Analysis and Design of Shear Capacity of Sisal Fibre-Reinforced Concrete Member in Rectangular Cross-Section

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1 Introduction

Natural fibre materials have sparked a lot of attention in recent decades from fundamental research to industrial applications. Concrete is characterised by compression strength and tension weakness [1]. The use of lightweight aggregate concrete frequently saves 10–20% of the overall cost of comparable standard. The lighter weight concrete allows for smaller structural element sections, while the lower density allows for smaller structural element sections. The most pressing concern in today's society is environmental deterioration. Many scientists focused their efforts on developing environmentally friendly materials and products based on ecologically sustainable concepts [2]. As a result, natural fibres have replaced mainly synthetic fibres due to their lower cost, ease of availability, minor waste collection challenges, equivalent strength, non-toxicity, and environmentally favourable surroundings.

Sisal fibre is a substantial natural fibre derived from a Sisal tree and is widely accessible in Andhra Pradesh. Bio fibres appear to be recyclable materials that might be pushed in the building industry [3]. There is a global trend towards using both treated and untreated industrial by-products, household wastes, and other wastes as cement and concrete raw materials. Sisal fibre helps not only with garbage repurposing but also with the production of a cleaner, greener environment [4]. The fibre utilised for reinforcement is called sisal fibre, and it is considered an alternate material in mount. This sisal fibre is thought to be the most cost-effective in terms of production while also providing social and economic benefits [5]. If sisal fibre is utilised for structural purposes, it will be helpful not only to the environment but also to low-income families, particularly in the vicinity of sisal farms. SF beams flexure

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and shear behaviour are crucial parameters for structural applications and must be thoroughly studied and established [6]. As a necessary consequence, the results of an experimental investigation of the shear behaviour of reinforced SF beams are presented in this paper [7].

2 Sisal

The Agave fibre is monocotyledonous, with roots that emerge from the base of the pseudostem. Sisal plants have sharp-edged leaves that grow to be 1.5–2 metres tall. Budding immature leaves feature small teeth along the margins of the leaves, as do adult leaves. Sisal plants have a life span of 7–10 years and yield 200–250 good leaves. Each bud comprises thousands of fibres [8]. The fibres account for 4% of the plant's bulk. Agave sisalana Perrine (Agavaceae), sometimes known as the sisal plant, is a monocotyledonous plant native to Mexico. Sisal fibre is the first natural fibre employed in corporate applications, accounting for more than half of all fibres used, which grows under 25 °C, and sunshine, which is considered to be a plant of the tropics and subtropics [9].

2.1 Process of Sisal Fibre Extraction

A retting process and a mechanical process are the two types of extraction processes for natural fibre. Decortication is the process of removing fibre from the plant. After crushing new leaves, the leaves are pounded in a revolving wheel with the point of confinement edges, and only the fibres remain at the end of the operation. The natural sisal fibre and property of sisal fibre are shown in Fig. 1 and Table 1. The leftover fibres are rinsed in water to remove any lingering leaves. The fibres were then dried, brushed, and bagged for use in concrete [10]. The yield of fibre in the retting process

Fig. 1 Natural sisal fibre

Table 1 Property of sisal fibre

Diameter	Density	Tensile strength	Elastic modulus	Elongation break
50–200 mm (micro-m)	1450 kg/m^3	68 MPa	3.77 GPa	5–14%

is around 5.5%, whereas the product in the mechanical function is approximately 3–4% based on the weight of green leaves. Hand extraction equipment, such as serrated or non-serrated blades, is used to extract the fibre.

2.2 Materials

The materials, which used for experimental verification of this study, are ordinary portland cement (OPC)—53 grade, natural fine aggregate—sand (IS 383:1970), natural coarse aggregate—crushed 12 mm and 20 mm maximum size (IS 383:1970), natural fibre—sisal fibre, steel—Fe500, Ye—500 N/mm^2, and ultimate tensile strength—545–600 N/mm^2.

3 Test Program

3.1 Details of Beam

A cube specimen and beams were built and tested as part of this experimental effort. The beams were made of sisal fibre. This beam is developed using the Park and Pauly technique, which includes inclined stirrups that resist the principal tensile force acting in an inclined direction, causing stress reversal through reinforcement. Beams were examined for standard shear reinforcements (NS) and Park and Pauly method shear reinforcements in each case (PS). In each pair, two parts of 10 mm (NS1 and PS1) and two parts of 12 mm (NS2 and PS2) diameters were used as reinforcements. Three sets of cubes and cylinders were tested on the same day as the beams to determine the concrete's properties. According to several publications, the size of the beam for shear testing must be 2600 m; if it exceeds the restrictions, the result will be inaccurate. For the reasons stated above, the following beam size is intended for the experimental specimen [11]. The beams' breadth (b), depth (d), and length (l) remained constant at 150 × 100 × 2600 mm, respectively. The size and length of the beam were designed to assure shear failure. PS beam reinforcement was developed in accordance with IS 456:2000, with minimum shear reinforcement standards also taken into account, guaranteeing that at least one web reinforcement intercepted a diagonal fracture. As a consequence, mild steel two-legged shear reinforcements with a diameter of 6 mm were put across the beams at 150 mm centre to centre [12]. Four-point load testing with a constant effective span was performed on the beam
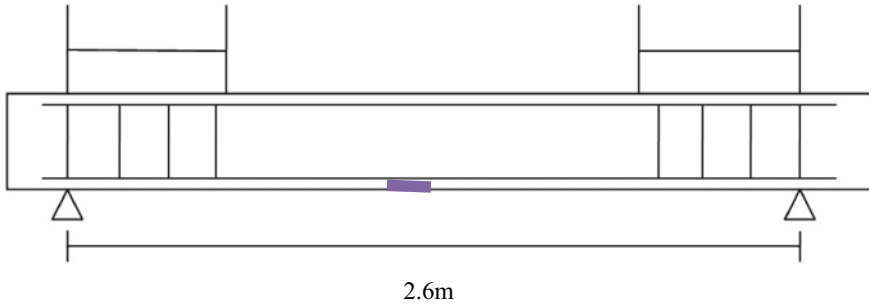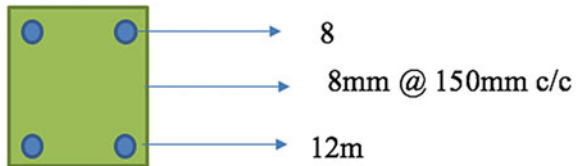
2.6m

Fig. 2 Normal shear reinforcement

2.6

Fig. 3 Park and Pauly method shear reinforcement

Fig. 4 Cross-sectional rod details


8

8mm @ 150mm c/c

12m

specimens. Figures 2, 3, 4, and 5 illustrate a beam and its reinforcing configurations in longitudinal and cross-sectional views using the Park and Pauly technique.

3.2 Beam Preparation and Instrumentation

For the beam size, formwork made of plywood was created. A 20-mm section of tension bar at the mid-span was ground smooth to permit the attachment of strain gauges. To remove rust, dust, and grease, the surfaces were smoothed with sandpaper and cleaned with acetone. The strain gauge was verified for 120 ± 1 X after it was

Fig. 5 Cross-sectional view
of park pauly method shear
reinforcement
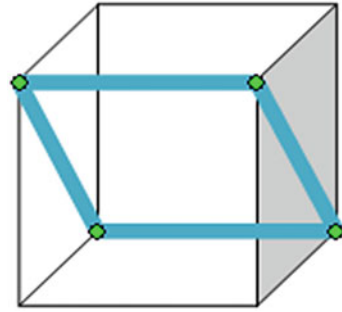
attached with steel reinforcement. The beams were immediately covered with a
plastic sheet following the beam casting to reduce water evaporation from the beam
specimen's surface. The formwork sides were removed after 24 h and treated with
a damp gunny bag on a regular basis for 28 days, following which the beams were
set alone until testing.

Beams were painted and the area on the beam's surface where strain gauges were
mounted was treated, prior to testing. The strain gauges on the outside, the effective
spans, the neutral axes, and the centre lines were all measured and recorded. To
smooth out the treated surface, fine sandpaper was scraped over it. The strain gauge
lead wires were linked to a 10-channel strain bridge, which was then partially to
the data logger. A linear variable displacement transducer (LVDT) was inserted in
the beam's centre, and one dial gauge was set at the (1/4)th distance on either side
of the span to measure deflection. The strain gauge diagram of the test is shown in
Fig. 6. The load was applied in 1 kN increments at first, increasing to 6 kN in the
end. Manual tension and deflection measurements were also taken. During testing,
the beams were preloaded with a force of around 0.5 kN to activate the LVDT and
strain gauges.

Fig. 6 Strain gauge

4 Results and Discussion

4.1 Structural Behaviour

All of the beams had typical structural shear behaviour. Since there were no horizontal fractures at the reinforcement level, there were no instances of bond separating the concrete from the reinforcement Table 2 shows the experimental data for the first shear fracture, ultimate shear loads, and shear force at yield, whereas Table 3 shows as a result of displacement ductility (Fig. 7).

For simplicity, the structural shear behaviour of all of the beams was conventional. There were no instances of bond separating the concrete from the reinforcement since there were no horizontal fractures at the reinforcement level.

The ductility ratio, defined as the displacement ratio at the yield to displacement at ultimate values, was 2.50–3.70 for SFRC beams and 1.80–2.67 for CC beams.

Table 2 Shear fracture, shear force, and ultimate beam shear

Type of beam	Shear fracture (kN)		Shear force (kN)		Ultimate shear (kN)	
	CC	SFRC	CC	SFRC	CC	SFRC
Normal shear reinforcement NS1	12.23	15.67	18.45	22.98	23.78	25.45
Normal shear reinforcement NS2	13.65	16.87	19.87	24.78	28.89	30.67
Park and Pauly method shear reinforcement PS1	15.44	17.82	40.42	41.67	51.34	55.78
Park and Pauly method shear reinforcement PS2	16.67	18.98	39.33	45.89	50.11	58.22

NS, Normal shear reinforcement; PS, Park and Pauly reinforcement; SFRC, Sisal fibre reinforcement; CC, Control concrete

Table 3 Displacement ductility experiment data

Beam design	Upper yield displacement (mm)		Ultimate displacement (mm)		Ductility ratio	
	CC	SFRC	CC	SFRC	CC	SFRC
Normal shear reinforcement NS1	15	16	30	65	2.67	3.50
Normal shear reinforcement NS2	22	24	31	70	2.11	3.90
Park and Pauly method shear reinforcement PS1	13	14	20	50	2.89	2.68
Park and Pauly method shear reinforcement PS2	15	20	23	61	1.87	2.98

Fig. 7 Shear test setup

This suggests that ductile failure modes were present in all SFRC beams. In the flexure mode, all of the beams with shear reinforcement were failed. Following the emergence of diagonal stress fractures, all beams lacking shear reinforcing broke in shear failure modes.

4.2 Behaviour of Cracking

The decreased flexural strength of SFRC beams may explain the early flexural fractures. They were all found at the flexural zone, there is initial flexural cracking. The flexural and shear cracks in the SFRC beams with normal shear reinforcement and Park and Pauly method reinforcement were larger and closer together than in the similar CC beams. These findings imply that the connection between the tension reinforcement and the SRRC is as strong as that between the tension reinforcement and the CC. Additionally, the higher frequency of fractures in SFRC beams resulted in smaller crack widths when compared to CC beams with larger crack widths. Without shear reinforcement at the support, diagonal tension fractures appeared in all SFRC and CC beams, and these cracks advanced towards load regions. Cracks in SFRC beams propagated more zigzag than smooth fractures in CC beams [13]. A strong aggregate interlock in SFRC is the most plausible explanation for this phenomenon.

4.3 Behaviour of Deflection

Figures 8 and 9 demonstrate the experimental mid-span deflections for beams reinforced with the Park and Pauly PS1 and PS2 shear reinforcing systems. Figures 10 and 11 depict the experimental mid-span deflections of beams reinforced with average shear (NS1 and NS2). The early linear part of all curves demonstrates that beam stiffness remains constant prior to flexural cracking. The performance of the SFRC beams was comparable to that of the CC beams at this level. When flexural fractures appeared, the stiffness of both the SFRC and CC beams was dramatically reduced. The deflections of the SFRC beams with average shear and Park and Pauly method

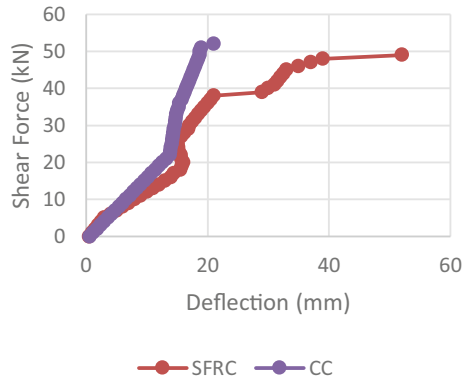**Fig. 8** Force versus
deflection of beams PS1

Fig. 9 Force versus
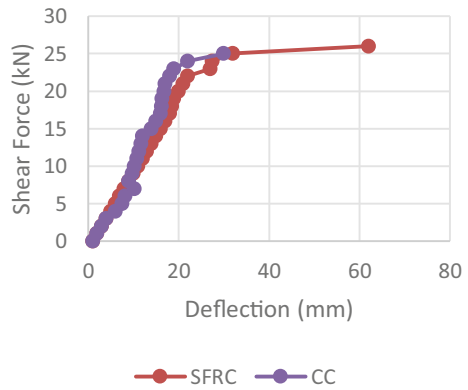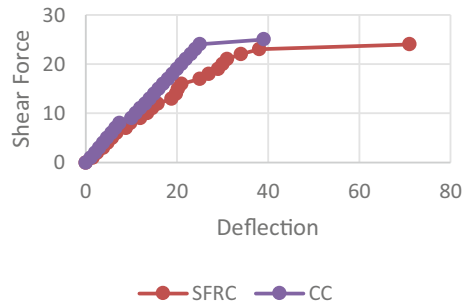deflection of beams PS2

Fig. 10 Force versus beam
deflection PS1

Shear reinforcement was greater than in CC beams. This might be because, despite deflections under service loads, SFRC has a lower elasticity modulus than CC [14]. Both types of concrete met acceptable standards. The ultimate deflection of SFRC shear reinforcement beams was greater than that of CC shear reinforcement beams.
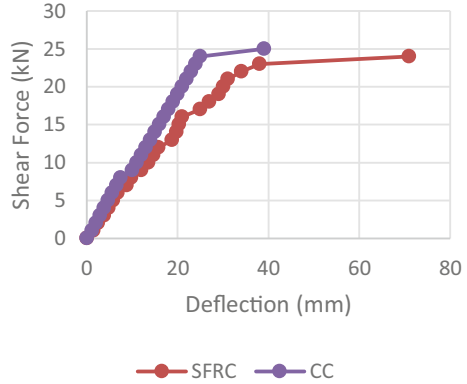
Fig. 11 Force versus beam deflection PS2

Table 4 Experimental result of beam

Beam type	Shear force (kN)	
	CC	SFRC
PS1	27.88	25.78
PS2	50.34	48.67
NS1	21.56	20.87
NS2	24.89	22.54

4.4 Comparison of Experimental

The beams' shear resistance was obtained and in comparison to the experimental results provided in Table 4. At the reinforcing level, no horizontal cracks have been seen, indicating that no bond failures occurred [15]. SFRC beams had around double the number of shears and flexural fractures as CC beams. Fracture spacing was determined to be closer in SFRC beams than in CC beams. The shear strength of the SFRC normal shear reinforcement beam was the same as that of the corresponding CC beams. Beams showed significant bending, providing enough notice of the impending failure.

5 Conclusion

Shear-reinforced beams frequently failed in flexure mode, but ordinary shear-reinforced beams failed in diagonal shear modes. The ductility ratios of SFRC beams with normal shear reinforcements and Park and Pauly technique shear reinforcements are more than the values for CC beams. There were no horizontal fractures seen at the reinforcing level, indicating no instances of bond failure. SFRC beams had approximately doubled as many shears and flexural fractures as CC beams. Fracture spacing

in SFRC beams was found to be closer than in CC beams. Shorter, narrower deformation with rough surfaces in SFRC normal shear reinforcement beams revealed excellent aggregate interlock, boosting the SFRC beams' shear strength. The SFRC shear reinforcement beam's shear strength was equivalent to that of comparable CC beams. Beams displayed substantial bending, indicating that it was on the verge of collapsing. Under shear loadings, the concrete compression strain and steel tension strain findings show that SFRC can attain its maximum strain capacity. In SFRC beams, slightly higher steel and concrete stresses suggested a strong SFRC steel connection.

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