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Numerical Design and Three-Dimensional Finite Element Analysis of a Conventional RCC Beam and a Timber-Core Composite RCC Beam

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ABSTRACT

Reducing the dead load and cement consumption of reinforced concrete (RCC) members is an important step towards lower-carbon construction, and lightly stressed zones near the neutral axis offer an obvious opportunity. This paper investigates a timber-core composite RCC beam in which a central 100×150 mm Sal-timber core replaces part of the concrete in a $230 \times 300 \times 1000$ mm beam, and compares it with an otherwise identical conventional RCC beam. Both beams (M30 concrete, Fe500 steel, 2–12 ϕ bottom and 2–10 ϕ top bars, 8 mm stirrups at 150 mm c/c, fixed–fixed supports) were designed to IS 456:2000 by the limit state method and analysed under a service uniformly distributed load of 80 kN/m plus self-weight using a dedicated three-dimensional linear-elastic finite element (FE) model with 8-node brick and embedded truss elements (14,976 bricks, 51,357 degrees of freedom). The FE model satisfies global equilibrium (error < 0.001 %), is mesh-converged to better than 1 %, and reproduces the analytical Timoshenko-beam deflection within 3.1 %. The timber core reduces self-weight by 14.0 % and concrete volume by 20.7 % while leaving the flexural capacity unchanged (24.04 kN·m sagging, 17.11 kN·m hogging). The cost is a 13.2 % larger mid-span deflection (28.8 → 32.6 μ m), an 8.3 % higher peak concrete tensile stress, a 2.8 % lower cracking load and a 6.3 % lower (conservative) shear capacity. Per unit self-weight, flexural rigidity, flexural capacity and shear capacity improve by 13.1 %, 16.3 % and 9.0 %, respectively. The results show that a timber core placed in the neutral-axis zone is a structurally acceptable means of lightening lightly stressed RCC beams.

Keywords: *timber-core composite beam; reinforced concrete; Sal timber; finite element analysis; IS 456:2000; fixed–fixed beam; Timoshenko beam; self-weight reduction*

HIGHLIGHTS

- A 100×150 mm Sal-timber core was embedded in the neutral-axis zone of a 230×300 mm RCC beam.
- Design to IS 456:2000 and a 3D FE model (51,357 DOF) were developed for both beams and cross-validated.
- The core cuts self-weight by 14.0 % and concrete volume by 20.7 % with no loss of flexural capacity.
- Deflection increases by 13.2 %, chiefly through reduced shear rigidity; stresses remain below cracking.

1. Introduction

Reinforced concrete remains the dominant structural material worldwide, but its environmental cost is substantial: cement manufacture is a major source of anthropogenic CO₂ and concrete is the most widely consumed construction material after water [1,2]. Inventory data show that the embodied energy and carbon of structural concrete are driven largely by its cement content, whereas structural timber carries a much lower embodied-carbon burden and stores biogenic carbon [3,4]. Replacing part of the concrete in a member with a renewable material such as timber is therefore an attractive route to lighter, lower-carbon construction, provided structural safety and serviceability are not compromised.

Timber and concrete have been combined for decades in *timber–concrete composite* (TCC) floors and bridges, in which a concrete slab acts in compression above timber joists acting in tension, and the two are joined by mechanical shear connectors [5–7]. The performance of TCC systems is governed chiefly by the stiffness and strength of the connection, which controls the degree of composite action [8,9], and by the long-term behaviour of timber and concrete under creep and shrinkage [10]. Design of such systems is codified, for example, through the γ -method of Eurocode 5 [11,12]. In all of these systems, however, the timber is the principal tension element and the concrete is the principal compression element.

A different strategy is examined here. In a rectangular RCC beam the material close to the neutral axis is lightly stressed in bending: it contributes little to the flexural lever arm, and the tension and compression resultants are carried almost entirely by the outer fibres and the reinforcement [13]. Replacing a central block of this zone with a lighter material can therefore reduce dead weight and cement consumption without altering the flexural capacity governed by the bars and the compression block. This concept — a *timber-core composite RCC beam* — has received much less attention than conventional TCC systems, and its effect on stiffness, shear behaviour, stress distribution and efficiency has not been

systematically quantified for the common Indian combination of M30 concrete, Fe500 steel and Sal (*Shorea robusta*) timber.

Finite element (FE) analysis is the natural tool for such a comparison. Since the pioneering work of Ngo and Scordelis [14], FE models have been used extensively to study RC members, and the theoretical basis for three-dimensional solid-element analysis is well established [15–17]. Nonlinear approaches based on fracture energy, crack-band and plastic-damage concepts are available for post-cracking behaviour [18–20], but a carefully verified linear-elastic model is a necessary first step for comparing alternative sections at service load.

The objectives of this paper are therefore: (i) to design a conventional RCC beam and a timber-core composite RCC beam of identical geometry and reinforcement to IS 456:2000 [21,22]; (ii) to develop and verify a three-dimensional FE model of each beam; (iii) to quantify the effect of the timber core on deflection, concrete, timber and reinforcement stresses, cracking load, capacity and self-weight; and (iv) to define efficiency indices that express the trade-off between weight saving and loss of stiffness. The remainder of the paper is organised as follows: Section 2 reviews the relevant literature, Section 3 describes the methodology, Section 4 presents the results, Section 5 discusses them, and Section 6 concludes.

2. Literature review

2.1 Timber–concrete composite members

Ceccotti [5] reviewed concrete–timber composite structures and identified the shear connection as the decisive design parameter, because the slip between the two materials determines how much of the theoretical composite stiffness is actually realised. Yeoh et al. [6] later consolidated the state of the art on TCC floors, covering connection types, design approaches, long-term behaviour and construction practice. Dias et al. [7] discussed how TCC solutions can increase the use of timber in building construction, and Rodrigues et al. [23] reviewed their application in bridges. Experimental work by Gutkowski et al. [8] on composite wood–concrete beam and floor specimens with different connection systems showed how connector type controls composite stiffness and strength, while Lukaszewska et al. [9] examined connections for prefabricated TCC floors, with emphasis on their stiffness, strength and ease of assembly.

Time-dependent behaviour is an important issue for any timber–concrete combination. Fragiacomano and Ceccotti [10] developed and validated an FE model for the long-term response of TCC beams that accounts for creep and shrinkage of concrete and the creep and mechano-sorptive behaviour of timber. Design provisions for timber and composite sections are given in IS 883 [24], Eurocode 5 [12] and the Wood Handbook [25], while concrete design is covered by IS 456 [21], Eurocode 2 [26] and ACI 318 [27].

2.2 Finite element modelling of reinforced concrete members

Ngo and Scordelis [14] introduced FE analysis of RC beams with discrete cracking and bond-link elements. Fracture-mechanics-based approaches followed, notably the fictitious crack model of Hillerborg et al. [18] and the crack-band theory of Bažant and Oh [19], and the plastic-damage model of Lee and Fenves [20] is widely used to describe the cyclic and monotonic response of concrete. For linear-elastic stress analysis, 8-node isoparametric hexahedral elements with $2 \times 2 \times 2$ Gauss integration are standard [15–17]. Exact solutions for shear-flexible beams follow Timoshenko [28], and the elasticity background for stresses near restrained edges is given by Timoshenko and Goodier [29]. Systematic reporting of mesh-refinement studies is recommended by Roache [30].

2.3 Research gap

Table 1 summarises the main contributions reviewed above. Existing studies focus on timber–concrete systems in which timber is the tension flange and a connector system is central, or on FE methodology for RC members in general. To the authors' knowledge, a quantitative numerical comparison of a conventional RCC beam with an RCC beam containing an embedded timber core in the lightly stressed neutral-axis zone — covering design capacity, shear and flexural rigidity, stress fields and efficiency per unit weight — has not been reported for M30/Fe500/Sal-timber sections. This paper addresses that gap.

Table 1. Summary of selected studies relevant to timber–concrete members and FE modelling of RC members.

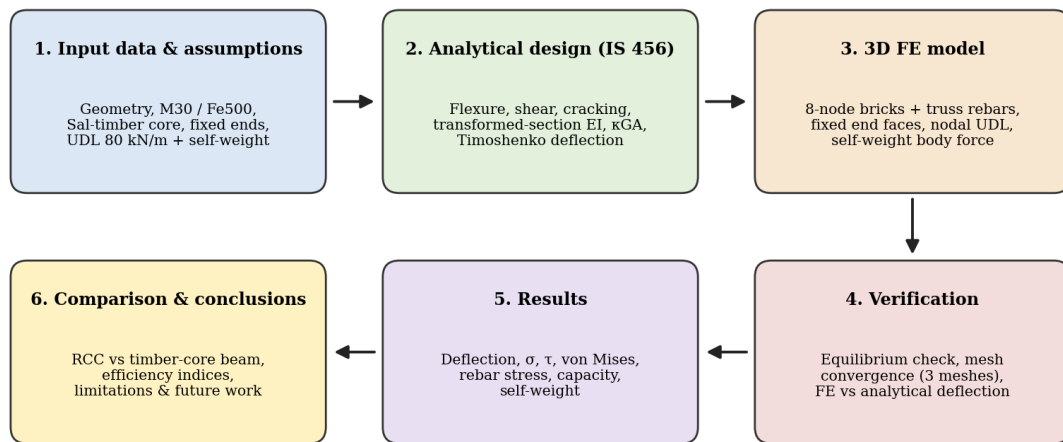
Reference	Year	Focus	Relevance to present study
Ngo and Scordelis [14]	1967	FE analysis of RC beams with discrete cracking	Origin of FE modelling of RC members
Hillerborg et al. [18]	1976	Fictitious crack model; fracture energy	Basis for post-cracking FE extension
Bazant and Oh [19]	1983	Crack band theory	Mesh-objective cracking model
Lee and Fenves [20]	1998	Plastic-damage model for concrete	Nonlinear extension (future work)
Ceccotti [5]	2002	Review of composite concrete–timber structures	Role of connection stiffness in TCC
Fragiacomo and Ceccotti [10]	2006	FE model for long-term behaviour of TCC beams	Creep and shrinkage issues for timber
Lukaszewska et al. [9]	2008	Connections for prefabricated TCC floors	Connection performance
Gutkowski et al. [8]	2008	Laboratory tests of composite wood–concrete beams/floors	Experimental behaviour of TCC members
Yeoh et al. [6]	2011	State-of-the-art review of TCC structures	Design practice and gaps
Rodrigues et al. [23]	2013	Review of TCC bridges	Applications of TCC
Dias et al. [7]	2016	TCC for increasing timber use in construction	Sustainability motivation
This study	2026	Timber-core RCC beam vs RCC beam: IS 456 design + 3D FE	Neutral-axis timber core; weight/stiffness trade-off

Entries are summarised by the present authors from the cited sources.

3. Methodology

3.1 Research framework

The study follows the six-stage workflow shown in Fig. 1. Input data and assumptions are first defined; both beams are then designed analytically to IS 456:2000 and their section properties and Timoshenko-beam deflections computed; a three-dimensional FE model of each beam is built and verified (equilibrium, mesh convergence, comparison with analytical deflection); and finally the responses are compared.

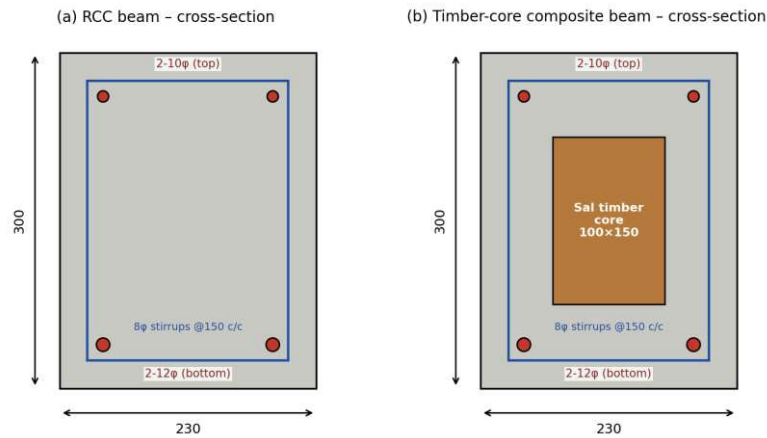
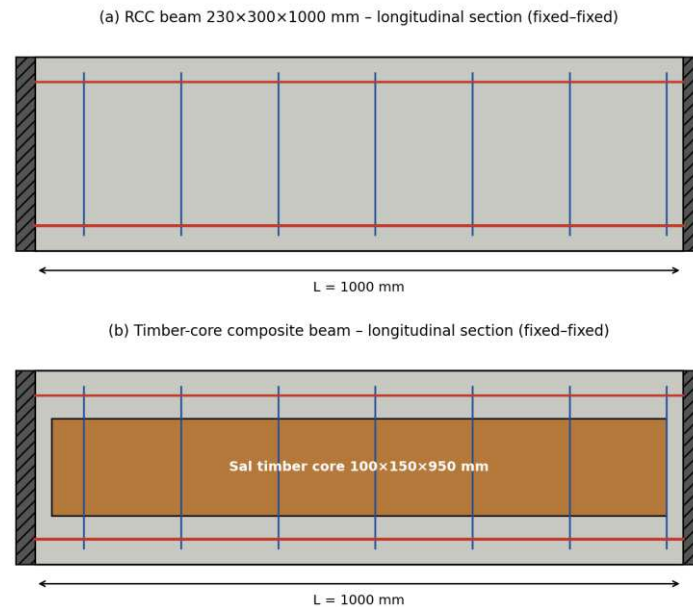
**Fig. 1.** Research workflow adopted in this study.

3.2 Beam geometry and reinforcement

Two beams of identical size (230×300 mm cross-section, 1000 mm length), concrete grade (M30), steel grade (Fe500), clear cover (25 mm) and reinforcement were considered; the only difference is a central Sal-timber core of 100×150 mm cross-section and 950 mm length in the composite beam, leaving 25 mm of concrete at each end (Table 2, Figs. 2 and 3). The core is centred in the section, so its top and bottom faces are 75 mm from the top and bottom concrete faces. Both end faces are fully fixed.

Table 2. Input parameters of the beams.

Sr. No.	Parameter	RCC beam	Timber-core composite RCC beam
1	Size of beam	230 × 300 mm	230 × 300 mm
2	Length of beam	1 m	1 m
3	Grade of concrete	M30	M30
4	Grade of steel	Fe500	Fe500
5	Clear cover (all sides)	25 mm	25 mm
6	Top reinforcement	2–10 mm dia.	2–10 mm dia.
7	Bottom reinforcement	2–12 mm dia.	2–12 mm dia.
8	Lateral ties (stirrups)	8 mm @ 150 mm c/c	8 mm @ 150 mm c/c
9	Central Sal-timber core	—	100 × 150 mm (950 mm long)
10	Boundary condition	Fixed	Fixed

**Fig. 2.** Cross-sections of the (a) RCC and (b) timber-core composite beams (dimensions in mm).**Fig. 3.** Longitudinal sections of the two beams showing stirrups, main bars, the 950 mm long timber core and the fixed ends.

3.3 Materials, loading and assumptions

Concrete properties follow IS 456 [21]: $E_c = 5000\sqrt{f_{ck}} = 27,386$ MPa and flexural tensile strength $f_{cr} = 0.7\sqrt{f_{ck}} = 3.83$ MPa. Sal-timber properties were taken as $E_t = 12,600$ MPa, $\nu = 0.30$ and unit weight 8 kN/m³, representative of the high-strength Group A timbers of IS 883 [24]; these values were assumed because they were not specified in the source data, and should be replaced by project-specific values where available (Table 3).

Table 3. Material properties used in design and FE analysis.

Material	Property	Value	Basis
Concrete M30	Characteristic strength, f_{ck}	30 MPa	Input
	Modulus, $E_c = 5000\sqrt{f_{ck}}$	27,386 MPa	IS 456 cl. 6.2.3.1
	Poisson's ratio / unit weight	0.20 / 25 kN/m ³	Assumed
	Flexural tensile strength, $f_{cr} = 0.7\sqrt{f_{ck}}$	3.83 MPa	IS 456 cl. 6.2.2
Steel Fe500	Yield strength, f_y / modulus, E_s	500 / 200,000 MPa	Input / IS 456
Sal timber	Modulus of elasticity, E_t	12,600 MPa	Assumed (IS 883, Group A)
	Poisson's ratio / unit weight	0.30 / 8 kN/m ³	Assumed (isotropic)

Loading. A service uniformly distributed load (UDL) of 80 kN/m (0.348 N/mm² on the 230 mm wide top face) plus self-weight was applied. The factored design load is $1.5 \times (80 + \text{self-weight}) = 122.6$ kN/m for the RCC beam and 122.2 kN/m for the composite beam. Self-weight is computed from concrete and timber volumes and excludes the reinforcement.

Boundary condition. Both end faces ($z = 0$ and $z = 1000$ mm) are fully fixed (all translations restrained).

Other assumptions. Linear-elastic, small-displacement behaviour (justified in Section 4.4 by peak stresses below the cracking stress); perfect bond between concrete, timber and steel; isotropic timber; stirrups included in the design checks but omitted from the FE model, since they have negligible influence on the un-cracked elastic response.

3.4 Analytical design to IS 456:2000

Both sections were designed by the limit state method [21,22]. For an under-reinforced rectangular section, the neutral-axis depth and moment of resistance are

$$x_u = 0.87 f_y A_{st} / (0.36 f_{ck} b) \quad (1)$$

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u) \quad (2)$$

with $x_{u,max} = 0.46d$ for Fe500. Effective depths are $d = 300 - 25 - 8 - 12/2 = 261$ mm (bottom) and 262 mm (top). The timber core begins 75 mm from the top and bottom faces, whereas the compression block is only 39.6 mm deep, so the core lies entirely outside the compression zone and its small tensile contribution is neglected; the flexural capacity of the composite beam therefore equals that of the RCC beam. Shear capacity is taken as the sum of concrete and stirrup contributions,

$$V_u = \tau_c A_c + 0.87 f_y A_{sv} d / s_v \quad (3)$$

where $\tau_c = 0.426$ MPa for $p_t = 100A_{st}/(bd) = 0.377\%$ and M30 (IS 456 Table 19, interpolated), $A_{sv} = 100.5$ mm² (two-legged 8 mm stirrups) and $s_v = 150$ mm. For the composite beam the concrete shear area A_c is reduced by the 100×150 mm core ($60,030 \rightarrow 45,030$ mm²) and the timber's own shear strength is conservatively ignored. The critical section is taken at the support face, which is conservative. For a fixed–fixed beam under a factored UDL w_u , the support moment, mid-span moment and end shear are

$$M_{sup} = w_u L^2 / 12, \quad M_{mid} = w_u L^2 / 24, \quad V_{end} = w_u L / 2 \quad (4)$$

3.5 Section properties, cracking load and analytical deflection

Un-cracked transformed-section properties about the mid-depth axis were computed with modular ratios ($E_s/E_c - 1$) = 6.30 for steel and $E_t/E_c = 0.460$ for timber, giving the flexural rigidity EI and shear rigidity κGA of each beam. The cracking moment and the UDL that first cracks the fixed–fixed beam (at the supports, in hogging) are

$$M_{cr} = f_{cr} I_{tr} / (D/2), \quad w_{cr} = 12 M_{cr} / L^2 \quad (5)$$

The analytical mid-span deflection of a fixed–fixed Timoshenko beam [28] under a UDL w is the sum of a bending and a shear part:

$$\delta = wL^4 / (384 EI) + wL^2 / (8 \kappa GA) \quad (6)$$

Because the beams are short ($L/D = 3.3$), the shear term is as large as the bending term, and Eq. (6) is used as the independent check on the FE model.

3.6 Finite element model

A dedicated three-dimensional FE model of each beam was implemented in Python using NumPy/SciPy and a sparse direct solver. Concrete and timber were discretised with 8-node linear hexahedral (brick) elements with $2 \times 2 \times 2$ Gauss integration and isotropic linear-elastic properties (Table 3) [15,16]. The four longitudinal bars (2–12 ϕ bottom, 2–10 ϕ top) were modelled as 2-node axial truss elements sharing nodes with the concrete mesh (perfect bond), and the concrete stiffness was subtracted along each bar by $(E_s - E_c)$ to avoid double counting. All nodes on both end faces were fixed in x , y and z . The UDL was applied as consistent nodal forces on the top face and self-weight as a body force (25 kN/m³ for

concrete, 8 kN/m³ for timber). The structured mesh is graded so that element faces coincide with the bar axes, the stirrup-cage line and the faces of the timber core; the reference mesh has a maximum element size of 20 mm (Fig. 6, Table 8).

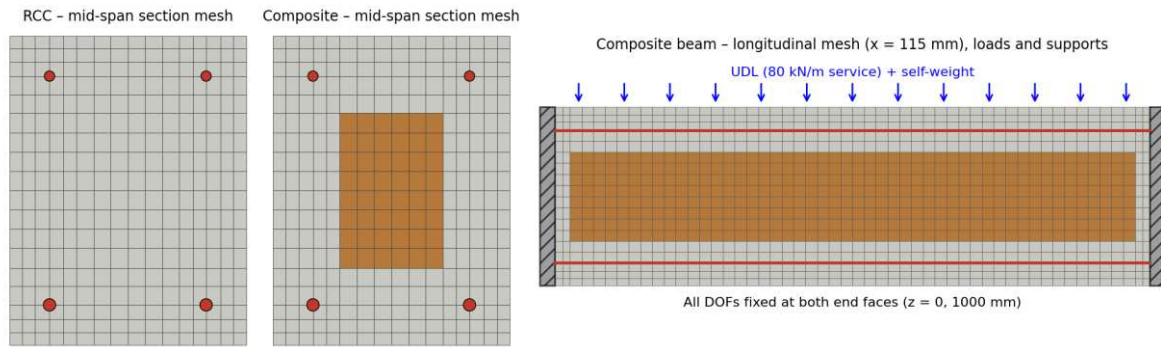


Fig. 6. Finite element model: mid-span section meshes of both beams (left), and longitudinal mesh with loading and supports of the composite beam (right). Brown = timber, grey = concrete, red = rebars.

Table 8. Summary of the reference FE models.

Item	RCC beam	Composite beam
Brick elements (8-node)	14,976	14,976
Nodes	17,119	17,119
Degrees of freedom	51,357	51,357
Truss (rebar) elements	208	208
Max. element size (mm)	20	20
Applied load: UDL + self-weight (kN)	81.725	81.483
Sum of support reactions (kN)	81.725	81.483
Equilibrium error	< 0.001 %	< 0.001 %

3.7 Verification and mesh convergence

Three successively refined meshes (maximum element sizes 50, 30 and 20 mm) were solved for both beams and the mid-span deflection was monitored, following the general practice of reporting grid-refinement studies [30]. Global equilibrium was verified by comparing the sum of support reactions with the applied load (Table 8). The deflection change between the two finest meshes is below 1 %, so the 20 mm mesh was adopted (Table 9, Fig. 7).

Table 9. Mesh convergence of the mid-span deflection at service load.

Max. element size (mm)	Elements	DOF	RCC δ (μ m)	Composite δ (μ m)	Δ vs previous (RCC / Comp.)
50	1,760	6,831	28.486	32.239	—
30	4,760	17,325	28.689	32.472	0.71 % / 0.72 %
20	14,976	51,357	28.837	32.649	0.52 % / 0.55 %

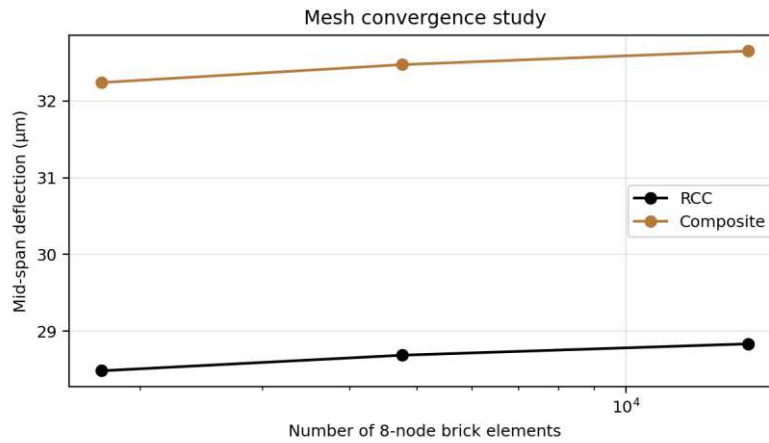


Fig. 7. Mesh convergence study – mid-span deflection versus number of elements.

4. Results

4.1 Design capacity and demand

Table 4 gives the flexural design of the common section: both layers are under-reinforced ($x_u = 39.61$ mm sagging and 27.51 mm hogging, well below $x_{u,max} \approx 120$ mm) and satisfy the minimum steel requirement. The sagging and hogging moments of resistance are 24.04 and 17.11 kN·m, respectively.

Table 4. Flexural design of the section (applies to both beams).

Item	Sagging (bottom 2–12 ϕ)	Hogging (top 2–10 ϕ)
A_{st} (mm ²)	226.2	157.1
Effective depth d (mm)	261	262
Minimum $A_{st} = 0.85bd/f_y$ (mm ²)	102.1 – OK	102.4 – OK
x_u (mm)	39.61	27.51
$x_{u,max} = 0.46d$ (mm)	120.1	120.5
Section type	Under-reinforced	Under-reinforced
M_u (kN·m)	24.04	17.11

For shear (Table 5), the removal of 15,000 mm² of concrete shear area lowers V_{uc} from 25.57 to 19.18 kN, so the shear capacity falls from 101.66 to 95.27 kN (–6.3 %). The stirrup contribution (76.09 kN) is unchanged, and the stirrup ratio (0.291 %) exceeds the 0.092 % minimum.

Table 5. Shear design (critical section at the support face – conservative).

Item	RCC beam	Composite beam
A_{sv} (2 legs of 8 mm) (mm ²)	100.5	100.5
Concrete shear area (mm ²)	60,030	45,030
$V_{uc} = \tau_c \times \text{area}$ (kN)	25.57	19.18
V_{us} (kN)	76.09	76.09
Shear capacity $V_u = V_{uc} + V_{us}$ (kN)	101.66	95.27
Maximum allowed $V_{u,max} = 3.5bd$ (kN)	210.1	210.1
Stirrup ratio $A_{sv}/(b s_v)$ vs minimum $0.4/(0.87f_y)$	0.291 % > 0.092 %	OK

Figure 4 shows the analytical shear-force and bending-moment diagrams under the factored UDL, and Table 7 compares demand with capacity. All utilisation ratios are at most 0.64. The ultimate UDL capacity (Fig. 5) is governed by shear: 203 kN/m (RCC) and 191 kN/m (composite), against 205 kN/m for first hinging at the supports in flexure and 329 kN/m if full moment redistribution were possible. Relative to the factored design loads (122.6 and 122.2 kN/m), the reserve factors are therefore 1.66 (RCC) and 1.56 (composite).

Fixed-fixed beam, factored UDL $w_u = 122.6 \text{ kN/m}$ (analytical)

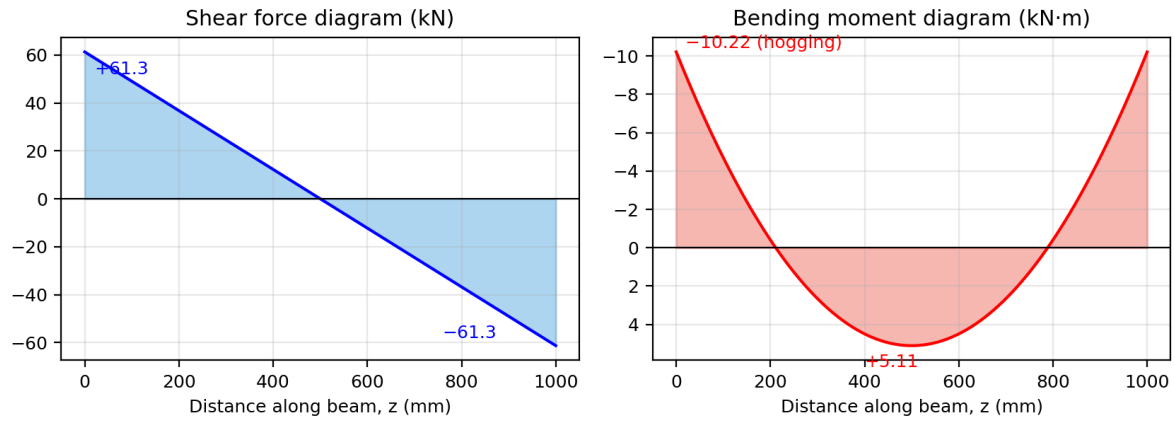


Fig. 4. Shear force and bending moment diagrams of the fixed-fixed beam under the factored UDL (analytical).

Table 7. Design checks at factored load.

Check	Demand RCC	Capacity RCC	Ratio	Demand comp.	Capacity comp.	Ratio
Hogging moment at support (kN·m)	10.22	17.11	0.60	10.19	17.11	0.60
Sagging moment at mid-span (kN·m)	5.11	24.04	0.21	5.09	24.04	0.21
Shear at support face (kN)	61.3	101.7	0.60	61.1	95.3	0.64

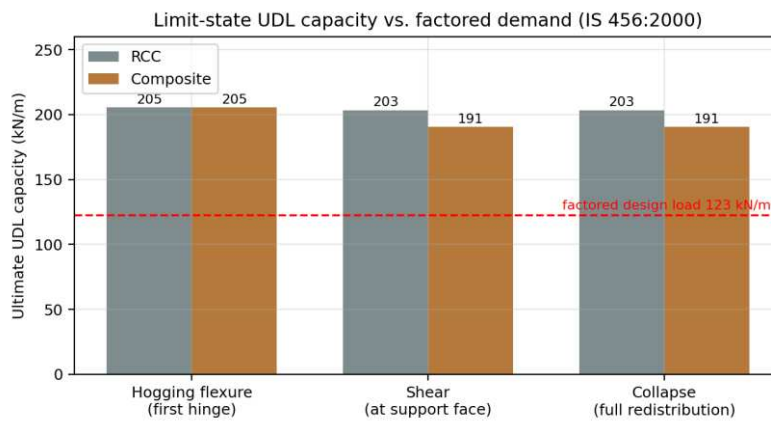


Fig. 5. Ultimate UDL capacity of the beams for different limit states compared with the factored design load.

4.2 Section properties, cracking load and weight

Table 6 shows that the timber core reduces the transformed second moment of area by 2.8 % ($5.475 \times 10^8 \rightarrow 5.323 \times 10^8 \text{ mm}^4$) and the flexural rigidity by the same amount, whereas the shear rigidity κGA falls by 12.5 %, because the core removes concrete that is effective in shear and replaces it with a much more compliant material. The cracking moment and cracking UDL decrease by 2.8 %. The self-weight of a 1 m beam falls from 1.725 to 1.483 kN (−14.0 %).

Table 6. Section properties, cracking load and weight.

Property	RCC beam	Composite beam	Change
Gross I of concrete rectangle (mm^4)	5.175×10^8	5.175×10^8	—
Transformed I (mm^4)	5.475×10^8	5.323×10^8	−2.8 %
Flexural rigidity EI ($\text{N} \cdot \text{mm}^2$)	1.499×10^{13}	1.458×10^{13}	−2.8 %
Shear rigidity κGA (N)	6.56×10^8	5.74×10^8	−12.5 %
Cracking moment M_{cr} (kN·m)	13.99	13.61	−2.8 %
Cracking UDL = $12 M_{cr}/L^2$ (kN/m)	167.9	163.3	−2.8 %
Self-weight of 1 m beam (kN)	1.725	1.483	−14.0 %
Equivalent mass (kg)	175.8	151.1	−14.0 %

4.3 Deflection

The FE mid-span deflection at service load is 28.84 μm for the RCC beam and 32.65 μm for the composite beam, an increase of 13.2 % (Table 10). The analytical Timoshenko solution (Eq. 6) gives 29.76 and 32.30 μm , so the FE results differ by -3.1 % and $+1.1$ %, respectively, which validates the model. The bending part of the analytical deflection is only 14.2–14.6 μm ; the remainder is shear deformation. The 2.5 % increase in the bending part (from the 2.8 % lower EI) is therefore small compared with the 8.5 % increase in the combined analytical deflection, which is driven mainly by the 12.5 % loss of shear rigidity. In absolute terms the deflections are negligible ($L/30,629$ at worst, against the IS 456 limit of $L/250 = 4$ mm). Contours, profiles and the load–deflection response are shown in Figs. 8–10.

Table 10. Mid-span deflection: FE results and analytical check (Timoshenko beam, fixed–fixed, UDL).

Quantity	RCC beam	Composite beam	Change
FE mid-span deflection, mid-height (μm)	28.84	32.65	+13.2 %
FE deflection – top fibre at mid-span (μm)	29.82	33.92	+13.7 %
FE deflection – bottom fibre at mid-span (μm)	27.99	31.52	+12.6 %
Analytical: bending part $wL^4/384EI$ (μm)	14.19	14.56	+2.5 %
Analytical: bending + shear (μm)	29.76	32.30	+8.5 %
FE vs analytical (bending + shear)	-3.1 %	$+1.1$ %	—
Span / deflection ratio (L/δ)	34,678	30,629	—

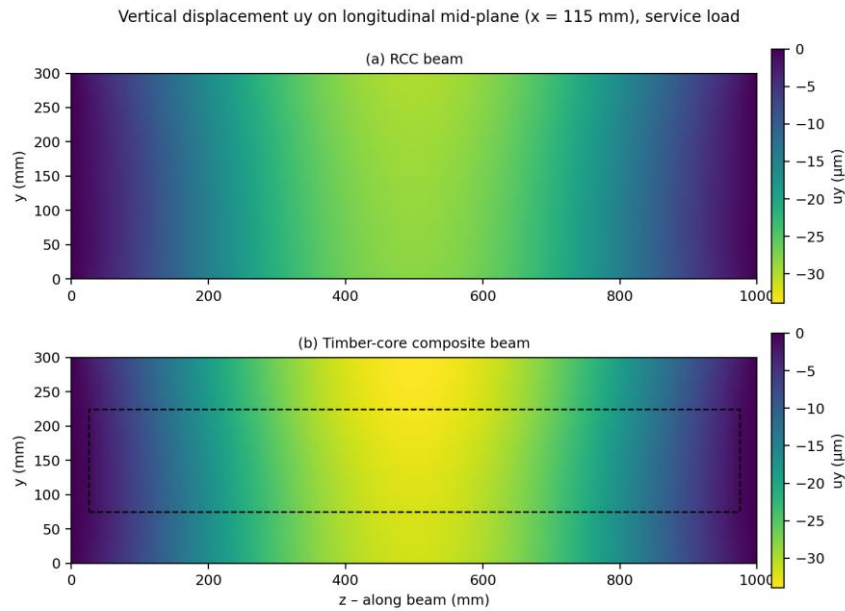


Fig. 8. Vertical displacement contours on the longitudinal mid-plane (μm).

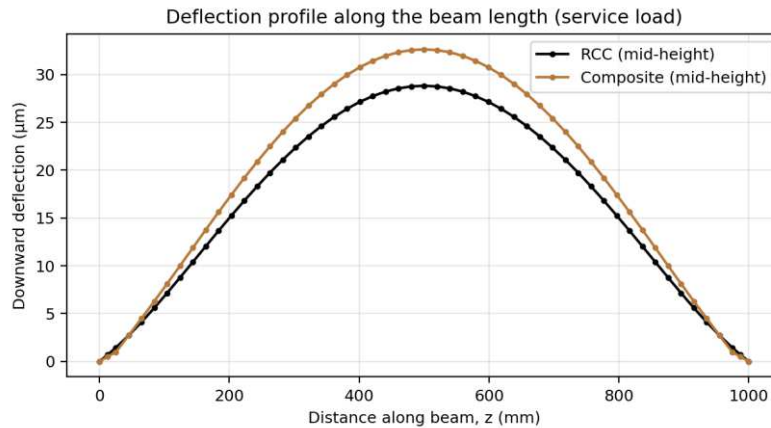


Fig. 9. Deflection profile along the beam length (mid-height line).

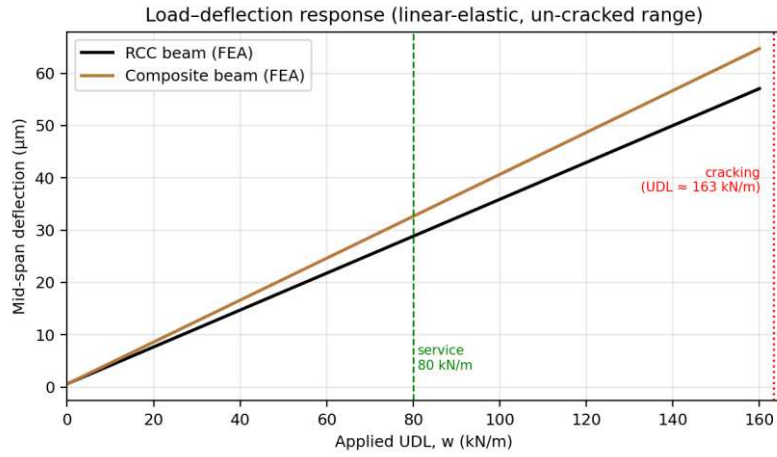


Fig. 10. Load–deflection response of both beams (linear-elastic range, valid up to the cracking load).

4.4 Stresses in concrete and timber

Peak stresses at service load are listed in Table 11. Tension is positive. The peak concrete tensile stress is 2.87 MPa in the RCC beam and 3.10 MPa in the composite beam, both below the flexural tensile strength $f_{cr} = 3.83$ MPa; the beams therefore remain un-cracked at service load and the linear-elastic assumption is justified. The maximum shear stress rises by 17.4 % (0.738 $\rightarrow$ 0.866 MPa). The timber core is lightly stressed (σ_{zz} between -0.240 and $+0.165$ MPa; $\tau_{yz} \leq 0.464$ MPa), because it lies close to the neutral axis. Stress fields are shown in Figs. 11–15.

Table 11. Peak stresses from FE analysis at service load (MPa).

Stress quantity	RCC beam	Composite beam	Change
Concrete σ_{zz} – maximum tension	2.87	3.10	+8.3 %
Concrete σ_{zz} – maximum compression	–2.59	–2.80	+7.9 %
Concrete flexural tensile strength f_{cr}	3.83	3.83	—
Maximum shear stress $ \tau_{yz} $ (overall)	0.738	0.866	+17.4 %
Maximum von Mises stress	2.83	3.04	+7.5 %
Timber σ_{zz} – maximum tension / compression	—	0.165 / –0.240	—
Timber $ \tau_{yz} $ – maximum	—	0.464	—

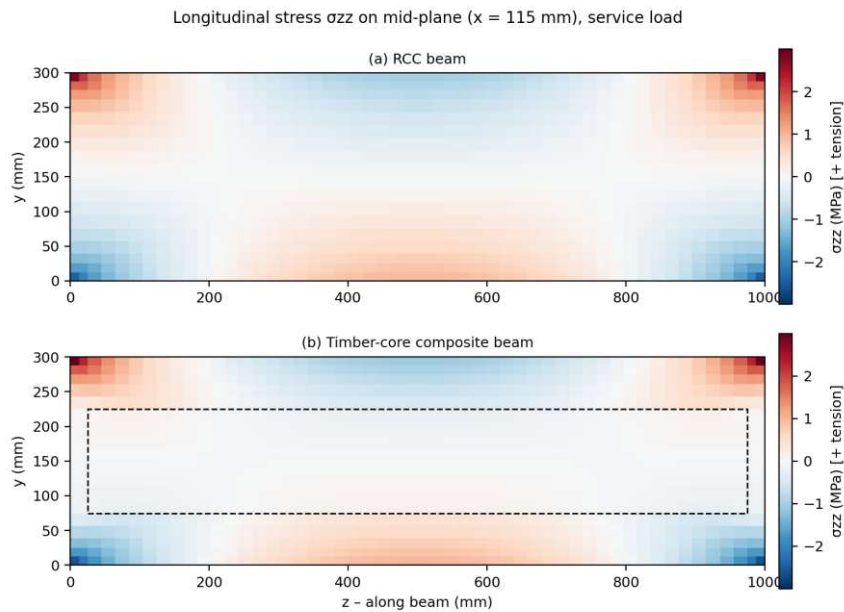


Fig. 11. Longitudinal stress σ_{zz} on the mid-plane (MPa). Dashed box = timber core.

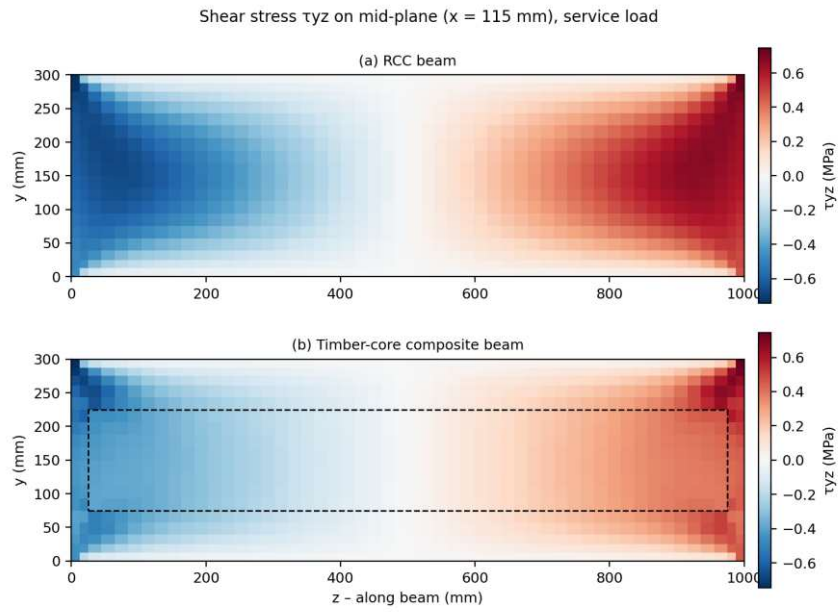


Fig. 12. Shear stress τ_{yz} on the longitudinal mid-plane (MPa).

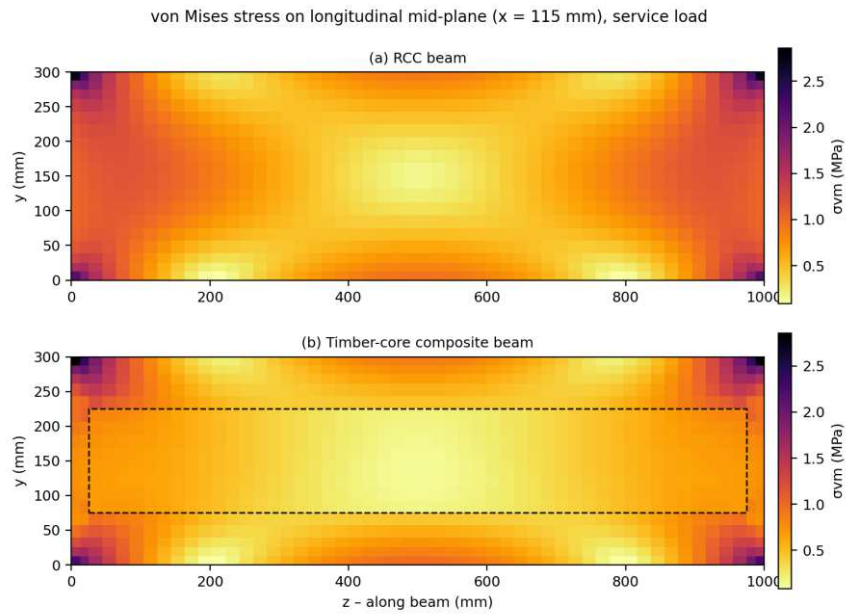


Fig. 13. von Mises stress on the longitudinal mid-plane (MPa).

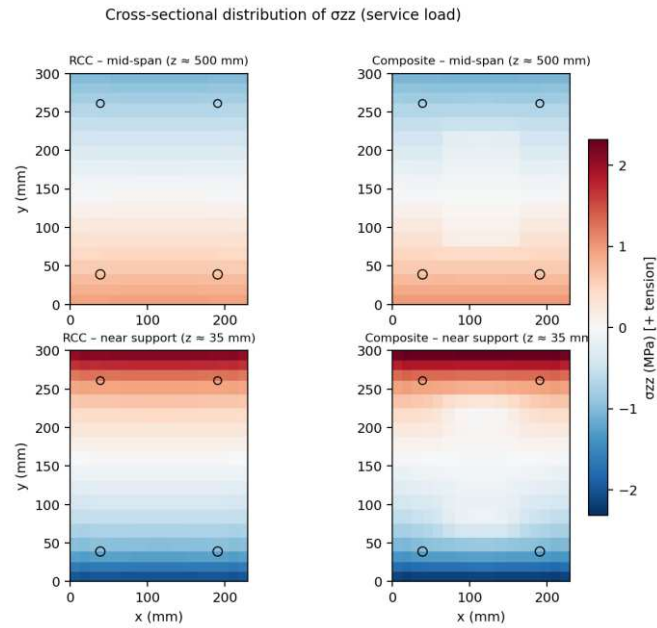


Fig. 14. σ_{zz} over the cross-section at mid-span (top row) and near the support (bottom row).

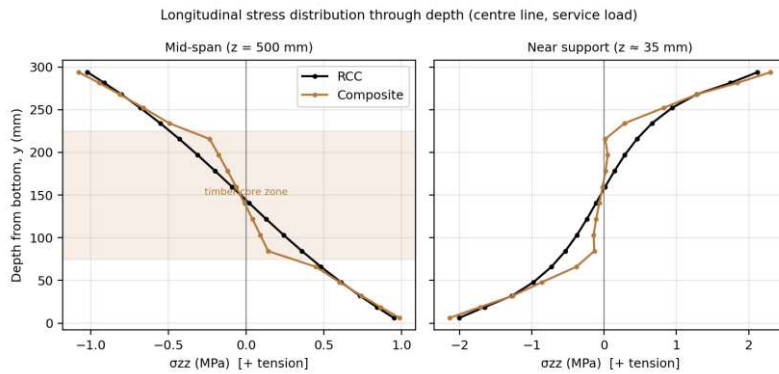


Fig. 15. Variation of σ_{zz} through the depth at mid-span and near the support.

4.5 Reinforcement stresses

The axial stresses in the bars are very low (Table 12): at most 7.42 MPa at the supports, i.e. less than 2 % of the design yield stress $0.87f_y = 435$ MPa. This is expected for an un-cracked section, in which the concrete carries most of the service-level bending; the steel becomes the principal tension carrier only after cracking, which is outside the scope of the linear analysis. The timber core increases bar stresses by 2.9–4.0 %.

Table 12. Axial stress in reinforcement from FE analysis at service load (MPa).

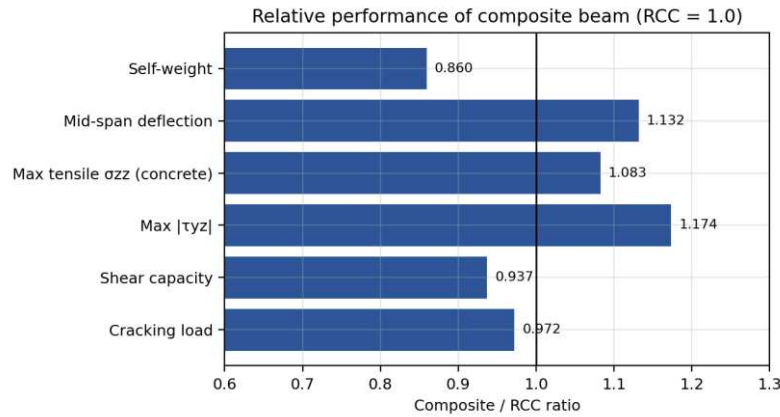
Location	RCC beam	Composite beam	Change
Bottom bars (12 ϕ) at mid-span	4.78	4.92	+2.9 %
Top bars (10 ϕ) at mid-span	-4.84	-4.98	+2.9 %
Top bars (10 ϕ) at support	7.14	7.42	+4.0 %
Bottom bars (12 ϕ) at support	-7.12	-7.39	+3.7 %
Design yield stress $0.87f_y$	435	435	—

4.6 Overall comparison

Table 13 and Fig. 16 summarise the comparison. The timber core removes 20.7 % of the concrete volume and 14.0 % of the self-weight, leaves the flexural capacity unchanged, and costs 6.3 % of the shear capacity, 2.8 % of the cracking load and 13.2 % in additional deflection.

Table 13. Overall comparison of the RCC and timber-core composite beams.

Parameter	RCC beam	Composite beam	Change
Self-weight of 1 m beam (kN)	1.725	1.483	−14.0 %
Mid-span deflection at service load (μm)	28.8	32.6	+13.2 %
Max. concrete tensile stress (MPa)	2.87	3.10	+8.3 %
Flexural capacity M_u sagging / hogging (kN·m)	24.0 / 17.1	24.0 / 17.1	0.0 %
Shear capacity (kN)	101.7	95.3	−6.3 %
Ultimate UDL capacity – shear governed (kN/m)	203	191	−6.3 %
Cracking UDL (kN/m)	168	163	−2.8 %
Concrete volume (m^3)	0.0690	0.0548	−20.7 %

**Fig. 16.** Composite-to-RCC ratio of key response quantities (RCC = 1.0).

5. Discussion

5.1 Why flexural capacity is unaffected

The flexural capacity of an under-reinforced section is controlled by the tension steel and the depth of the compression block [13,21]. In both beams the compression block is 39.6 mm deep (sagging) and the bars lie at the outer faces, whereas the timber core occupies the central 150 mm and starts 75 mm from the extreme fibres. The core therefore interferes with neither the compression resultant nor the tension steel, so M_u is identical (24.04 / 17.11 kN·m). This is consistent with the stress distributions in Figs. 14 and 15, where the core is subjected to small stresses close to the neutral axis.

5.2 Why deflection increases more than flexural rigidity decreases

With $L/D = 3.3$ the beam is a deep member and shear deformation is of the same order as flexural deformation (Table 10). The timber core, with a modulus only 46 % of that of concrete, lowers EI by 2.8 % but lowers κGA by 12.5 %, so the shear part of Eq. (6) grows much faster than the bending part. This explains why the combined deflection rises by 13.2 % in the FE model although EI changes by under 3 %, and shows that, for short deep beams, a purely Euler–Bernoulli estimate would understate the effect of the core. The FE and analytical values agree to within 3.1 %, which supports the use of Eq. (6) as a quick design check.

5.3 Stress levels and the role of the support singularity

Peak FE stresses occur at the fixed end faces, where the full restraint of all translations creates a geometric singularity at the restrained edges [17,29]. As a rough cross-check, the elementary-beam hogging stress at the support at service load can be estimated as $\sigma = My/I_{tr}$ with $M = wL^2/12 = 6.81$ kN·m, $y = 150$ mm and $I_{tr} = 5.475 \times 10^8$ mm⁴, which gives about 1.87 MPa for the RCC beam (about 1.91 MPa for the composite beam), compared with 2.87 and 3.10 MPa from the FE analysis. The FE peaks are therefore higher than the beam-theory values, as expected at a fully restrained edge, and they are conservative. The ratio of FE peak stress to f_{cr} (0.75 and 0.81) and the beam-theory ratio (about 0.49 and 0.50) both indicate un-cracked behaviour at service load. The 8 % increase in concrete stress in the composite beam reflects the lower stiffness of the core and the redistribution of load to the outer concrete shells.

5.4 Weight–stiffness–capacity trade-off

To express the trade-off compactly, efficiency indices were defined as the ratio of a stiffness or capacity quantity to the self-weight of the 1 m beam, normalised by the RCC value (Table 14, Fig. 17). Although each of EI , M_u and V_u is equal to or lower in the composite beam in absolute terms, all three improve per unit weight: EI/W by 13.1 %, M_u/W by 16.3 % and V_u/W by 9.0 %. In other words, the composite beam gives up a small amount of shear capacity and stiffness but

sheds proportionally more weight. In a building frame, the 14.0 % reduction of beam self-weight reduces the dead load transferred to columns and foundations, while the reduction of 20.7 % in concrete volume proportionally lowers cement demand, in line with the sustainability motivation discussed in the Introduction [1,3,4]. A quantitative life-cycle assessment would be needed to confirm cost and carbon benefits.

Table 14. Efficiency indices per unit self-weight (derived from Tables 4–6; RCC = 1.000).

Index	RCC beam	Composite beam	Change
Flexural rigidity per unit weight, EI/W ($N \cdot mm^2/kN$)	8.69×10^{12}	9.83×10^{12}	+13.1 %
Sagging moment capacity per unit weight, M_u/W ($kN \cdot m/kN$)	13.94	16.21	+16.3 %
Shear capacity per unit weight, V_u/W (kN/kN)	58.9	64.2	+9.0 %

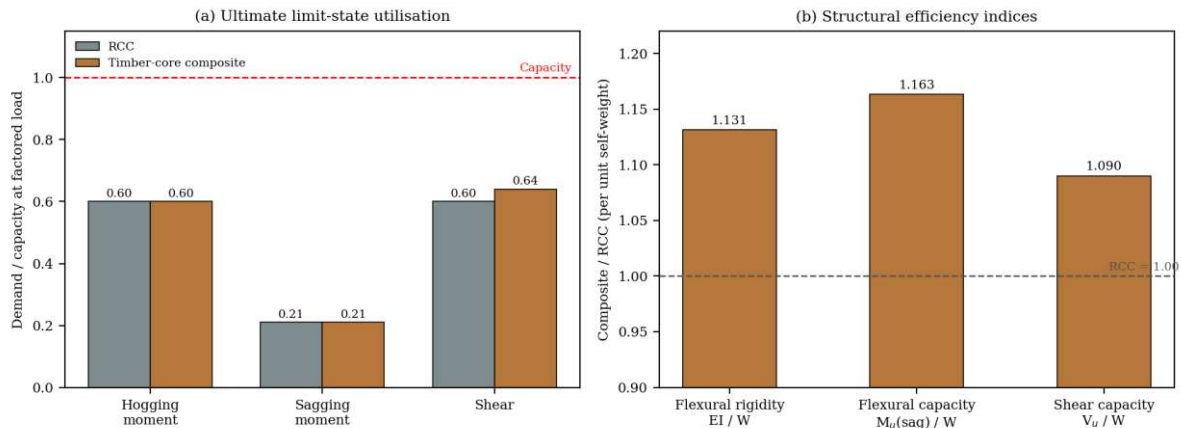


Fig. 17. (a) Demand-to-capacity ratios at the factored load, and (b) structural efficiency indices of the composite beam relative to the RCC beam.

5.5 Practical implications and limitations

The results indicate that a 100×150 mm Sal-timber core is a viable means of reducing dead load and concrete consumption in lightly stressed beams, but it is not structurally efficient in the sense of contributing to strength, because its stresses are very low. The conclusions are subject to the following limitations. (i) Loads and timber properties were assumed and results scale linearly with load only within the un-cracked range. (ii) The analysis is linear-elastic; cracking, crushing, steel yielding, concrete–timber interface slip, and the creep, shrinkage and moisture effects of timber are not modelled [10,20]. (iii) Timber was treated as isotropic, whereas real timber is orthotropic with the grain along the beam axis [25]. (iv) Because $L/D = 3.3$, the member behaves as a deep beam near the supports, where strut-and-tie action may govern and the simplified IS 456 shear check used here may not capture the actual failure mechanism [31]. (v) Mesh convergence was assessed with three meshes using the change in mid-span deflection; the successive changes (0.71 % $\rightarrow$ 0.52 %) are below 1 %, but a formal error estimate such as Richardson extrapolation was not performed, and peak stresses at the fixed faces remain mesh-dependent. (vi) No experimental validation was carried out.

6. Conclusions

A conventional RCC beam and a timber-core composite RCC beam ($230 \times 300 \times 1000$ mm, M30/Fe500, fixed–fixed) were designed to IS 456:2000 and analysed with a verified three-dimensional FE model. The main conclusions are:

- Both beams satisfy IS 456:2000 with comfortable margins: at the factored load of about 123 kN/m the utilisation is 0.60 in hogging flexure and 0.60 (RCC) / 0.64 (composite) in shear. The ultimate UDL capacity is governed by shear (203 kN/m for RCC, 191 kN/m for the composite beam).
- The 100×150 mm Sal-timber core reduces self-weight by 14.0 % and concrete volume by 20.7 % without changing the flexural capacity (24.04 / 17.11 kN·m), because the core lies in the neutral-axis zone, away from the compression block and the reinforcement.
- The composite beam is slightly more flexible: mid-span deflection increases by 13.2 % (28.8 $\rightarrow$ 32.6 μm), mainly through the 12.5 % loss of shear rigidity; the cracking load decreases by 2.8 % and the conservative shear capacity by 6.3 %.
- The FE model satisfies global equilibrium (error < 0.001 %), is converged to better than 1 % between the two finest meshes, and reproduces the analytical Timoshenko-beam deflection to within 3.1 %.

5. Peak concrete tensile stresses (2.87 and 3.10 MPa) remain below the flexural tensile strength (3.83 MPa), so the beams are un-cracked at service load; reinforcement stresses are below 2 % of yield, and the timber stresses are very low (≤ 0.24 MPa axial).
6. Per unit self-weight, flexural rigidity, flexural capacity and shear capacity of the composite beam are 13.1 %, 16.3 % and 9.0 % higher than those of the RCC beam, indicating that a neutral-axis timber core is a viable way of lightening lightly stressed beams.

Future work. Nonlinear FE analysis with a damaged-plasticity model, orthotropic timber and interface slip; experimental testing of full-scale specimens; parametric study of core size, grade and length; and a life-cycle assessment of cost and embodied carbon.

Declarations

Credit authorship contribution statement

¹Nilesh B. Gawali: Conceptualization, Methodology, Software, Formal analysis, Investigation, Visualization, Writing – original draft. ²Sunil B. Thakare: Validation, Supervision, Writing – review & editing. ³Dattatray P. Kamble: Supervision, Validation, Writing – review & editing.

Declaration of competing interest

I declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

The data supporting the findings of this study, including the numerical design calculations and finite element analysis results, are available from the corresponding author upon reasonable request.

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References

- [1] Monteiro, P.J.M., Miller, S.A., Horvath, A. (2017). Towards sustainable concrete. *Nature Materials*, 16(7), 698–699.
- [2] Mehta, P.K., Monteiro, P.J.M. (2014). *Concrete: Microstructure, Properties, and Materials*, 4th ed. McGraw-Hill, New York.
- [3] Hammond, G.P., Jones, C.I. (2008). Embodied energy and carbon in construction materials. *Proceedings of the Institution of Civil Engineers – Energy*, 161(2), 87–98.
- [4] Churkina, G., Organschi, A., Reyer, C.P.O., et al. (2020). Buildings as a global carbon sink. *Nature Sustainability*, 3(4), 269–276.
- [5] Ceccotti, A. (2002). Composite concrete-timber structures. *Progress in Structural Engineering and Materials*, 4(3), 264–275.
- [6] Yeoh, D., Fragiocomo, M., De Franceschi, M., Heng Boon, K. (2011). State of the art on timber-concrete composite structures: literature review. *Journal of Structural Engineering*, 137(10), 1085–1095.
- [7] Dias, A.M.P.G., Skinner, J., Crews, K., Tannert, T. (2016). Timber-concrete-composites increasing the use of timber in construction. *European Journal of Wood and Wood Products*, 74(3), 443–451.
- [8] Gutkowski, R., Brown, K., Shigidi, A., Natterer, J. (2008). Laboratory tests of composite wood–concrete beam and floor specimens. *Construction and Building Materials*, 22(6), 1059–1066.
- [9] Lukaszewska, E., Johnsson, H., Fragiocomo, M. (2008). Performance of connections for prefabricated timber–concrete composite floors. *Materials and Structures*, 41(9), 1533–1550.
- [10] Fragiocomo, M., Ceccotti, A. (2006). Long-term behavior of timber–concrete composite beams. I: Finite element modeling and validation. *Journal of Structural Engineering*, 132(1), 13–22.
- [11] Möhler, K. (1956). *Über das Tragverhalten von Biegeträgern und Druckstäben mit zusammengesetzten Querschnitten und nachgiebigen Verbindungsmitteln*. Habilitation thesis, Technische Hochschule Karlsruhe.
- [12] European Committee for Standardization (2004). *EN 1995-1-1:2004+A2:2014 Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings*. CEN, Brussels.
- [13] Park, R., Paulay, T. (1975). *Reinforced Concrete Structures*. Wiley, New York.
- [14] Ngo, D., Scordelis, A.C. (1967). Finite element analysis of reinforced concrete beams. *ACI Journal Proceedings*, 64(3), 152–163.

- [15] Zienkiewicz, O.C., Taylor, R.L. (2000). *The Finite Element Method, Vol. 1: The Basis*, 5th ed. Butterworth-Heinemann, Oxford.
- [16] Bathe, K.-J. (1996). *Finite Element Procedures*. Prentice Hall, Englewood Cliffs, NJ.
- [17] Cook, R.D., Malkus, D.S., Plesha, M.E., Witt, R.J. (2002). *Concepts and Applications of Finite Element Analysis*, 4th ed. Wiley, New York.
- [18] Hillerborg, A., Modéer, M., Petersson, P.-E. (1976). Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements. *Cement and Concrete Research*, 6(6), 773–781.
- [19] Bažant, Z.P., Oh, B.H. (1983). Crack band theory for fracture of concrete. *Materials and Structures*, 16(3), 155–177.
- [20] Lee, J., Fenves, G.L. (1998). Plastic-damage model for cyclic loading of concrete structures. *Journal of Engineering Mechanics*, 124(8), 892–900.
- [21] Bureau of Indian Standards (2000). *IS 456:2000 Plain and Reinforced Concrete – Code of Practice*. BIS, New Delhi.
- [22] Bureau of Indian Standards (1983). *SP 24:1983 Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete*. BIS, New Delhi.
- [23] Rodrigues, J.N., Dias, A.M.P.G., Providência, P. (2013). Timber-concrete composite bridges: state-of-the-art review. *BioResources*, 8(4), 6630–6649.
- [24] Bureau of Indian Standards (1994). *IS 883:1994 Design of Structural Timber in Building – Code of Practice*. BIS, New Delhi.
- [25] Forest Products Laboratory (2010). *Wood Handbook: Wood as an Engineering Material*. General Technical Report FPL-GTR-190. USDA Forest Service, Madison, WI.
- [26] European Committee for Standardization (2004). *EN 1992-1-1:2004 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings*. CEN, Brussels.
- [27] American Concrete Institute (2019). *ACI 318-19 Building Code Requirements for Structural Concrete and Commentary*. ACI, Farmington Hills.
- [28] Timoshenko, S.P. (1921). On the correction for shear of the differential equation for transverse vibrations of prismatic bars. *Philosophical Magazine*, Series 6, 41(245), 744–746.
- [29] Timoshenko, S.P., Goodier, J.N. (1970). *Theory of Elasticity*, 3rd ed. McGraw-Hill, New York.
- [30] Roache, P.J. (1994). Perspective: a method for uniform reporting of grid refinement studies. *Journal of Fluids Engineering*, 116(3), 405–413.
- [31] Schlaich, J., Schäfer, K., Jennewein, M. (1987). Toward a consistent design of structural concrete. *PCI Journal*, 32(3), 74–150.