

EXPERIMENTAL DEMONSTRATION OF THE POOR STRUCTURAL PERFORMANCE OF BAMBOO-REINFORCED CONCRETE FLEXURAL MEMBERS

Kent A. Harries, University of Pittsburgh, USA, kharries@pitt.edu

Chase Rogers, University of Pittsburgh, USA

Elayne Silva, Federal Center of Technological Education of Minas Gerais, Brazil

ABSTRACT

Bamboo-reinforced concrete is an ill-advised concept. Despite this, many researchers continue to investigate the replacement of steel with bamboo as reinforcement in concrete flexural members. This study provides a side-by-side comparison of the flexural behaviour of plain unreinforced concrete, steel reinforced concrete and two variations of bamboo-reinforced concrete comparable to the steel reinforced beam tested. As expected, the bamboo-reinforced concrete performs marginally better than unreinforced concrete but is entirely unable to approach the performance of under-reinforced steel-reinforced concrete. This paper underscores the fact that bamboo-reinforced concrete is not only ill-advised, but may indeed be structurally dangerous.

KEYWORDS

bamboo reinforced concrete; flexure; structural safety

INTRODUCTION

The use of small diameter whole-culm (poles) and/or split bamboo (splints or strips) is often proposed as an alternative to reinforcing steel in reinforced concrete. The motivation for such replacement is typically cost and the drive to find more sustainable alternatives in the construction industry.

In 2018, Archila et al. [1] published a rigorous review of the history and states of both practice and research of bamboo-reinforced concrete (BRC). Archila et al. argued that BRC is an “ill-considered concept” and has a number of practical barriers in terms of building performance and construction [1]:

1. Bamboo culms have characteristic tensile strength on the order of 10% that of reinforcing steel; modulus may be 3%-5% that of steel. In either case, the volume of bamboo required to be comparable to steel is excessive and impractical.
2. Bamboo in tension exhibits a brittle behaviour and is therefore unsuited to replicate the strain-energy absorbing role of steel reinforcement in concrete – especially in seismic applications.

As a result, BRC must be designed to remain uncracked [1, 2]; the presence of 3 to 5% bamboo reinforcing is intended to impart a degree of ductility to the member – and may impart some post-cracking residual capacity – in the event of an overload that results in cracking. A premise of reinforced concrete design – aimed at avoiding brittle failure – is that the reinforced capacity must exceed the cracking capacity; this is typically thought of as a nominal moment capacity, M_n exceeding $1.2M_{cr}$. In steel reinforced concrete, this condition is met by providing a minimum longitudinal reinforcing ratio [3]. Nonetheless, post-cracking behaviour is only possible if there is sufficient bond between the bamboo and concrete [1].

3. Without treatment, the bond capacity of bamboo reinforcement is a fraction of that of steel reinforcement [2, 4].

Some bond-enhancing surface treatments have been demonstrated [5, 6]. Nonetheless, the required ‘uncracked’ design increases concrete member dimensions significantly.

Similarly, the relative poor durability of bamboo is emphasised [1]:

4. The high pH of concrete pore water may degrade lignocellulosic materials [7].
5. Bamboo reinforcement embedded in concrete remains susceptible to biological attack and decay and therefore continues to require chemical treatment [8].
6. Water absorption and hygrothermal cycling result in continuous volumetric change of the embedded bamboo reinforcing leading to interfacial damage and micro- and macro-cracking [9]. These effects increase permeability, driving the deleterious processes further.

Poor durability and bond characteristics of bamboo require through-thickness treatment and additional surface treatment of the bamboo reinforcement, respectively. Such treatments are labour intensive, costly and often utilise materials of known toxicity. They “have little practical value” [10].

Finally, through simple design examples, Archila et al. show that bamboo reinforcement – if used safely – is not an ‘environmentally friendly’ or sustainable alternative to steel [1]. In their examples, approximately 275% more concrete was required for BRC than for comparable steel-reinforced concrete.

Archila et al. [1] has been cited at least 37 times and is the top ranked article published in *Materials and Structures* according to Altmetric [11]. In a review of these citations, a number appear to cite [1] in support of the use of BRC – it is unclear whether the authors have even read [1] in some cases. In many citations, the intent of [1] is correctly reported, although in some cases, then neglected. In no known citation of [1] is its primary thesis refuted: bamboo-reinforced concrete is an ill-advised concept.

Research Objective

The objective of the present study is to illustrate the behaviour of BRC in flexure and to contrast this with unreinforced (plain) concrete and steel-reinforced concrete, illustrating why BRC is structurally unsafe. In the conduct of this study, some practical issues were also identified in terms of the use of BRC.

BAMBOO REINFORCED CONCRETE SPECIMENS

Four 200 mm deep, 150 mm wide and 2300 mm long concrete beams were cast. Beam **U** was entirely unreinforced. Beam **S** was reinforced with two #3 ASTM A615 steel reinforcing bars (diameter, $d = 9.5$ mm; area of single bar $A_b = 71$ mm²) resulting in a gross section reinforcement ratio:

$$\rho_g = A_r/bh = 142/(200 \times 150) = 0.0047 \quad [\text{Eq. 1}]$$

where A_r is the total area of reinforcement provided, h is the overall depth (200 mm) of the beam and b is the width (150 mm) of the beam. The minimum permitted reinforcement ratio for this beam is 0.0037 [3].

Beams **Bf** and **BE** were reinforced with locally obtained *Phyllostachys aurea* bamboo. The measured density of the bamboo was 794 kg/m³ or 0.259 kg/m length. The bamboo was seasoned (dried in an indoor environment) but left untreated – no expectation of biological attack or decay was expected during the testing programme. The amount of bamboo reinforcement provided was based on either strength or modulus as follows:

Beam Bf: providing *strength* equal to that of the two #3 bars provided in Beam **S**. A strength ratio, $f_{\text{steel}}/f_{\text{bamboo}} = 4.4$ was assumed resulting in two culms having total $A_r = 625$ mm² being used. The culms had an average diameter, $D = 29.6$ mm and wall thickness, $t = 3.9$ mm. This resulted in $\rho_g = 0.0208$ and the reinforcing ratio normalised for strength being equal to 0.0047:









$$\rho_f = (A_r/bh) \times (f_{bamboo}/f_{steel}) = (625/(200 \times 150)) \times (91/400) = 0.0047 \quad [\text{Eq. 2}]$$

Beam BE: providing axial *stiffness* equal to that of the two #3 bars provided in Beam S. A modular ratio, $E_{steel}/E_{bamboo} = 11.5$ was assumed. This required five larger culms having total $A_r = 1630 \text{ mm}^2$ being used. The culms had an average diameter, $D = 34.6 \text{ mm}$ and wall thickness, $t = 3.3 \text{ mm}$ and were split into quarters in order to be placed in the forms. This resulted in $\rho_g = 0.0543$ and the reinforcing ratio normalised for modulus being equal to 0.0047:

$$\rho_E = (A_r/bh) \times (E_{bamboo}/E_{steel}) = (1630/(200 \times 150)) \times (17.4/200) = 0.0047 \quad [\text{Eq. 3}]$$

It is noted that the modular ratio $E_{steel}/E_{bamboo} = 11.5$ is quite low. Values of E_{steel}/E_{bamboo} exceeding 20 are common. The lower value was used simply so that the specimen remained constructible. Bamboo material properties and details of each beam are provided in Table 1 along with images of the reinforcement.

Table 1: Beam specimen details

Beam	U	S	Bf	BE
reinforcing	none	2 - #3 steel bars $d = 9.5 \text{ mm}$ $A = 72 \text{ mm}^2$	2 bamboo culms $D = 29.6 \text{ mm}$ $t = 3.9 \text{ mm}$	5 split bamboo culms (20 pieces) $D = 34.6 \text{ mm}$ $t = 3.3 \text{ mm}$
n	0	1	$f_{steel}/f_{bamboo} = 4.4$	$E_{steel}/E_{bamboo} = 11.5$
A_r	0	142 mm^2	625 mm^2	1630 mm^2
f	-	400 MPa	91 MPa (typical range: 70 – 140 MPa)	
E	-	200 GPa	17.4 GPa (typical range: 15 – 20 GPa)	
ρ_g	0	0.0047	0.0208	0.0543
ρ_f			0.0047	0.0123
ρ_E			0.0018	0.0047
photo of bars in form				
photo following concrete placement showing bar geometry				

The concrete mix design and material properties used are presented in Table 2. The concrete was provided by a local ready-mix supplier; the total batch size was 2 m^3 . Beam tests were conducted at a concrete age of 108 days.

Table 2: Concrete mix design and measured material properties

Mix Design	Type I/II cement (ASTM C150)	400 kg/m ³
	fine aggregate (ASTM C33)	703 kg/m ³
	#8 limestone coarse aggregate (ASTM C33)	1026 kg/m ³
	AE: Sika Air 260 (ASTM C260)	1.5 l/m ³
	10N (ASTM C494)	4.5 l/m ³
	Sikaplast 200 (ASTM C494)	3.0 l/m ³
	water	154 l/m ³
Material Properties	slump	140 mm
	air content	6.2%
	unit weight	2310 kg/m ³
	28 day compressive strength (ASTM C39)	$f_c' = 37.8 \text{ MPa (COV} = 0.06)$
	28 day split cylinder strength (ASTM C496)	$3.27 \text{ MPa (COV} = 0.07) = 0.53\sqrt{f_c'}$

Bamboo Specimens

The apparent quality of Beams **U** and **S** was good. Both bamboo beams, however, exhibited some issues with their placement. Swelling of the relatively large diameter culms of Beam **Bf** caused a degree of splitting at one end of the beam as shown in Figure 1. This splitting became apparent shortly after concrete placement. By the time the beam was tested, the other end also exhibited similar splitting.



a) end elevation



b) side elevation

Figure 1: Cracking of Beam **Bf** resulting from swelling of culms during concrete cure.

The 20 quarter-culm splits in Beam **BE** were placed in four layers of five (see Table 1). Due to congestion it was necessary to first place only the lower two layers of bamboo and place the lower portion of concrete. The remaining two layers of bamboo were then placed and a second concrete lift placed. It was not practical to consolidate the concrete across this lift. The congestion in Beam **BE** resulted in severe honeycombing and a lack of consolidation across the concrete lifts. The as-cast Beam **BE** is shown in Figure 2a. This beam was subsequently repaired with SIKAGROUT 212 repair mortar prior to testing. The repaired beam is shown in Figure 2b.



a) elevation and detail as cast



b) same elevation following mortar repair

Figure 2: Beam **BE**.

FLEXURAL TEST PROGRAMME

Each beam was tested in midpoint flexure over a simple span of 2160 mm. Tests were conducted in a 900 kN capacity servo-hydraulic controlled test frame equipped with a reaction beam for flexural testing. All tests were run in displacement control at a rate of 6 mm/min. Total applied load and cross head deflection are recorded from the test frame.

In the following discussion, total applied load, P (kN) is reported. Midspan moment is therefore: $0.54P$ (kNm) and shear is $0.5P$ (kN). The beams have a shear span-to-overall depth ratio of 5.4 and a nominal shear capacity of 23.6 kN (corresponding to an applied load of $P = 47.2$ kN); they are very much flexure-critical members. The midspan deflection, δ (mm), reported is that from the machine crosshead and therefore includes the effects of machine compliance. Although actual beam deflections will be marginally less than those reported, comparison across specimens remains valid.

TEST RESULTS

Figure 3 shows the applied load versus midspan deflection for all beams; Figure 3a shows the entire response while 3b shows only the initial displacements less than 5 mm ($L/432$). Figure 4 shows images of beams following testing and Table 3 presents a summary of key response parameters obtained from this data.

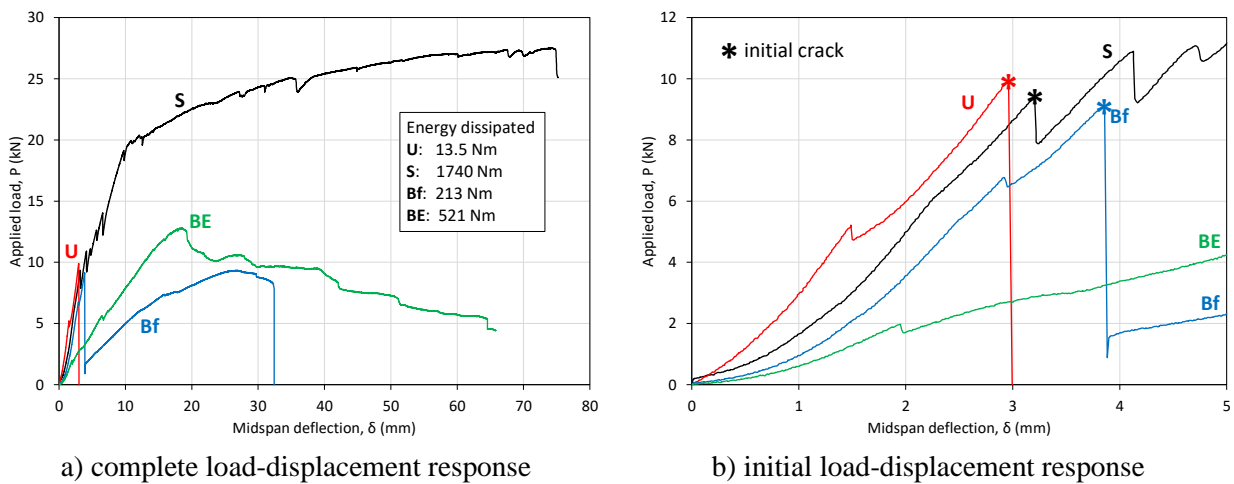


Figure 3: Load-deflection responses of beams.

Table 3: Summary of key response parameters.

		U	S	Bf	BE
load at initial crack, P_{cr}	kN	9.91	9.35	9.09	-
displacement at initial crack	mm	2.96	3.19	3.86	-
load at steel yield	kN	-	19.7	-	-
displacement at steel yield	mm	-	10.6	-	-
peak load	kN	9.91	27.5	9.34	12.8
displacement at peak load	mm	2.96	74.0	26.8	18.6
failure mode		brittle concrete fracture	ductile yield of reinforcing	brittle longitudinal splitting	loss of continuity of section resulting from reinforcement congestion
energy dissipated	Nm	13.5	1740	213	521

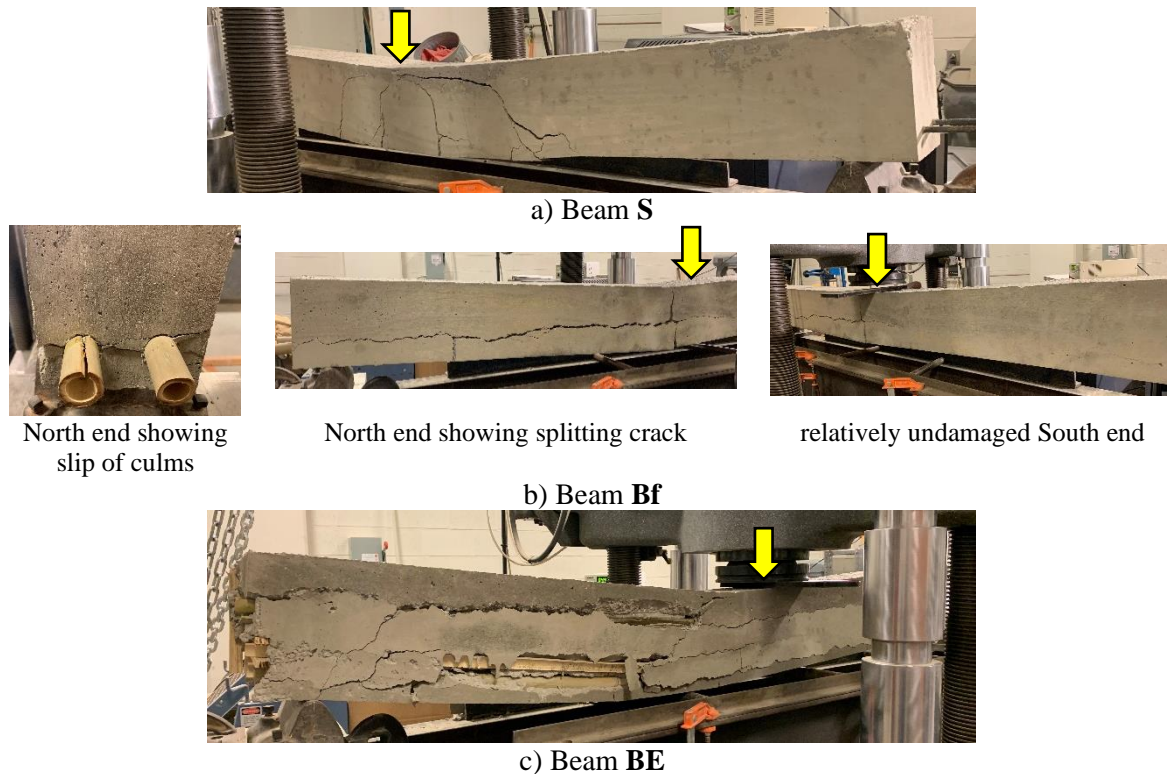


Figure 4: Beams following testing (location of applied load shown)

Cracking Load and Beam U Response

The load to cause initial concrete cracking should, theoretically, be the same for all four beams and is easily determined from the initial loading curves (Figure 3b) as the first drop in load. Based on the cracking load of Beam **U** (9.91 kN), the *in situ* modulus of rupture of concrete may be estimated to 5.35 MPa. **S** and **Bf** exhibited very similar cracking loads (9.35 kN and 9.09 kN, respectively). The poor concrete placement of **BE** resulted in this beam behaving as being initially cracked, resulting in a considerably less stiff initial behaviour and no discernible cracking load (Figure 3b). If **BE** had been initially uncracked, the cracking load would be similar to the other beams, approximately 9 to 10 kN applied load. Following cracking, the internal reinforcement must resist the tension crossing the crack in order to maintain equilibrium of the cross section. Thus, upon cracking the unreinforced beam **U** fails entirely.

Beam S Response

Beam **S** exhibited a very clear change in stiffness corresponding to steel yield at an applied load of 19.7 kN, approximately twice the cracking load despite the small reinforcement ratio. As loading continued, the neutral axis shifted toward the compression face and the steel likely exhibited strain hardening, the capacity continued to increase to 27.5 kN, 277% of the cracking load and 140% of the yield load. Failure at 27.5 kN corresponded to the formation of flexure-shear crack (Figure 4a) and some loss of bond capacity of the reinforcing bars near midspan. A residual capacity of about 20 kN remained (not shown in Figure 3). The displacement at the peak load was 74 mm ($L/29$) and corresponded to a ductility ratio of 7. Beam **S** behaved as one would expect of an under-reinforced concrete beam.

Beam Bf Response

Beam **Bf** exhibited a precipitous drop in load capacity corresponding to concrete cracking at 9.09 kN. The fact that the internal bamboo reinforcement was unable to mitigate this dramatic loss of capacity is an indication of the lack of bond between bamboo and concrete. A small residual capacity of about 10% of the cracking load remained and the test was restarted. The second phase of the test essentially tests **Bf** as an initially cracked section. Thus the post cracking stiffness is considerably reduced. **Bf** eventually recovered a load marginally greater than its cracking load (9.34 kN) but was unable to

sustain this and failed catastrophically (Figure 4b) at a displacement of 26.8 mm. The reinforcing ratio normalised for strength, $\rho_f = 0.018$, is about one half that prescribed for minimum reinforcement (0.0037) to control post-cracking behaviour. Therefore the observed behaviour should be expected. Longitudinal splitting failure, as shown in Figure 4b is indicative of complete loss of reinforcement bond with the concrete.

Beam BE Response

As stated previously, **BE** behaved as though initially cracked. **BE** exhibited a stiffer response than the post-cracked response of **Bf** and achieved a capacity of 12.8 kN, greater than 120% of its presumed cracking load. **BE** also exhibited a less brittle post-peak behaviour (Figure 3a) although the residual capacity rapidly fell below 80% of the peak load which is a conventional definition of failure. The behaviour of **BE**, shown in Figure 4c, may have resulted from its vertically layered construction – as described above – allowing some relative movement along the weak panes of both the bamboo strips and at the concrete lift boundaries.

Energy Dissipation

An important measure of structural performance and ductility is energy absorption – measured as the area under the load-displacement curve in this instance. Using **U** as a baseline, it is seen that the ductile behaviour of **S** absorbs 130 times the energy. **Bf** and **BE** absorb an order of magnitude less: 16 and 39 times that of **U**, respectively.

‘Real World’ Loading

The beams were tested under conditions of increasing displacement and the corresponding load was recorded (i.e., ‘displacement control’). In the ‘real world’ loading is ‘load controlled’. A higher load is applied and – due to the effects of gravity – is not shed as the member deflects. Thus, in the case of **Bf**, and presumably **BE** had it been uncracked, failure would correspond to beam cracking – just as if the beam were unreinforced (Beam **U**). The precipitous drop in load-carrying capacity upon cracking would be catastrophic under load control or simply static gravity load conditions. Neither **Bf** nor **BE** would have had the opportunity to recover and exhibit their residual capacities seen in this test program. Regrettably, in the laboratory, load control is dangerous and should not be used except when a very ductile response is anticipated. Using load control also eliminates the opportunity to investigate a post-peak response.

Immediate Post-Cracking Residual Capacity

In steel reinforced concrete, initial cracking only results in a small immediate load drop as the steel reinforcement engages ‘immediately’ to resist the unbalanced tension force resulting from the concrete cracking. This is possible because of the so-called ‘perfect’ bond between steel and concrete. The dramatic drop in capacity of **Bf** upon cracking is a clear indication that bond between the bamboo culm and concrete was minimal at the location of the crack. Since **BE** has much more bamboo and greater surface area of bamboo, the bond should be better than in the case of **Bf**. Thus the drop in load upon initial cracking for **BE** may not have been as dramatic as in **Bf**.

CONCLUSIONS

The experimental programme presented clearly illustrated the poor structural behaviour of bamboo reinforced concrete described in depth by [1]. Regardless of whether designed for equivalent strength (**Bf**) or stiffness (**BE**), the bamboo reinforced concrete beams exhibited precipitous loss of capacity upon cracking and were barely able to recover their initial cracking loads. The findings of this study confirm the premise that bamboo reinforced concrete must be designed to remain uncracked. Furthermore, the minimum ratio of bamboo reinforcement required to control post-cracking behaviour should be taken such that $\rho_E > 0.0037$ (Eq. 3).

Furthermore, the results show that the bamboo reinforcement, when designed based on equivalent strength (**Bf**), is unable to improve upon the cracking strength of the plain concrete beam and would therefore also fail in a brittle, catastrophic manner.

As expressed in [1], designing bamboo reinforcement for equivalent stiffness (**BE**) results in marginal improvement in behaviour following cracking. However, as demonstrated in the construction of **BE**, producing a constructible design is difficult with such a large reinforcing ratio required.

Video recordings of the beam tests presented may be viewed at: https://youtu.be/vn_5ae5-HdA.

POSTSCRIPT: EQUIVALENT UNCRACKED BEAMS

As a point of comparison, limiting the bamboo-reinforced concrete beams to an uncracked response, the cross section of **Bf** or **BE** would have to be increased about three-fold to approximately 350 x 250 mm to have the same capacity as **S**. While affecting a savings of 2.7 kg of steel per beam, the concrete weight would increase from 180 kg to 510 kg per beam. This is not a sustainable alternative. Furthermore, the volume of bamboo required also increases with the concrete section since it is necessary to provide a minimum reinforcement ratio.

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CONFLICT OF INTEREST

The authors declare that they have no conflicts of interest associated with the work presented in this paper.

DATA AVAILABILITY

Data on which this paper is based is available from the authors upon reasonable request.