Data Availability

All data used to support this study are included within the article.

Conflicts of Interest

The authors declare no conflicts of interest.

Authors' Contributions

K. Gunasekaran and S. Nithya contributed to conceptualization; S. Nithya contributed to methodology, investigation, and original draft preparation; K. Gunasekaran and G. Sankar contributed to review and editing and supervision. All authors have read and agreed on the published version of the manuscript.

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