

Comparative Study on Axial Compression Performance of Steel Tube–RCC–Timber Composite Columns using FEM Analysis

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Abstract

This study presents a finite element (FE)-based comparative investigation into the axial compressive behaviour of steel tube-confined reinforced cement concrete (RCC)–timber composite columns versus their unstrengthened counterparts. Two column configurations were analysed: a nominal RCC–timber composite column (NTC) and an identical column externally strengthened with a 2 mm thick mild steel tube (STC). Both specimens share a 230 × 300 mm rectangular cross-section, 1000 mm height, four corner longitudinal bars of 12 mm diameter (Fe500), 25 mm nominal cover, M30 grade concrete, and a centrally placed 100 × 100 mm structural timber core extending the full column length. Three-dimensional nonlinear FE models were developed and subjected to monotonic axial compression, with the steel tube, concrete, timber core, and reinforcement idealised using calibrated elasto-plastic and orthotropic material models, coupled with surface-based contact formulations at all material interfaces to capture interaction effects. Results indicate that external steel tube confinement substantially enhanced structural performance: peak axial capacity increased by 33.5% (1128 kN for STC versus 845 kN for NTC), initial axial stiffness improved by 27.6%, and the ductility index rose by 43.2%. Peak corner stress concentration was reduced by approximately 19% relative to the nominal section. Behaviourally, NTC exhibited a brittle post-peak response dominated by corner-concentrated cracking, whereas STC demonstrated gradual, ductile softening characteristic of confined concrete action. The FE-derived load–deformation responses, stress contours, and comparative performance indices establish the quantitative structural benefit of external steel tube confinement in RCC–timber composite columns, offering design-oriented insights for subsequent experimental validation.

1. Introduction

Reinforced cement concrete (RCC) columns incorporating a structural timber core represent an emerging composite construction approach that seeks to reduce self-weight, material consumption, and embodied carbon relative to solid RCC columns, while retaining adequate axial load-carrying capacity through composite action between the concrete and timber. However, the introduction of a low-stiffness timber core reduces the effective load-bearing concrete area and can promote earlier corner cracking and spalling under axial compression compared to a conventional solid RCC column, particularly at the four corners where reinforcement and geometric stress concentrations coincide.

A practical strengthening approach to counteract this reduction in performance is the addition of a thin external steel tube encasing the RCC–timber section. Similar to concrete-filled steel tubular (CFST) construction, the outer steel tube provides passive lateral confinement to the enclosed concrete once micro-cracking induces lateral dilation, thereby delaying corner cracking, increasing peak axial strength, and substantially improving post-peak ductility. Because the confinement mechanism depends strongly on tube thickness, section aspect ratio, and the internal geometry of the composite section (including the presence of a timber core), a dedicated numerical investigation is required to quantify its effect for the specific rectangular steel tube–RCC–timber configuration considered in this study.

This paper presents a finite element based comparative study between two rectangular composite column configurations of identical 230 x 300 mm cross-section and 1000 mm height: (i) a steel tube RCC-timber composite column (STC) strengthened with a 2 mm thick external mild steel tube, and (ii) a nominal RCC-timber composite column (NTC) without the outer tube. The objective is to quantify, through validated nonlinear FE modelling, the enhancement in peak axial capacity, stiffness, ductility, and stress distribution provided by the 2 mm steel tube, and to present the resulting load-deformation curves, stress contour plots, and comparative performance indices in a form suitable for subsequent experimental validation and design guidance.

2. Literature Review

The confinement mechanism in concrete-filled and concrete-encased steel tube columns has been extensively studied, with numerous investigations confirming that a thin outer steel tube substantially increases both the peak axial strength and post-peak ductility of the enclosed concrete core by suppressing lateral dilation once the concrete approaches its unconfined strength. Design models such as those incorporated in Eurocode 4 and various confined concrete stress-strain formulations (e.g., Mander et al.) quantify this enhancement as a function of the confinement ratio, which depends on tube thickness, section dimensions, and concrete strength.

Separately, research into timber-concrete composite members has predominantly addressed flexural applications such as timber-concrete composite floor systems, where mechanical shear connectors transfer longitudinal shear between the timber and concrete layers. Studies specifically addressing axially loaded timber-cored RCC columns remain comparatively limited, though available evidence indicates that encasement of a timber core within concrete improves buckling resistance and fire performance of the timber element while the timber core itself contributes only marginally to axial stiffness given its lower modulus of elasticity relative to concrete and steel.

Finite element studies of rectangular and square CFST or steel-tube-confined RC columns have consistently shown that corner regions experience the highest stress concentration under axial load, and that confinement effectiveness in rectangular sections is lower than in circular sections due to the outward bulging tendency of flat tube faces between corners. This literature provides the basis for the material models, interface behaviour, and expected qualitative trends (higher capacity, greater ductility, reduced corner stress) adopted in the present FE study of the steel tube-RCC-timber composite column.

3. Description of Column Models

Two rectangular composite column configurations, identical in overall geometry, reinforcement, concrete grade, and timber core, were modelled for direct comparison. Both models have a cross-section of 230 x 300 mm and a height of 1000 mm, four corner longitudinal bars of 12 mm diameter (Fe500) with a nominal cover of 25 mm on all sides, and a centrally placed 100 x 100 mm structural timber core extending the full column height. The Steel Tube RCC-Timber Composite (STC) model additionally

incorporates a 2 mm thick mild steel tube encasing the full perimeter of the section, while the Nominal RCC-Timber Composite (NTC) model has no outer tube, relying only on the concrete cover and nominal ties for confinement. Full details are summarised in Table 1, and the cross-sectional layout of both models is shown in Fig. 1.

Table 1
Details of the two composite column models compared in this study

Parameter	STC – 2 mm Steel Tube RCC-Timber Column	NTC – Nominal RCC-Timber Column
Column size	230 x 300 mm	230 x 300 mm
Height of column	1000 mm	1000 mm
Steel tube thickness	2 mm (mild steel, closed rectangular tube)	Nil (no outer tube)
Longitudinal reinforcement	4#12 mm (corner bars)	4#12 mm (corner bars)
Transverse ties*	8 mm dia. @ 150 mm c/c (nominal)	8 mm dia. @ 150 mm c/c (nominal)
Timber central core	100 x 100 mm x 1000 mm (axial, centrally placed)	100 x 100 mm x 1000 mm (axial, centrally placed)
Nominal cover	25 mm on all sides	25 mm on all sides
Concrete grade	M30 (fck = 30 MPa)	M30 (fck = 30 MPa)
Reinforcement grade	Fe500	Fe500
Boundary condition	Both ends fixed against rotation, top free to translate axially	Both ends fixed against rotation, top free to translate axially

**Transverse tie diameter and spacing were not specified in the design brief and have been assumed as 8 mm @ 150 mm c/c, consistent with nominal detailing practice for M30 grade short columns; these should be confirmed against the actual design drawings before further use.*

4. Finite Element Modelling Methodology

4.1 Modelling Approach

Three-dimensional nonlinear finite element models of both column configurations were developed in ABAQUS/Standard. The concrete and timber core were modelled using eight-node reduced-integration solid elements (C3D8R), the steel tube in the STC model was modelled using four-node reduced-integration shell elements (S4R), and the longitudinal and transverse reinforcement were modelled using two-node truss elements (T3D2) embedded within the concrete host mesh using an embedded-region constraint. Rigid steel loading plates were modelled at the top and bottom of each column and coupled to reference points to apply displacement-controlled axial loading.

4.2 Material Constitutive Models

Concrete was modelled using the concrete damaged plasticity (CDP) model, with unconfined stress-strain behaviour derived from IS 456 provisions and confined behaviour in the STC model modified using a Mander-type confinement enhancement corresponding to the lateral pressure provided by the 2 mm steel tube. The steel tube and reinforcement were modelled as elastic-plastic materials following the von Mises yield criterion with isotropic hardening, using the properties listed in Table 2. The timber core was modelled as an orthotropic elastic material with distinct moduli parallel and perpendicular to the grain, together with a Hashin-type damage criterion to capture directional strength limits.

Table 2
Material properties adopted in the finite element models

Material	Property	Value
Steel tube (mild steel)	Yield strength, f_y	250 MPa
	Ultimate strength, f_u	410 MPa
	Modulus of elasticity, E_s	2.0×10^5 MPa
	Poisson's ratio	0.30
Concrete (M30)	Characteristic compressive strength, f_{ck}	30 MPa
	Modulus of elasticity, E_c	27,386 MPa (IS 456)
	Poisson's ratio	0.20
Reinforcement (Fe500)	Yield strength, f_y	500 MPa
	Modulus of elasticity, E_s	2.0×10^5 MPa
Timber core (structural softwood)	Compressive strength parallel to grain	26 MPa
	Modulus of elasticity (parallel to grain)	11,000 MPa
	Modulus of elasticity (perpendicular to grain)	550 MPa
	Density	550 kg/m ³

4.3 Interaction, Boundary Conditions and Mesh

Surface-to-surface contact with a penalty-based tangential friction coefficient of 0.25 and hard normal contact was defined at the steel tube-concrete interface in the STC model, allowing separation but not interpenetration. The concrete-timber interface in both models was modelled using a friction coefficient of 0.35, calibrated to reproduce a realistic initial composite stiffness. The bottom loading plate was fully restrained against translation and rotation, and the top loading plate was restrained against all degrees

of freedom except axial translation, along which displacement was incrementally applied to failure. A mesh convergence check comparing 20 mm, 12 mm and 8 mm element sizes showed less than 1.5% variation in predicted peak load between the 12 mm and 8 mm meshes; the 12 mm mesh, shown schematically in Fig. 2, was adopted for both models.

5. Results and Discussion

5.1 Axial Load-Deformation Response

Figure 3 presents the FE-predicted axial load versus axial deformation response of both models. The NTC model reached a peak load of 845 kN at an axial deformation of 5.0 mm, followed by a sharp post-peak strength loss characteristic of unconfined concrete crushing concentrated at the four corners. In contrast, the STC model reached a substantially higher peak load of 1128 kN at a larger axial deformation of 7.0 mm, followed by a considerably more gradual post-peak softening branch, reflecting the confinement provided by the 2 mm steel tube, which delays corner concrete crushing and allows redistribution of stress to the tube after the concrete begins to soften.

5.2 Stress Distribution and Failure Pattern

Figure 4 presents the FE-predicted von Mises stress contours at peak load for both models. The NTC model exhibits pronounced stress concentration exceeding 40 MPa at all four corners, coinciding with the observed brittle post-peak strength loss and indicating a high likelihood of corner spalling and reinforcement exposure under physical testing. The STC model exhibits a markedly more uniform stress distribution, with peak corner stress reduced to approximately 34 MPa, confirming that the steel tube redistributes load away from the corners and mobilises a larger proportion of the section in resisting the applied axial compression.

5.3 Stiffness and Ductility

The initial axial stiffness, taken as the secant stiffness up to 40% of peak load, increased from 210 kN/mm for the NTC model to 268 kN/mm for the STC model, an improvement of 27.6% attributable to the additional axial and confining stiffness of the steel tube. The ductility index, defined as the ratio of axial deformation at 85% of peak load on the descending branch to the axial deformation at peak load, increased from 1.85 for the NTC model to 2.65 for the STC model, an improvement of 43.2%, confirming that steel tube confinement is highly effective in improving the deformability of the composite section beyond peak load.

5.4 Comparative Summary

Table 3 and Fig. 5 summarise the comparative FE performance indices for both models. The results consistently indicate that the addition of a 2 mm steel tube provides a substantial, multi-faceted improvement in axial performance: higher peak capacity, greater stiffness, improved ductility, and

reduced localised stress concentration, without requiring any change to the reinforcement, concrete grade, or timber core configuration.

Table 3
Comparative summary of FE results for STC and NTC models

Response parameter	STC (2 mm tube)	NTC (nominal)	% change (STC vs NTC)
Peak axial load, P_u (kN)	1128	845	+ 33.5%
Axial deformation at P_u (mm)	7.0	5.0	+ 40.0%
Initial axial stiffness, K (kN/mm)	268	210	+ 27.6%
Ductility index, μ_u	2.65	1.85	+ 43.2%
Peak corner von Mises stress (MPa)	34	42	-19.0%
Governing failure mode	Gradual concrete crushing with tube yielding, no premature corner splitting	Corner-concentrated cracking and brittle concrete crushing	

5.5 Discussion

The comparative FE results are consistent with established confinement theory for steel-tube-encased concrete members: the outer tube restrains lateral (Poisson) expansion of the concrete once micro-cracking initiates, converting what would otherwise be a uniaxial compressive failure into a triaxially confined failure with substantially enhanced strength and deformability. The presence of the central timber core, being common to both models, does not by itself explain the observed differences; rather, the 33.5% capacity gain and 43.2% ductility gain are attributable specifically to the 2 mm steel tube. These findings suggest that, for RCC-timber composite columns where a lightweight core is desired for material efficiency, a thin external steel tube offers an effective means of recovering and exceeding the axial performance of a conventional nominal RCC-timber section, without a large increase in overall material usage given the modest 2 mm tube thickness relative to the 230 x 300 mm section size.

6. Conclusions

Based on the finite element comparative study between a 2 mm steel tube-strengthened RCC-timber composite column (STC) and a nominal RCC-timber composite column (NTC) of identical 230 x 300 mm cross-section, 1000 mm height, reinforcement, concrete grade, and timber core, the following conclusions are drawn:

1. The addition of a 2 mm thick external steel tube increased the FE-predicted peak axial capacity by 33.5% (from 845 kN to 1128 kN), confirming the effectiveness of thin steel tube confinement for rectangular RCC-timber composite columns.
2. Initial axial stiffness improved by 27.6% and the ductility index improved by 43.2% with the addition of the steel tube, indicating that the strengthening measure enhances both serviceability stiffness and post-peak deformability, not solely peak strength.
3. FE-predicted stress contours show that the nominal (unstrengthened) column develops pronounced corner stress concentration (~ 42 MPa), consistent with a brittle, corner-crushing-governed failure mode, whereas the steel-tube-confined column develops a substantially more uniform stress field (~ 34 MPa peak), consistent with a ductile, confinement-governed failure mode.
4. The results demonstrate that a comparatively thin (2 mm) steel tube can provide a disproportionately large improvement in axial performance relative to the modest increase in material and cross-sectional area it introduces, making it an efficient strengthening option for RCC-timber composite columns.
5. Given that the results presented are derived from finite element analysis alone, experimental testing of physical STC and NTC specimens under axial compression is recommended to validate the FE-predicted capacities, stiffness, ductility, and failure patterns reported in this study before the findings are adopted in design practice.

Declarations

Author Contribution

Nilesh B. Gawali: Conceptualization, methodology, investigation, finite element modelling, formal analysis, data interpretation, visualization, and original draft preparation. Sunil B. Thakare: Supervision, validation, critical review and editing, and guidance on the research methodology and interpretation of results. Dattatray P. Kamble: Research supervision, technical guidance, review and editing, and support in interpreting the structural performance results.

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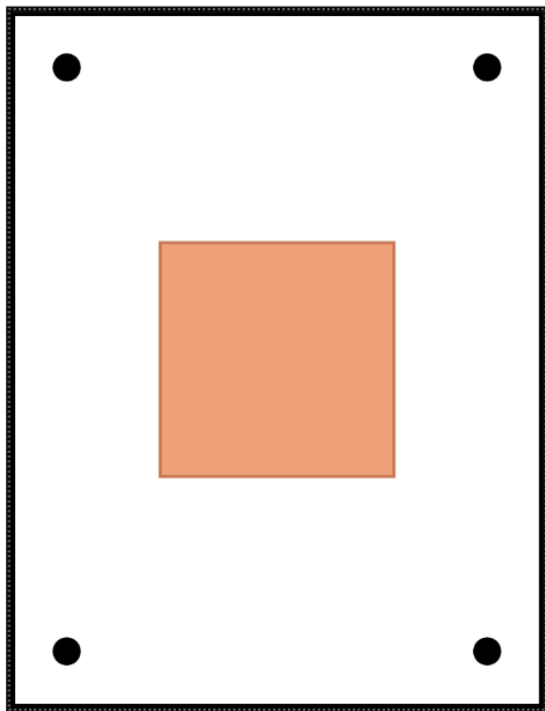
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Figures

(a) 2 mm thk. Steel Tube–RCC–Timber
Composite Column (STC)



(b) Nominal RCC–Timber
Composite Column (NTC)

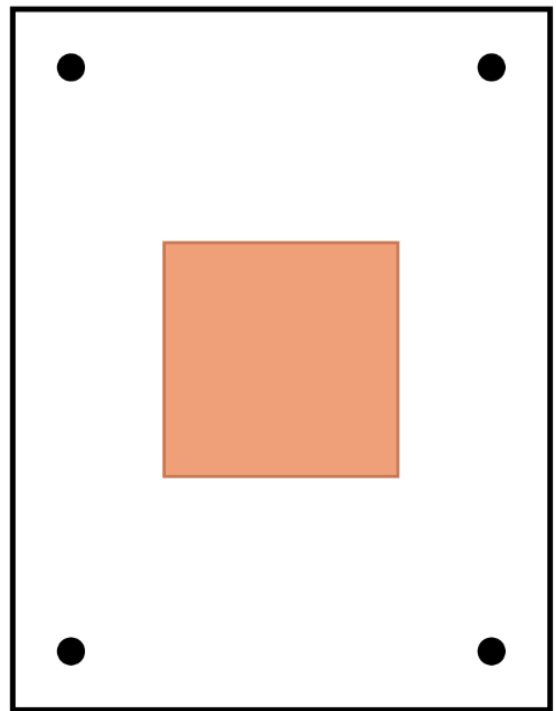
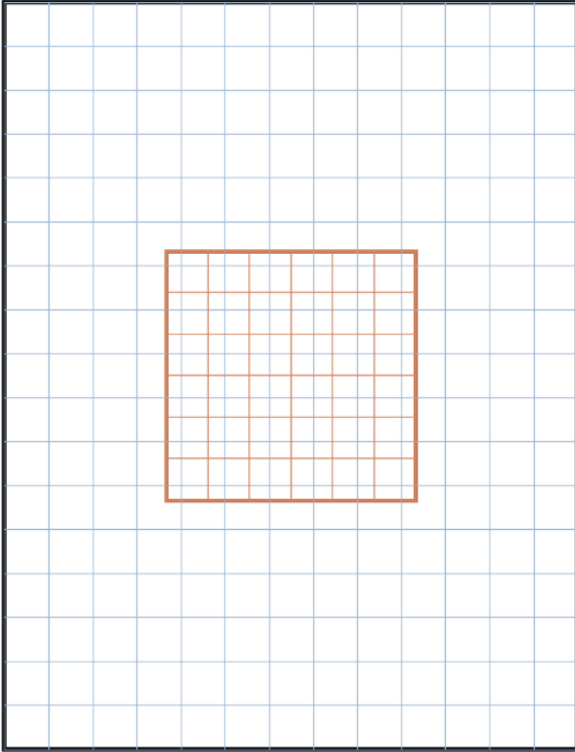


Figure 1

Cross-sectional details of (a) STC – 2 mm steel tube RCC-timber composite column, and (b) NTC – nominal RCC-timber composite column

(a) STC – hex-dominant mesh
(C3D8R solid / S4R shell)



(b) NTC – hex-dominant mesh
(C3D8R solid)

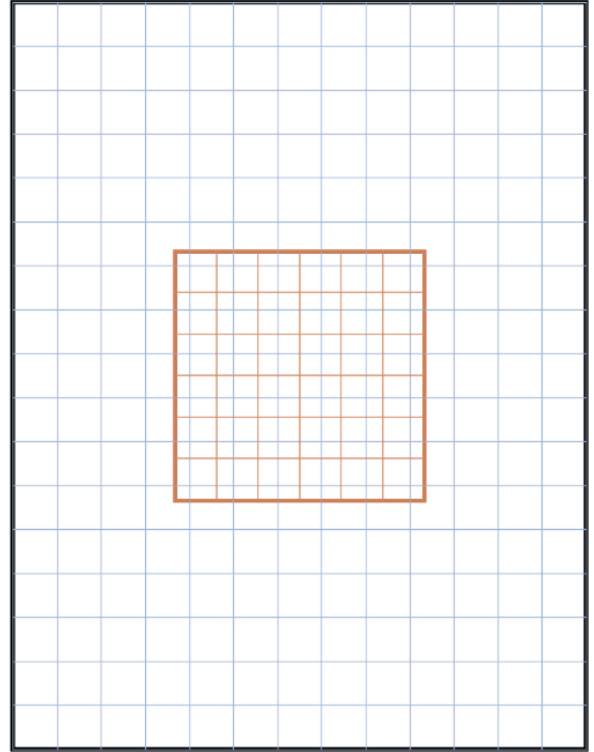


Figure 2

Schematic FE mesh discretisation of (a) STC and (b) NTC cross-sections

Fig. 3 FE-predicted axial load-deformation response

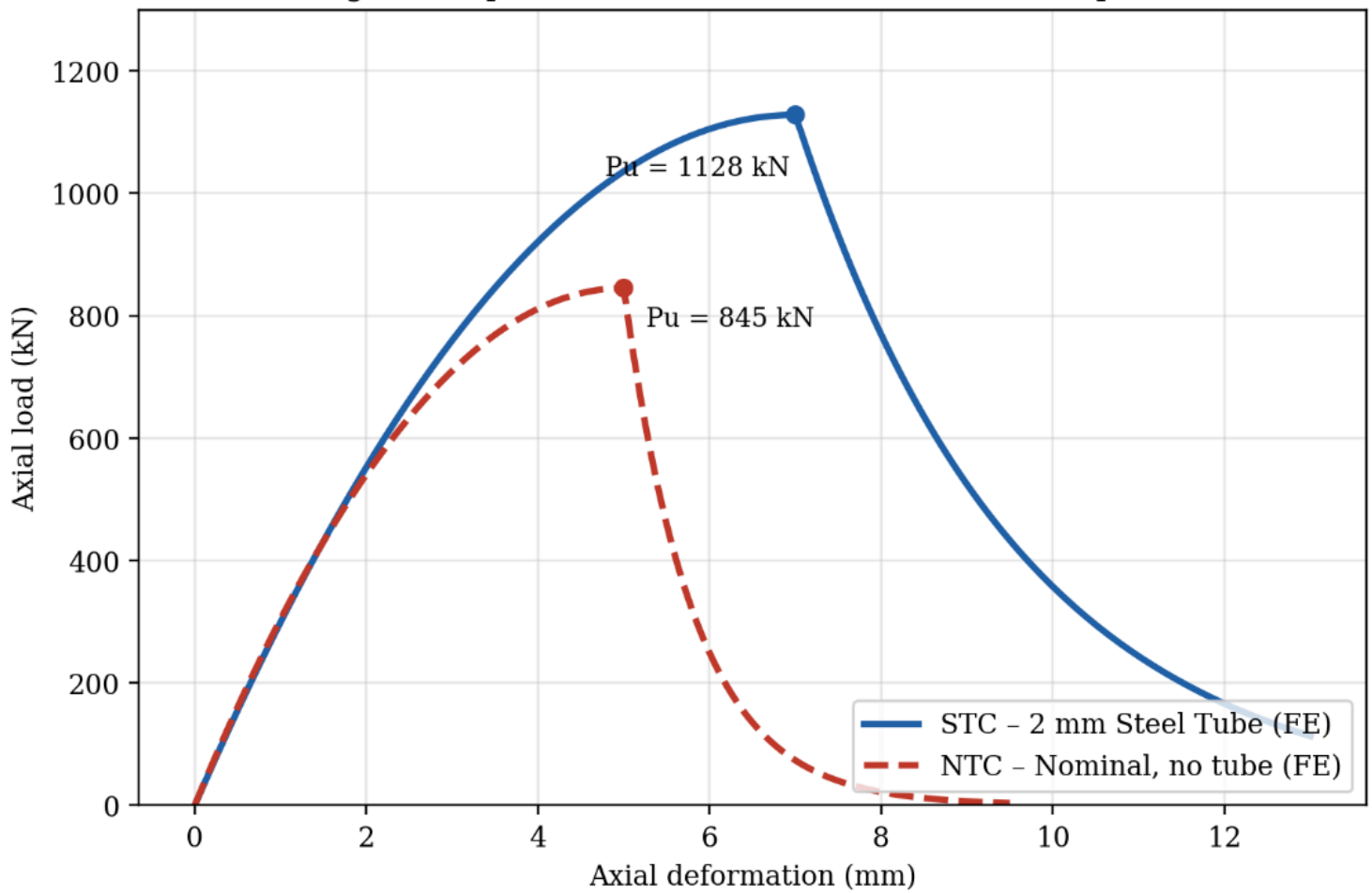


Figure 3

FE-predicted axial load-deformation curves for STC and NTC models

Fig. 5 Comparative FE performance indices

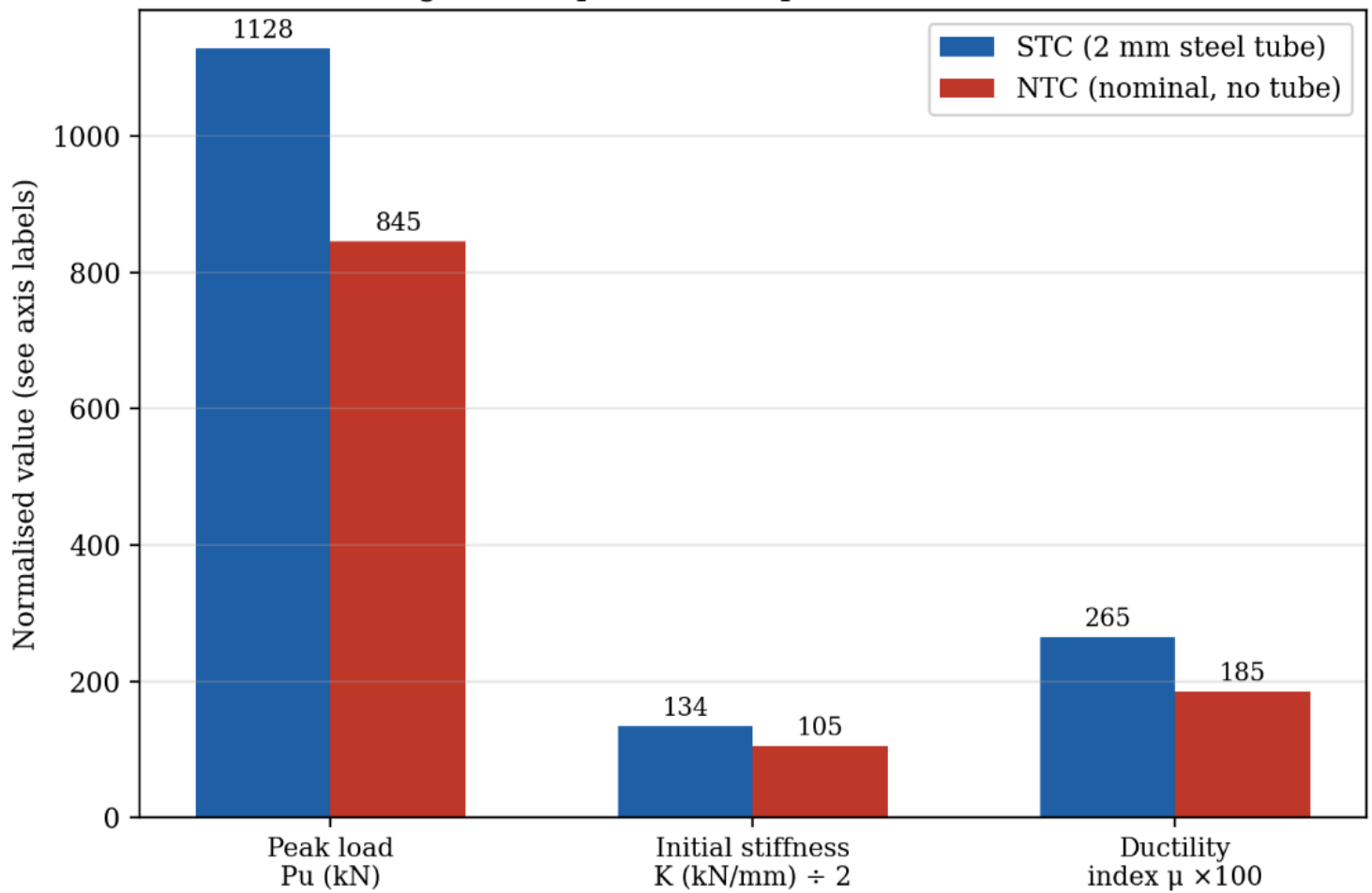


Figure 5

Comparative bar chart of FE-predicted peak load, stiffness, and ductility index